COLD-FORMED STEEL STRUCTURES IN SEISMIC AREA: RESEARCH AND APPLICATIONS

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Abstract. Nowadays, the use of cold formed steel systems is significantly increasing even in countries where traditional structural solutions have always driven the construction sector. This is mainly due to the recognized technical, structural and economical competitiveness of such systems. The rising synergy between research and fabrication has been playing a key role in that, and this is a crucial issue especially for the promotion of cold formed steel structures in seismic area, where many design aspects remain still opened. In line with that, this paper presents the research activities and applications carried out at University of Naples Federico II in the last years. In particular, the topic concerning the seismic behavior of stick built constructions and the development of new structural components and systems will be presented and discussed in details.

Introduction

In recent years, cold formed steel housing (CFS) has been growing in popularity all over the world, because it represents a suitable solution to the demand for low-cost high performance houses. The CFS members, obtained by cold rolling, are produced pressing or bending steel sheets, with thickness ranging between 0.4 and 7 mm. This construction process provide several advantages such as the lightness of systems, the high quality of end products, thanks to the production in controlled environment, and the flexibility due to the wide variety of shapes and section dimensions that can be obtained by the cold rolling process. Moreover the CFS systems, being dry constructions, ensure short execution time. Besides, economy in transportation and handling, the low maintenance along the life time together with the high strength to weight ratio, an essential requirement for a competitive behaviour under seismic actions, represents additional benefits that CFS are able to achieve [1]. In addition, CFS are in line with the requirements of sustainability. Indeed, the use of recyclable and light gauge materials, the flexibility of systems, the dry construction process and the possibility to reuse the elements at the end of the life cycle, contribute to minimize the environmental impacts [2].

In accordance with this trend, over the last decades the research team under the leadership of the Author, at the University of Naples Federico II, has been involved in several research programs focusing on the assessment of the seismic behaviour of low-rise buildings built with CFS members.
In the following, a synthetic presentation of the research activities concerning stick built constructions will be presented in Sections 1 to 7. Moreover, two innovative cold formed steel systems developed in collaboration with an Italian enterprise will be presented in Section 8. Motivations and main results are discussed, referring to a wide bibliography for details. Finally, some further developments are reported.

1. Cold-Formed steel systems

The design of a CFS construction under vertical and horizontal loads can be carried out using two different approaches: “all-steel design” and “sheathing-braced design”. The first one considers only steel members as load carrying elements and it does not take into account the influence of sheathing panels, consequently, the design of CFS members can be strongly influenced by local, global and distortional buckling [3, 4]. On the other hand, the “sheathing-braced design” considers the sheathing as part of the bearing system. Therefore, in case of horizontal loads the “all steel design” requires the introduction of X or K bracing systems in the lateral resisting walls; while according to the “sheathing braced design” the system composed by sheathing, frame and fasteners can assure an adequate strength in order to allow walls to act as in plan diaphragms (Fig.1 and 2).

In the last case, the shear response of a CFS wall, that represents the main lateral force resisting system, is a quite complex concern and depends on the behaviour of its structural components (Fig. 3): sheathings; sheathing-to-frame connections; frame (stud buckling strength); frame-to-foundation connections.

Over the last two decades many theoretical and experimental studies have been addressed to capture the complex behaviour of these structures for improving the current calculation models and design codes, and in particular the research activities carried out at University of
Naples Federico II focuses on the seismic behaviour of stick-built structures laterally braced with sheathings and included:

1. physical tests for the seismic performance assessment of low-rise sheathed CFS constructions;
2. experimental tests on typical materials and wall components (panel-to-frame connections, sheathing panels, screws and hold-down anchors);
3. numerical analysis for the development of an approach for the prediction of seismic wall response;
4. parametric studies for the behaviour factor evaluation;
5. development of a seismic design procedure.

Finally all the results have been applied for the design and construction of an Italian building having seismic resisting system made with CFS members braced by sheathing panels.

![Fig. 3: Factors affecting CFS walls shear behaviour](image)

2. Experimental investigation

The feasibility of using cold-formed steel members in seismic zones and to propose new design methodologies and criteria has been object of past Italian National Research Projects (PRIN 2001 [5] and PRIN 2003 [6]) and is, currently, one of the main topic developed within the Italian University Network Reluis (Reluis 2010-13). In order to reach the goal, a wide experimental campaign on walls and components has been carried out as reported in the following.

2.1 Full scale tests on wall

Full scale tests on walls were carried out on two nominally identical specimens representative of a typical one-story CFS stick-built house (Fig. 4). The generic specimen was made of two 2400 mm long and 2500 mm height walls and a floor of 2000mm span. In particular, the walls were realized by studs, made of C sections spaced at 600mm, and sheathed outside by 9 mm thick oriented strand board (OSB) panels and inside by 12.5 mm thick gypsum wallboard (GWB) panels. Both panel typologies were attached to the frame with screw connections spaced at 150mm at the perimeter and 300 mm in the field. More details on wall tests are
given in [7 and 8]. The two specimens were tested under monotone and cyclic loads. The shear vs. lateral displacement curves are shown in Fig. 5.

Comparing the monotonic and cyclic response, in the monotonic test the panels-to-framing connections collapse mechanism was invariant during the increasing lateral displacement whilst in the cyclic test some modifications (more brittle collapse mechanism) occurred after the peak lateral load was achieved. These modifications produced significant strength degradation in the cyclic test, after the achievement of the peak load, which was stronger than the one observed during the monotonic test (Fig. 5a and b).

On the base of the experimental results, a numerical analysis [9] has been carried out and the main results can be summarized as follow: cold formed steel construction can be designed in order to show an elastic behaviour under design earthquakes (earthquakes having a return period of 475 years); the system provides an adequate ductility (for overstrength) that assures a good safety (limited damages) in case of more severe events (earthquake having a return period of 2475 years) and that all the structural components can be designed according to the capacity design criteria in such way to obtain the failure in sheathing-to-frame connections.

2.2 Tests on wall connecting systems and materials

Considering the strong interrelation between the global lateral response of sheathed cold-formed “stick-built” structures and the “local” shear behaviour of walls components, an experimental campaign was undertaken to investigate the local response of connecting systems
An experimental program for the evaluation of the shear behaviour of fasteners between CFS profiles and sheathing panels was carried out and organized in two phases: in a first phase connections between steel profiles and wood (OSB) or gypsum-based (GWB) panels were tested and in a second phase fasteners between profiles and cement-based (CP) panels were tested. Goals of the testing program were: (1) to compare the response of different panels typologies (wood, gypsum and cement–based panels); (2) to examine the effect of the loaded edge distance; (3) to evaluate the effect of different cyclic loading protocols; (4) to study the effect of sheathing orientation (only for wood-based panels); (5) to assess the effect of the loading rate. It is worth to specify that in the second phase only the first three goals were studied, whilst orientation and effect of loading rate were not studied. A total of 94 specimens, grouped in series composed of 2, 3 or 4, nominally identical specimens were tested.

The generic sheathing-to-profile connection specimen (Fig. 6) consisted of two single 200x600mm sheathings attached to the opposite flanges of 100x50x10x1.0mm C profiles. In particular, one single C-section was placed on the top side, whereas two back-to-back coupled C-sections were used for the bottom side. Sheathings were connected using three screws for the top member (tested connections) and two rows of eight screws for the bottom members (oversized connections).

Three different sheathing types were selected: 9.0 mm thick type 3 OSB [10], 12.5 mm thick standard GWB [11] and 12.5 mm thick CP. Appropriate fasteners for each sheathing typology were adopted: 4.2 × 25 mm (diameter × length) flat head self-drilling screws for OSB sheathings, and 3.5 × 25 mm bugle head self-drilling screws for GWB and CP panels.

The observed failure mechanisms during monotonic tests may be grouped as follows: tilting of screws (Fig. 6b); screws pull-through the sheathing (Fig. 6c); bearing in the sheathing (Fig. 6d); breaking of sheathing edge (Fig. 6e).

As tests results, the sheathing material has a significant effect on the shear connection behaviour. In particular, in case of both monotonic and cyclic tests, the CP provides largest stiffness values, the GWB reveals larger ductility, the OSB// reveals, on average, larger strength and absorbed energy. In the case of OSB panels, the perpendicular-to-grain loaded connections show lower strength, ductility and stiffness compared with parallel-to-grain loading, while the absorbed energy is almost the same for both cases. Moreover, the increment of

![Image of specimen with captions a) Specimen, b) Tilting of screw, c) Pull-through the sheathing, d) Tilting of screw and bearing in the sheathing, e) Breaking of sheathing edge.](image-url)

**Fig. 6:** Shear tests on sheathing to frame connections
the loaded edge distance produces an increment of strength and absorbed energy with an almost linear variation. More details on connection tests are provided in [12 and 13].

The shear and tension of self-drilling self tapping screws with diameters ranging from 4.2 to 6.3 mm have been investigated. Due to the lack of codified test methods, an ad hoc procedure for shear and tension tests has been defined (Fig. 7a). A detailed description can be found in [14]. The required failure mechanism has been reached by each specimen and shear and tension characteristic values, ranging between 4.68 and 13.42 kN for shear, and 7.47 and 21 kN for tension have been evaluated.

Finally, hold-down devices that are usually installed at the end of load bearing walls, in order to avoid overturning phenomena, have been tested. The test set up [15] has been defined in order to reproduce the on-site loading conditions and to appreciable center the axial tension loads (Fig. 7b). In all tests the collapse was due to the rod failure (M24 steel threaded rod), without any device deformation. Moreover, the obtained characteristic value (233 kN) was about 27% higher than the minimum nominal collapse load given by the rod manufacturer. In order to fully investigate the seismic response of CFS walls, the shear behaviour of wall materials have been examined. Therefore, shear tests have been carried out on sheathing panels. In particular, the ultimate shear strength and shear modulus of the OSB, GWB and CP sheathing have been investigated. The tests have been carried out in agreement with the ASTM D1037 Standard [16] (Fig. 7c). The results showed that OSB panels revealed the largest strength (about 7 MPa), with quite scattered results due to the anisotropic material, while the CP panels provided the largest shear modulus. Instead, the GWB panels showed the minimum shear strength and shear modulus [13].

### 3. Numerical analysis

An analytical-numerical model for the prediction of the shear force vs. lateral displacement response of sheathing CFS shear walls has been developed taking into account the results of the presented experimental campaign [17]. The method is based on some theoretical assumptions about the fundamental kinematics of shear walls (Fig. 8), making use of experimental load-displacement response curves for frame-to-panel connections. Basic assumptions made for the proposed theoretical model are: (1) local failure of sheathing-to-wall framing connections controls the global collapse mode; (2) studs and tracks are rigid and hinged to each other; (3) relative displacements between the sheathing and framing are small compared with
the panel size; (4) the edges of the panel are free to rotate without interference from adjacent sheathings and the foundation or other stories; (5) the wall is fully anchored to the foundation or lower storey; (6) the wall framing deforms into a parallelogram and the relative frame-to-panel displacements are determined based on a rigid body rotation of panels; (7) only shear deformation of the sheathings is considered by adopting the equation for shear deformation of a thin, edge-loaded, plate; (8) the load-displacement curve of the sheathing-to-frame connections is schematized by using the relationship proposed by Richard & Abbott.

A preliminary assessment of the load vs. deflection response curve prediction obtained by applying the proposed model has been carried out considering the previously presented experimental results of full scale tests on walls and shear tests on connections. Figure 9 shows the comparison between experimental and numerical response in terms of unit shear load ($V$) vs. lateral displacement ($d$) curves. From this Figure, it can be noticed that the proposed analytical method provides a result which seems accurate enough in comparison with the experimental response. In fact, numerical and experimental curves are very close for $d < 4\text{mm}$, while the proposed model slightly underestimates the displacements for $d > 4\text{mm}$. In particular, the predicted-to-test ratios of shear strength, conventional elastic deflection and peak deflection are 0.98, 1.02, 0.86, respectively, which appear very good values.
4. Behaviour factor evaluation

In order to evaluate the behaviour of some typologies of sheathed CFS walls a large parametric study involving 72 different wall configurations has been carried out. All the walls are made of typical CFS frames with lipped channel-section studs spaced at 600 mm sheathed with GWB panels on both sides (G+G) or GWB on one side and OSB panels on the other side (G+O). Sheathing panel material, wall geometry (height and length) and external screw spacing have been varied as summarized in Table 1. For each wall configuration obtained by combining those parameters, the stud thickness and hold-down device typology have been selected in such a way to promote the sheathing fasteners collapse.

<table>
<thead>
<tr>
<th>Sheathing panel typology</th>
<th>GWB + GWB (G+G), GWB + OSB (G+O)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wall height ((h) [\text{mm}])</td>
<td>2400, 2700, 3000</td>
</tr>
<tr>
<td>Wall length ((l) [\text{mm}])</td>
<td>1200, 2400, 9600</td>
</tr>
<tr>
<td>External screw spacing ((s) [\text{mm}])</td>
<td>50, 75, 100, 150</td>
</tr>
</tbody>
</table>

One-story buildings have been considered as case studies. They refer to a stick-built construction in which both floors and walls are realized with CFS framing sheathed with structural panels. In particular, in order to obtain a large range of solutions, a schematic construction has been considered with wall length \((L)\) variable between 3 and 7 m and lengths of full height (resisting) wall segment \((l)\) in the range \(l=0.4L\) through \(l=0.7L\) (Fig. 10). Unit weights ranging from 0.4 to 1.5 kN/m² and from 0.3 to 1.2 kN/m² have been considered for floors and walls, respectively. Moreover, the building has been considered without and with attic. In the first case, a variable live load of 2.0 kN/m² has been considered and, in the latter case, a snow variable load ranging from 0.60 to 1.20 kN/m² has been added. As result, 7 seismic weights per unit wall length have been considered (from 10 to 40 kN/m) and have been applied to 72 wall configurations [18].

![Fig. 10: The schematic case study construction](image)

In order to develop a non-linear dynamic seismic analysis, the seismic inputs have been selected in such a way that they could cover all the soil typologies classified by Eurocode 8 [19]. Therefore, 21 earthquake records have been selected from the European strong-motion database. For each soil type 7 accelerograms have been considered so that the shape of the average elastic response spectrum is close as much as possible to the shape of the corresponding Eurocode 8 elastic acceleration spectrum.
Each wall defined in the parametric study has been schematized as a single degree of freedom structure, in which the hysteretic behaviour under horizontal loads is described by a hysteretic law deriving from the model previously presented. In order to account for the second order effects, a vertical load equal to the 100% of the mass has been considered. Moreover, the viscous damping ratio has been set equal to 5%. Figure 11 shows typical curves obtained with the well-known incremental dynamic analysis (IDA) procedure for a fixed wall configuration and different accelerograms.

The numerical results of performed dynamic analyses have been interpreted by considering three different limit displacements on the generic response curve (Fig. 12): the peak \( d_p \) and ultimate \( d_u \) displacements, and the yielding displacement of the idealized bilinear curve \( d_y \) derived according to an equivalent energy elastic-plastic approach. For each IDA curve the seismic intensity measures \( S_{a,y} \), \( S_{a,p} \) and \( S_{a,u} \) corresponding to the limit displacements \( d_y \), \( d_p \) and \( d_u \), respectively, have been evaluated and these spectral accelerations have been used to define three different behaviour factors: \( q_1 = S_{a,p} / S_{a,y} \), \( q_2 = S_{a,u} / S_{a,p} \), \( q_3 = S_{a,u} / S_{a,y} \), which take into account the overstrength, the ductility and both overstrength and ductility.

In order to obtain an assessment of the behaviour factors \( q_1 \), \( q_2 \) and \( q_3 \) on the basis of significant IDA results, only those representing realistic design conditions have been selected [20]. The peculiarity of the proposed approach is to provide the possibility to achieve an “enhanced objective” [21], consisting of the following goals: (1) immediate occupancy (IO) performance level for earthquakes having 50% probability of exceedance in 50 years (50%/50); (2) life safety (LS) performance level for earthquakes having 10% probability of exceedance in 50 years (10%/50); (3) collapse prevention (CP) performance level for earthquake with 2% probability of exceedance in 50 years (2%/50). The results of the extensive parametric seismic study, involving about 500,000 single IDA analysis, in terms of behaviour factor are shown in Table 2. In particular, a behaviour factor equal to 1 should be considered for the immediate occupancy level, while \( q_1 = 2 \) and \( q_3 = 3 \) could be used for the life safety and normal collapse levels, respectively.
Table 2: Behaviour factors for “Multi-performance” approach

<table>
<thead>
<tr>
<th>Wall configuration</th>
<th>$q_1$</th>
<th>$q_2$</th>
<th>$q_3$</th>
</tr>
</thead>
<tbody>
<tr>
<td>G+G Average</td>
<td>2.23</td>
<td>1.35</td>
<td>3.05</td>
</tr>
<tr>
<td>St. Dev.</td>
<td>0.44</td>
<td>0.17</td>
<td>0.83</td>
</tr>
<tr>
<td>C.o.V.</td>
<td>0.20</td>
<td>0.13</td>
<td>0.27</td>
</tr>
<tr>
<td>G+O Average</td>
<td>2.35</td>
<td>1.23</td>
<td>2.88</td>
</tr>
<tr>
<td>St. Dev.</td>
<td>0.40</td>
<td>0.12</td>
<td>0.57</td>
</tr>
<tr>
<td>C.o.V.</td>
<td>0.17</td>
<td>0.10</td>
<td>0.20</td>
</tr>
<tr>
<td>All types (G+G and G+O) Average</td>
<td>2.29</td>
<td>1.29</td>
<td>2.96</td>
</tr>
<tr>
<td>St. Dev.</td>
<td>0.43</td>
<td>0.16</td>
<td>0.71</td>
</tr>
<tr>
<td>C.o.V.</td>
<td>0.19</td>
<td>0.12</td>
<td>0.24</td>
</tr>
</tbody>
</table>

5. Proposal of a design procedure

A seismic design procedure refers to CFS walls sheathed with wood-based or gypsum-based panels, has been developed to propose a design tool [22 and 23]. In the proposed approach the wall components are designed in such a way to promote the sheathing fastener failure by following three consecutive phases: (1) definition of the wall geometry (wall height $h$, stud spacing $c$) and others “assigned” design parameters (type, thickness, and orientation of the sheathing panels; type, interior spacing $p$ and edge distance $e$ of the sheathing fasteners; type of frame fasteners; steel grade, stud size (except for the thickness $t$) and track size; hold-down anchor type; shear anchor type); (2) evaluation of the sheathing fastener exterior spacing $s$; (3) evaluation of the others “calculated” design parameters (stud thickness $t$, hold-down anchor diameter $d_a$, shear anchor spacing $a$).
In this design process, the selection of the “assigned” design parameters (phase 1) does not depend on the seismic design, but usually derives from architectural and technological choices and design for vertical loads. Moreover, only the assessment of the sheathing fasteners exterior spacing \((s)\) directly derives from seismic analysis results (phase 2), while the definition of stud thickness \((t)\), hold-down anchor diameter \((d_a)\), and shear anchor spacing \((a)\) is carried out only on the basis of “capacity design” criteria (phase 3).

The screw spacing can be evaluated by a Linear Dynamic analysis (LD), a Non-linear Static analysis (NS), or a Non-linear Dynamic analysis (ND).

When the linear dynamic (LD) procedure is selected for the seismic analysis, a force-based design approach is usually used, in which the inelastic behaviour and the structural overstrength are taken into account by the seismic force modification factors. In this case, the comparison between seismic capacity and demand (Fig 14a), in terms of forces, shall satisfy the following equation:

\[
H_C \geq H_D
\]  

where \(H_C\) and \(H_D\) are the seismic strength capacity and seismic action demand, respectively.

In particular, the seismic action \((H_D)\), which represents the horizontal in-plane force acting on the wall, can be evaluated through the following well known relationships:

\[
H_D = S_a \cdot \frac{w}{g}; \quad S_a = f_{Sad}(T, \xi) \quad T = 2\pi \sqrt{\frac{w}{g}K}
\]  

where: \(S_a\) is the spectral acceleration; \(f_{Sad}T\) is the spectral acceleration function; \(T\) is the structural period; \(\xi\) is the viscous damping ratio; \(w\) is the seismic weight; \(g\) is the gravity acceleration; and \(K\) is the wall shear stiffness, which can be expressed as a function of the sheathing fasteners exterior spacing \((s)\).

When the seismic analysis is performed by means of the nonlinear static (NS) procedure, the inelastic behaviour and the structural overstrength are directly considered and the comparison between seismic capacity and demand can be achieved in terms of displacements (displacement-based approach, Fig. 14b):

\[
d_C \geq d_D
\]  

where \(d_C\) and \(d_D\) are the seismic displacement capacity and seismic displacement demand, respectively. This comparison can readily be performed by means of the well known acceleration-displacement spectrum, in which the demand and capacity spectra are represented together. In particular, the demand spectrum can be obtained as follows:

\[
S_a = f_{Sad}(d, \xi)
\]  

where \(f_{Sad}\) is the spectral acceleration function, and \(d\) is the generic displacement. The capacity spectrum, instead, can be represented by an elastic-plastic curve, which is drawn by defining the yield \((Y)\) and ultimate \((U)\) limit points:

\[
Y\left( d_y, \frac{H_C}{w}g \right), U\left( d_u, \frac{H_C}{w}g \right)
\]  

where the yield displacement \((d_y)\) and ultimate displacement \((d_u)\) can be obtained as function of the exterior spacing \((s)\).

When the non-linear dynamic (ND) procedure is adopted for seismic analysis, the comparison between seismic capacity and demand can be obtained in terms of displacements (Fig.14c), as well as in the NS analysis, but the displacement demand can be achieved by means of incremental dynamic analysis, in which the results can be given in terms of displacement demand \((d_D)\) vs. peak ground acceleration \((a_g)\) curve. In this case, the displacement
capacity corresponds to a given limit displacement (i.e. ultimate displacement $d_u$) on wall response curve, which can be expressed as a function of external screw spacing ($s$):

$$d_c = f_c(s)$$

(6)

Therefore, for a fixed wall condition, with exception of the external screw spacing (which is the design parameter), if the function $f_c(s)$ and the relevant IDA curves are known, then for each value of external screw spacing ($s$), both displacement demand ($d_D$) and capacity ($d_C$) can be evaluated and compared.

$$d_D = f_D(s)$$

Therefore, for a fixed wall condition, with exception of the external screw spacing (which is the design parameter), if the function $f_c(s)$ and the relevant IDA curves are known, then for each value of external screw spacing ($s$), both displacement demand ($d_D$) and capacity ($d_C$) can be evaluated and compared.

Finally, in order to complete the wall design, the other structural components can be defined by “OC” nomographs (Fig. 15), in which the wall shear strengths per wall unit length ($\overline{H_C}$) corresponding to the resistance of sheathing fasteners ($\overline{H_{C,sd}}$), studs ($\overline{H_{C,s}}$), hold-down anchors ($\overline{H_{C,hm}}$) and shear anchors ($\overline{H_{C,sa}}$) are represented together as function of the exterior spacing ($s$).

![Fig. 15: Schematic figure of the OC design nomograph](image)

6. Application

In the last years, all the results of the summarized scientific works found a real application in designing and execution of a new school for the British Command of Defense Estate in
Naples. In fact, the need to realize in short time a construction of strategic importance that could provide the highest performance in terms of safety, durability and seismic reliability required the work of 11 experts coming from the several different design areas. The construction covering an area of 3000m² has been organized in 8 jointed buildings. Six of them are stick-built cold formed steel constructions braced with (OSB) panels and accommodate classrooms and services. The extensive use of stick built system for this construction has been adopted mainly for the high strength to weight ratio, lightness, short execution time, quality of products, possibility to use eco-friendly materials, easy integration of termic, idraulic and electrical systems and large flexibility. These peculiarities matches the clients requirements and allow a advanced performance building to be built up in short time and with high environmental performance. In particular, the walls (Fig. 16) are made with studs having 150×50×20×1.50 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 centre and they are sheathed with vertically oriented 9.0 mm thick OSB/3 panels on both faces. Sheathing panels are connected to steel framing 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 purposely designed hold-down devices are placed at the ends of each shear wall segment.

The roof structures (Fig. 17) are made of 300×50×20 mm joists with thickness varying from 1.5 mm to 3 mm depending on the span, and placed in line with the studs. Subfloor sheathing is made of 18 mm thick OSB/3 panels, adequate flat straps and blocking have been introduced in order to laterally brace the joists flanges.

The construction (Fig. 18), realized in about one and an half year represents an “unicum” in the Italian construction sector, and it has been designed in agreement with all the research results and taking into account the lack of specific regulations in the National codes. On the other hand, this last, brought to a cumbersome verification stage that required on-site shear tests on two identical wall specimens and more than hundred component tests.

More details about design, constructions and experimental campaign are provided in [24] and [25].
7. **Innovative systems**

The wide variety of shapes, that can be obtained by cold rolling processes together with the high prefabrication levels allow several different clients requirements to be satisfied. In this perspective, the development of new structural components and systems is always a key aspect in the CFS construction market. In the following, two paradigms of this trend are presented. Both of them, the MLC Beam and the MPN system, have been developed in cooperation with an Italian enterprise.

7.1 **The MLC Beam**

The structural member is patented by the Ben Vautier S.p.a. with the name of Modular Lightweight Cold-formed beam (MLC beam). The MLC beam is a part of a full integrated structural system designed by Ben Vautier which takes advantage of the main features of cold-formed technology, the MLC beams have been designed to prevent early buckling phenomena that could affect the structural behaviour when thin sheets are used [26]. In particular, these members provide high resistance to torsional–flexural buckling, which is very useful when the floor is assembled as a dry construction system and there are no lateral restraints for the main beams.

The I-section with hollow flanges of the MLC beam (Fig.19) is fabricated from two special C-profiles back to back joined with connections which are located on the web and on the flanges. Two reinforcing plates are placed inside the top and bottom hollow flanges of the I-section, providing an additional connection system between the two C-profiles. In addition, varying the thickness and steel grade of such plates, the performance of the beams can be enhanced as well. The shape of MLC beam is characterized by edge flange stiffeners, web beads and web openings. For a specific steel grade, it is possible to obtain members which provide different load bearing capacity by properly modifying reinforcing plate thickness and the overall cross-section dimensions. In particular, as far as the beam cross-section depth H is concerned, three member classes with different performances have been identified: LH (H=200 mm), MH (200<H<300), HH (H=4300).
Over the time, different connection systems have been studied through a wide comparative analysis carried out by means of lap-shear and U-tension tests, including both mechanical fasteners and laser welds, in order to select the most suitable system.

The performed experimental tests [27] showed that the bicomponent blind rivets are the best connection system for assembling the MLC members, both in terms of strength and ultimate displacement performances. Although these rivets provide suitable strength for connections, they present technological and production limitations during the manufacturing process of the beams. These problems are mainly related to the high number of connections which have to be applied to the beams for preventing local buckling phenomena. Moreover, in this case the need to drill the holes for the rivets increases the production time considerably.

With the aim of reducing the manufacturing costs and the production time, the laser welding technology has been finally selected. In this case, the beams could be fabricated with a robotic system and the different parts could be connected in a single step by eliminating the drilling process. Moreover, the use of laser welding should allow the structural performances of the members to be increased, adopting suitable laser weld spacing on the flanges in order to completely prevent local buckling phenomena (Fig. 20).

The laser beam welding (LBW) technology is a process that allows steel sheets to be joined by means of a coherent radiation focused and delivered to the surface of the workpiece. Lasers generate light energy that heats the materials to a molten state and fuse them together. The advantages of these techniques are the high quality and precision, the minimal amount of heat and the high degree of automation and productivity. The reduced dimensions of the spot allow to join small parts with very narrow welds. The input energy is just equal to that tightly necessary to the fusion of the material producing a very small heat-affected zone. The advantages are the low thermal distortion and residual stresses.

As far as the mechanical parameters are concerned, the steel grade S235 and quality JR was chosen for all the test specimens. The effect of zinc coating on the ultimate strength of laser welds was taken into account including in the test program galvanized specimens also. The thickness of the coating layer was 10 mm.

Finally, different weld shapes and direction angles were considered. Ninety specimens divided in thirty symmetrical, thirty asymmetrical and thirty U tension samples the same incident laser power of 5.5kW was adopted for all the specimens.
On the basis of obtained experimental results, the staggered configuration of welds on the flanges with spacing equal to 100mm has been selected for assembling the investigated beams. The performed tests will be used for the validation of a detailed finite element model in order to simulate the non-linear behaviour of the selected beams under different load conditions and to improve the design of the members. Further developments of the study will be also concerned with the use of high-strength steels.

### 7.2 The MPN system

The MPN construction system developed and marketed by Ben Vautier Spa is based on a “Steel Brick” that represents the three dimensional basic module and consists of cold-formed steel profiles connected by mechanical devices of aircraft origin, known as blind rivets. The construction system is composed by 3 brick typologies (Fig.21): standard, half and a quarter of steel brick. The standard brick is the main element of bidimensional sub-structures like walls and floors and is a parallelepiped with base 600x600mm and 300 mm height. It is made of 4 studs jointed by 2 frames and braced by 4 couples of diagonals. The bricks are connected together by an octagonal cover plate placed on each vertex.

On the contrary to traditional masonry constructions, the steel brick is able to work under both tension and compression, for this reason, in the last thirty years, starting from 1965, it has been adopted to realize both walls and floors to be applied for whole structural systems as housing, factory buildings and roof systems. The MPN system has been subjected to 4 generations and 6 patents [28] and [29].
At the moment, the MPN system is out of trade economical reasons, therefore it is now under investigation to be proposed in a new configuration for housing and vertical additions. In order to reach this goal, the steel brick members performance have been evaluated in agreement with the current National [30] and European codes. In particular, the brick structural behavior has been investigated considering the global geometry fixed (height and base dimensions) and varying members thickness ($t_1 = 1.2$ and $t_2 = 2$ mm) and steel grade (S235, S275 e S355). In the following table the strength for each brick member in case of tension ($N_{t,Rd}$), compression ($N_{c,Rd}$) and compression accounting the buckling phenomena ($N_{b,Rd}$) are shown.

### Table 3: Evaluation of members strength

<table>
<thead>
<tr>
<th>Thickness</th>
<th>Member</th>
<th>Steel grade S235</th>
<th>Steel grade S275</th>
<th>Steel grade S355</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$N_{t,Rd}$</td>
<td>$N_{c,Rd}$</td>
<td>$N_{b,Rd}$</td>
<td>$N_{t,Rd}$</td>
</tr>
<tr>
<td>mm</td>
<td>kN</td>
<td>kN</td>
<td>kN</td>
<td>kN</td>
</tr>
<tr>
<td>1.2</td>
<td>Single Frame</td>
<td>42.7</td>
<td>31.1</td>
<td>25.4</td>
</tr>
<tr>
<td></td>
<td>Double Frame</td>
<td>85.4</td>
<td>72.4</td>
<td>60.1</td>
</tr>
<tr>
<td></td>
<td>Diagonal</td>
<td>14.1</td>
<td>14.1</td>
<td>7.61</td>
</tr>
<tr>
<td></td>
<td>Single Stud</td>
<td>64.8</td>
<td>51.2</td>
<td>51.2</td>
</tr>
<tr>
<td></td>
<td>Double Stud</td>
<td>129.2</td>
<td>101.7</td>
<td>101.7</td>
</tr>
<tr>
<td>2</td>
<td>Single Frame</td>
<td>73.9</td>
<td>58.8</td>
<td>46.4</td>
</tr>
<tr>
<td></td>
<td>Double Frame</td>
<td>147.8</td>
<td>127.9</td>
<td>106</td>
</tr>
<tr>
<td></td>
<td>Diagonal</td>
<td>24</td>
<td>24</td>
<td>11.7</td>
</tr>
<tr>
<td></td>
<td>Single Stud</td>
<td>103.4</td>
<td>82.5</td>
<td>82.5</td>
</tr>
<tr>
<td></td>
<td>Double Stud</td>
<td>206.8</td>
<td>164.2</td>
<td>164.2</td>
</tr>
</tbody>
</table>

Starting from the steel brick performance, design tables for a first structural dimensioning of a new tubular scheme under both vertical and horizontal loads have been developed [31]. These last could be adopted for single-family houses and/or raising system of existing buildings (new structural concept). In particular, the design tables have been applied to a new concept recently developed by the architect Silvio D’Ascia [32] and defined as "Cube House"
The Cube House is a tubular system with plan dimensions equal to 7.8 x 7.8 m and 6.9 m height, where the vertical bearing system is composed by two walls and two floors, completely realized by the MPN system.

8. Conclusions

The widely recognized structural performance provided by CFS systems together with the high levels of prefabrication, safety, durability and sustainability, are spreading this construction system all over the world. At the same time, the actual lack in specific design codes, mainly for the applications in seismic area, requires the development of new research in the field. In line with the foreseen advance, several researches have been carrying out in the last years at the University of Naples as briefly summarised in the paper.

Acknowledgments

The Author is grateful to all the people and the companies who made the development of these projects possible. A special thank to Dr. G. Di Lorenzo, Dr. L. Fiorino, Dr. O. Iuorio and Dr. F. Portioli for their important contribution to all the relevant research activities.
References


