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**Seismic Design Method for CFS Diagonal Strap-Braced Stud Walls: Experimental Validation**

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## ABSTRACT

The search for innovative methods to ensure high structural, technological and environmental performance is an important issue in the development of new constructions. Among the several available building systems, constructions involving the structural use of Cold-Formed Steel (CFS) profiles represent an efficient and reliable solution. In an effort to characterize the seismic response of CFS structures and to support the spreading of these systems, a theoretical and experimental research has been carried out at University of Naples Federico II within the Italian research project RELUIS-DPC 2010-2013. It focused on the "all steel design" solution, in which CFS diagonal strap-braced stud walls are the main lateral resisting system. In order to overcome the lack of information in the current European codes, a critical analysis of the requirements for these systems provided by the AISI S213-2007 has been carried out by comparing them with those given by Eurocodes for hot-rolled X-braced steel frames (tension-only). On the basis of the design hypothesis outlined from this analysis, a case study has been developed with the aim to define an extended experimental campaign involving 12 tests on full-scale CFS diagonal strap-braced stud walls. Finally, on the basis of experimental results, the assumed design prescriptions and requirements, such as the force modification factor and the capacity design rules, have been verified.

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## INTRODUCTION

The Cold-Formed Steel (CFS) structures are able to ensure a good structural response in seismic areas. In these structures, the lateral load bearing systems are CFS stud walls, that are generally realized with frames in CFS profiles braced by sheathing panels (“sheathing-braced” systems) or light gauge steel straps installed in a X configuration (“all-steel” systems). The search for innovative buildings to ensure high structural, technological and environmental performance is promoting the development of light gauge steel structural systems (Landolfo, 2011).

Despite the several advantages related to their use, the main European structural code for seismic design, the Eurocode 8 Part 1 – EN 1998-1 (CEN, 2005), does not provide any prescription for the seismic design of CFS structures. Presently, the AISI S213-2007 “North American Standard for Cold Formed Steel Framing - Lateral Design” (AISI, 2009) represents the main reference for the design of this structural typology under seismic actions. This document codifies the design of seismic resistant CFS systems for Canada, Mexico and United States for both “sheathing-braced” and “all-steel” systems. In particular, design provisions for strap-braced stud walls in terms of force modification factor and capacity design approach are based on the research carried out by Serrette (1997), Al-Kharat and Rogers (2005, 2006, 2007), Comeau (2007) and Velchev (2008). An evaluation of seismic requirements of AISI S213 was carried out by Velchev et al. (2010).

In the last decade, many research activities on the CFS structures were undertaken at University of Naples “Federico II”. These studies manly focused on the assessment of seismic behaviour of such construction systems designed according the “sheathing-braced” approach (Landolfo et al., 2006; Della Corte et al., 2006; Fiorino et al., 2007; Landolfo et al., 2010, Fiorino et al., 2012a, Fiorino et al., 2012b, Fiorino et al., 2014, Iuorio et al. 2014).

This topic was also studied in several European researches, in which monotonic and reversed cyclic tests on different configurations of strap-braced stud wall prototypes have been performed (Fülöp and Dubina, 2004; Tian et al., 2004; Casafont et al., 2007).

As an effort to define the seismic design criteria for “all-steel” CFS systems, a theoretical and experimental study aiming to investigate the seismic behaviour of diagonal CFS strap-braced stud walls, simply named in the following as "strap-braced walls", has been carried out within the RELUIS–DPC 2010-2013 Italian research project. The research included a wide experimental campaign and theoretical analyses in order to define seismic design criteria for strap-braced CFS structures.

## AISI S213 STRAP-BRACED WALLS VS. EUROPEAN HOT-ROLLED TENSION-ONLY X-BRACED FRAMES SEISMIC PRESCRIPTIONS

The applicability of a structural system in a seismic area is related to the clarity and the interpretation of design prescriptions. In order to identify the peculiarities of the seismic design of strap-braced stud walls, the prescriptions provided by the AISI S213 have been examined. Although there are significant difference between cold-formed and hot-rolled steel structures, the AISI prescriptions have been compared with those provided by EN 1998-1 for hot-rolled steel X-braced frames. The latter ones represent a seismic resistant system similar to the investigated one, because both systems consider tension-only diagonals as dissipative elements. This comparison aims to define the design issues of the strap-braced stud walls and to individuate the analogies and differences with the design rules of hot-rolled steel braced systems provided by Eurocodes, with the objective of introducing specific prescriptions for strap-braced walls according to the European design philosophy. The comparison of the two codes is described in the following.

The prescribed value of the force modification factor (R) or behavior factor (q), using the European terminology, provided by EN 1998-1 for hot-rolled X-braced steel frames is equal to 4 in the case of buildings that are regular in elevation, have seismic resistant systems running without interruption from foundation to the top roof, and do not present abrupt changes of mass and stiffness between the different storeys or significant setbacks. On the other hand, the AISI S213 for Canada defines the R factor for CFS buildings as the product of ductility related factor, $R_d$, and overstrength related factor, $R_o$. In particular, the AISI defines two categories of seismic-resistant systems. A first category called “Limited ductility braced wall” follow the rules of capacity based design approach assuming that the braces act as the energy-dissipating element (gross cross-section yielding). For the “Conventional construction”, the capacity design approach is not required and the seismic resistant system is not specifically detailed for ductile performance. In the case of

“Limited ductility braced wall”, the AISI S213 provides R equal to 2.5 ($R_o$= 1.3 and $R_d$= 1.9) while, for “Conventional construction” category, the R factor is equal to 1.6 ($R_o$= 1.3 and $R_d$= 1.2). In the case of United States, the seismic modification factor should be taken equal to or less than 3 according to the applicable building code for non-detailed systems, while greater values can be assumed for structures designed through the capacity design approach. For the latter ones, the American code ASCE-07 (ASCE/SEI, 2010) provides a factor value equal to 4.

It has to be noticed that, contrarily to EN 1998-1 provisions for hot-rolled structures, the AISI S213 does not provide any prescriptions about the diagonal global slenderness, since the diagonals adopted in the examined system are straps which are not able to resist to any compression loads. In fact, the AISI S213 expressly allows global slenderness values for strap members exceeding 200. Moreover, the AISI S213 does not provide any limitations for local (cross-section) slenderness, because studs and tracks are generally made of “slender” CFS cross-sections (Class 4, according to Eurocode classification).

In both CFS and hot-rolled X-bracing systems, the tension diagonal is the energy-dissipative element. Its yielding represents the most ductile failure mechanism and the rupture of the net cross section at fasteners holes should be prevented. Therefore, the prescriptions provided by EN 1998-1 (Eq. 1) and AISI S213 (Eq. 2) can be compared:

$$\frac{A_{net}}{A} \geq 1.1 \cdot \frac{\gamma_{M2}}{\gamma_{M0}} \cdot \frac{f_y}{f_u} = \alpha \cdot \frac{f_y}{f_u} \quad \text{with} \quad \alpha = 1.1 \cdot \frac{\gamma_{M2}}{\gamma_{M0}} = 1.38 \qquad (1)$$

$$\frac{A_{net}}{A} \geq \frac{R_y}{R_t} \cdot \frac{F_y}{F_u} = \beta \frac{F_y}{F_u} \quad \text{with} \quad \beta = \frac{R_y}{R_t} \qquad (2)$$

Nevertheless the codes have approaches conceptually different, both are inclined to avoid the diagonal failure at fasteners holes. In particular, the EN 1998-1 (Eq. 1) compares the design resistance of the gross and net section resistances by considering the partial safety factors and the effect of the hardening through the coefficient 1.1. On the other hand, the AISI S213 (Eq. 2) compares the expected resistances by introducing the $R_y$ and $R_t$ coefficients, which are obtained on the base of a survey on mills by North American CFS producers and are representative of actually produced steels. These coefficients are not provided for European steel in Eurocodes, but there is an analogy between $R_y$ by AISI and the factor $1.1\gamma_{ov}$ given by Eurocode, in which 1.1 represents the hardening effect and $\gamma_{ov}$ is the material overstrength factor defined as the ratio between the average and characteristic values of the yield strength. As conclusion, Eurocode compares factored design resistance whereas AISI S123 compares expected values of resistance. Despite this conceptual difference, $\alpha$ and $\beta$ coefficients can be compared to evaluate the safety level of the two prescriptions and it can be noted that the coefficient $\alpha$=1.38 represents an upper limit and it is conservative with regard to the coefficient $\beta$ values, which ranges from 1.00 to 1.27.

For the design of non-dissipative elements (beams, columns and connections), the examined codes provide different capacity design rules. In particular, the EN 1998-1 provision considers that the seismic forces acting in the non-dissipative elements are those corresponding to the first plastic event in the diagonals:

$$N_{pl,Rd}\left(M_{Ed}\right) \geq N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \tag{3}$$

Taking into account the i$^{th}$ diagonal and the relevant $\Omega_i$, the fulfilment of this equation consists in designing the non-dissipative elements for a force corresponding to the attainment of the plastic resistance of the tension diagonals. In this case, the application of Eq. (3) for beams and columns would be the same as the use of equation proposed for the design of connections:

$$F_{C,Rd} \geq 1.1 \cdot \gamma_{ov} \cdot \frac{A \cdot f_{yk}}{\gamma_{M0}} \tag{4}$$

In addition, in order to obtain a uniform dissipative behaviour and to promote a global mechanism, in the case of buildings with more than two storeys, the code requires that the maximum overstrength factor ($\Omega_i$) does not differ from the minimum one by more than 25%. On the contrary, AISI S213 requires that these elements have to resist the force corresponding to the expected yield strength of the diagonal according the following equation:

$$A_g \cdot R_y \cdot F_y \tag{5}$$

Therefore, the fulfilment of the capacity design principles consists in designing the non-dissipative elements, at each level, by considering the plastic resistance of the relevant ductile element (diagonal in tension). In order to compare the capacity design rules provided by the two codes, the Eq. (4), assumed as general formulation for EN 1998-1, can be written as follows:

$$1.1 \cdot \gamma_{ov} \cdot \frac{A \cdot f_{yk}}{\gamma_{M0}} = \delta \cdot A \cdot f_{yk} \text{, with } \delta = 1.1 \cdot \frac{\gamma_{ov}}{\gamma_{M0}} = 1.38 \tag{6}$$

It has to be noticed that the mathematical meaning of the $\delta$ coefficient is the same of $R_y$ in Eq. (5). In particular, the $\delta$ coefficient is constant and equal to 1.38, while $R_y$ depends on the steel yield strength ($f_y$) and ranges from 1.1 to 1.5. The comparison of the two coefficients shows that the coefficient $R_y$ decreases with the increasing of the yield strength and it is higher, then conservative, than $\delta$ for low values of yield strength (from 230 to 255MPa).

By comparing the capacity design prescriptions, it can be noticed that both codes are oriented to promote a global failure mechanism. In particular, the EN 1998-1 attempts to obtain a global behaviour through the prescription on the uniform distribution of the overstrength factors ($\Omega_i$), which directly affects also the design of the diagonal members. The AISI S213 does not clearly provide a prescription to promote the global mechanism, but the capacity design rules consider acting a force corresponding to the attainment of the expected yield in all diagonals.

In addition, a specific prescription, even if not precisely related to the seismic design, is provided by EN 1993-1-3 (CEN, 2006b) for self-drilling screw connections, which are the main connecting system used in CFS structures. According to this prescription , in order to provide an adequate deformation capacity and to avoid the brittle failure of the fasteners, the following equations should be satisfied:

$$F_{v,Rd} \geq 1.2F_{b,Rd} \text{ or } \Sigma F_{v,Rd} \geq 1.2F_{n,Rd} \tag{7}$$

where $F_{v,Rd}$ is the shear resistance of the screw, $F_{b,Rd}$ is the bearing resistance of the connection, $F_{n,Rd}$ is the net area resistance of the connected member and 1.2 is an overstrength factor.

## CASE STUDY

In order to plan the experimental campaign and to define the configurations of strap-braced walls to be examined, three residential buildings have been designed according to different hypotheses on the design criteria. The studied structures are residential buildings having the same rectangular plan with an area of 220 $m^2$ and constituted by one and three storeys, with storey height of 3.00 m. Three symmetric plan distributions of the seismic resistant systems, which correspond to two, four and eight walls per each direction, have been assumed in order to obtain realistic seismic force acting on the walls (Fig. 1).

A range of dead loads has been considered to account different possible technological and architectural configurations of the constructive elements (flooring, claddings, insulating systems, etc.). Hence, dead loads ranging from 0.60 to 1.50 $kN/m^2$ for floors and from 0.30 to 1.00 $kN/m^2$ for walls have been assumed. Live loads for residential buildings equal to 2.00 $kN/m^2$ have been considered for both floors and roofs. The buildings have been designed considering the environmental loads of two different Italian locations: Rome and Potenza. According to the Italian construction technical code (Ministero delle infrastrutture, 2008), Rome is characterized by medium-low seismic and snow actions, corresponding to a peak ground acceleration equal to 0.11g and a snow load of 0.48 kN/m$^2$, while Potenza is characterized by medium-high seismic and snow actions, corresponding to a peak ground acceleration equal to 0.20g and a snow load of 1.81 kN/m$^2$. The assumed foundation soil is type C (deep deposits of dense or medium dense sand, gravel or stiff clay with thickness from several tens to many hundreds of meters). The main parameters for the calculation of the seismic action at Life Safety limit state are summarized in Tab. 1.

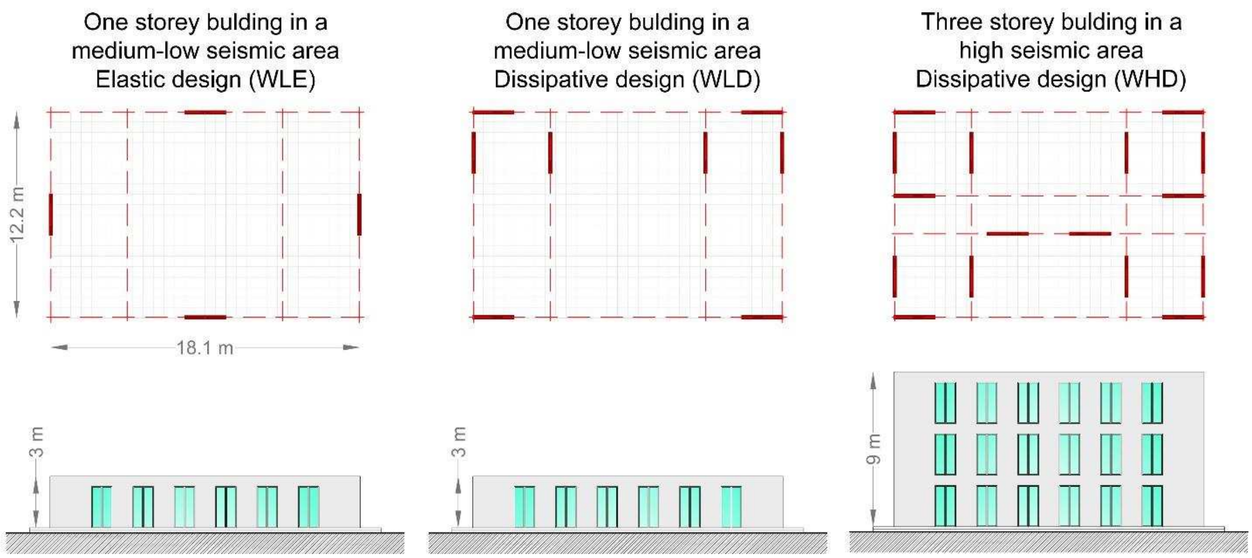



**Figure 1:** Schematic views of case study buildings and assumed plan distribution of walls: (a) one storey building in a medium-low seismic area Elastic design (WLE); (b) one storey building in a medium-low seismic area Dissipative design (WLD); (c) three storey building in a high seismic area Dissipative design (WHD)

**Table 1:** Parameters for the definition of seismic action

| Spectrum parameter | medium-low seismicity | medium-high seismicity |
|---|---|---|
| $a_g$ [g] | 0.110 | 0.202 |
| $F_o$ | 2.628 | 2.446 |
| $T^*_C$ [s] | 0.306 | 0.363 |
| $S_S$ | 1.500 | 1.403 |
| $S_T$ | 1.000 | 1.000 |

The design of the seismic-resistant systems has been carried out through a linear dynamic analysis. In the analysis, the floors are assumed as rigid diaphragms and the effects of accidental eccentricity are neglected. The selected diagonal strap-braced wall configurations have dimension 2.4 m x 2.7 m. For the sake of simplicity, in the case of multi-storey buildings, the wall components have been designed by assuming the forces due to gravity and seismic loads of the ground storey and the same configuration has been assumed for the upper floors. The seismic resistant systems (walls) have been designed by adopting two different approaches: elastic and dissipative. The different design criteria assumed for the three selected wall configurations and the linear dynamic analysis results are summarised in Tab. 2 and 3, respectively.

**Table 2:** Design hypotheses and results for selected wall configurations.

| Property | Wall configuration | | |
|---|---|---|---|
| | WLE | WLD | WHD |
| Location | medium-low seismicity | medium-low seismicity | medium-high seismicity |
| n. of storeys | 1 | 1 | 3 |
| n. of walls per direction | 2 (Plan A) | 4 (Plan B) | 8 (Plan C) |
| Design approach | Elastic | Dissipative | Dissipative |
| Behaviour factor (q) | 1 | 2.5 | 2.5 |
| Dead loads | min | min | max |
| Seismic weight [kN] | 365 | 365 | 2171 |
| Fundamental Period [s] | 0.46 | 0.30 | 0.52 |
| Seismic action on single wall ($H_d$) [kN] | 50.0 | 40.0 | 80.0 |
| Lateral wall resistance ($H_c$) [kN] | 50.5 | 40.8 | 81.6 |
| Lateral wall stiffness (K) [kN/mm] | 3.40 | 4.12 | 6.73 |

WLE: Elastic light wall

WLD: Dissipative light wall

WHD: Dissipative heavy wall

**Table 3:** Design criteria

| Design criteria | Configuration walls | | |
|---|---|---|---|
| | WLE | WLD | WHD |
| R | 1.0 | 2.5 | 2.5 |
| $H_d$ | 50 kN | 40 kN | 80 kN |
| Eq (1) | NO | OK | OK |
| Eq (4) | NO | OK | OK |
| Eq (7) | OK | OK | OK |

The first wall configuration (elastic light wall, WLE) is representative of the one-storey building located in a medium-low seismicity zone and designed according to an elastic approach (R=1). In this case, all wall elements are made of steel S350GD+Z (characteristic yield strength $f_y$=350 MPa, characteristic ultimate strength $f_u$=420 MPa; CEN 2004a) and they are designed without following any prescription aimed at avoiding brittle failure mechanisms, with the only exception of the brittle failure of the fasteners, for which the Eq. (7) has been applied. As a consequence, the collapse mechanism expected in the design phase, is the failure of diagonal net area at the fastener holes location.

The other two wall configurations have been designed according to the dissipative approach (R=2.5) and applying the capacity design rules. These configurations are named dissipative light wall (WLD) and dissipative heavy wall (WHD). The dissipative configurations are referred to buildings with different geometric dimensions and seismic scenarios. In particular, the WLD wall is representative of a one-storey building in a medium-low seismicity level zone, while the WHD corresponds to a three-storeys building in a medium-high seismicity level zone. In the design of dissipative walls, the yielding of the tension diagonal has been considered as the weakest failure mode, without any control on the distribution of the overstrength factors ($\Omega_i$) prescribed by EN 1998-1. For these reason, the connection between the diagonal brace and the gusset plate, with main reference to the net area fracture, has been calculated by satisfying the Eq. (1). This condition implied a specific care in the definition of  connection details and in the steel grade choice for diagonal straps. In particular, in order to obtain a greater net section area, the screws of the diagonal to gusset plate connections are placed in staggered position. In addition, the diagonals are made of S235 steel ($f_y$= 235 MPa, $f_u$ = 360 MPa; CEN 2004b), because it is characterized by a high $f_u/f_y$ ratio (1.53), while all the other elements are made of S350GD+Z steel. The capacity design rules for all the non-dissipative elements (studs, tracks, connections and anchorages) have been applied by considering the Eq. (4). This design procedure corresponds to the prescription given by the AISI S213 in terms of global mechanism control and it is equivalent to adopt the relevant overstrength factor ($\Omega_i$) at each storey. For the connections, Eq. (7) has also been satisfied.

## EXPERIMENTAL CAMPAIGN

### TEST PROGRAM

The lateral response of strap-braced walls has been investigated by testing each of the three selected configurations by two monotonic and two cyclic tests for a total of twelve tests on full-scale wall specimens in size of 2.4 m x 2.7 m. Moreover, taking into account that materials and components affect the wall seismic global response in terms of lateral resistance, stiffness and ductility, the component response has been investigated by means of 17 tests on materials, 8 shear tests on elementary connections between steel profiles and 28 shear tests on connections between gussets and strap braces. The experimental campaign has been carried out at the Department of Structures for Engineering and Architecture of the University of Naples Federico II. In the following, the results of full-scale wall tests are briefly discussed. Information about the component and material tests are provided in Iuorio et al. (2014).

### TEST SPECIMENS

For all wall specimens, the steel framing is made with stud members, having lipped channel sections (C-sections), spaced at 600 mm on the centre and connected at the ends to track members, having unlipped channel sections (U-sections). Chord studs are composed by double C-sections screwed back-to-back. In order to reduce the unbraced length of the chord and interior studs, flat straps are placed at the mid-height of the wall specimens and are connected to blocking members at the ends of walls. The end part of the tracks are reinforced with C-section profiles, by creating box sections. Uplift forces are transferred from the chord studs to the testing frame by hold-down devices made with S700 steel grade ($f_y$= 700 MPa and $f_u$ = 750 MPa), each of which is connected to the studs by four M16 8.8 grade bolts ($f_y$= 640 MPa, $f_u$ = 800 MPa) and to the bottom beam of the testing frame by one M24 8.8 grade bolt. These devices were fabricated by Guerrasio S.r.l. and tested in the framework of another research (Iuorio et al., 2014). Those tests showed that the hold-downs have an experimental characteristic strength greater than 200 kN and a design ultimate strength of 160 kN. The upper and bottom tracks of the walls are connected respectively to the loading (top) and bottom beams of the testing frame by M8 8.8 grade bolts spaced at 300 mm on the centre. The wall specimens are completed with strap braces installed in an X configuration on both sides and connected to the wall framing by gusset plates. All the connections are made with self-drilling screws. For each wall configuration an appropriate fastener was chosen: 6.3 x 40 mm (diameter x length) hexagonal flat washer head self-drilling screws for WLE and WHD specimens, and 4.8 x 16 mm modified truss head self-drilling screws for WLD prototypes. All the steel members are fabricated by S350GD steel grade, except the diagonal straps of dissipative systems, which are made with S235 steel grade. Tab. 4 lists the nominal design dimensions and material properties of the wall components. Schematic drawings of the WHD wall configuration is provided in Fig. 2.

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

| Wall element | WLE Section [mm] | WLD Section [mm] | WHD Section [mm] |
|---|---|---|---|
| Studs[1] | C150x50x20x1.5 | C150x50x20x1.5 | C150x50x20x3.0 |
| Tracks[2] | U153x50x1.5 | U153x50x1.5 | U153x50x1.5 |
| Straps[3] | 90x1.5 | 70x2.0 | 140x2.0 |
| Gusset plates[4] | 270x270x1.5 | 290x290x1.5 | 365x365x1.5 |
| Track reinf.[1] | C150x50x20x1.5 | C150x50x20x1.5 | C150x50x20x3.0 |
| Block.[1] | C150x50x20x1.5 | C150x50x20x1.5 | C150x50x20x3.0 |

[1]C-section: outside-to-outside web depth x outside-to-outside flange size x outside-to-outside lip size x thickness;

[2]U-section: outside-to-outside web depth x outside-to-outside flange size x thickness;

[3]Strap: width x thickness;

[4]Plate: width x height x thickness; All steel elements are S350GD steel grade, except straps for WLD and WHD walls, which are S235 steel grade

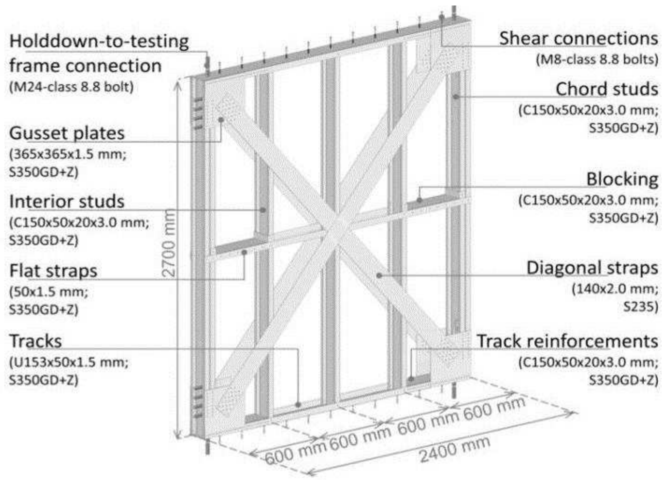



**Figure 2.** WHD wall configuration

## TEST SET-UP AND INSTRUMENTATIONS

Tests on full-scale wall specimens were carried out by using a specifically designed testing frame for in-plane horizontal loading. Horizontal loads were transmitted to the upper wall track by means of a 200x120x10 mm (width x height x thickness) steel beam with rectangular hollow section. The wall prototype was constrained to the laboratory strong floor by the bottom beam of testing frame. The out-of-plane displacements of the wall were avoided by two lateral supports realized with HEB 140 columns and equipped with double roller wheels. The tests have been performed by using a hydraulic actuator having 500 mm stroke displacement and 500 kN load capacity. A sliding-hinge has been placed between the actuator and the tested wall in order to avoid the transmission of any vertical load on the specimen.

Eight LVDTs have been used to measure the specimens displacements, as shown in Fig. 3. In particular, three LVDTs ($W_1$, $W_2$ e $W_3$) have been installed to record the horizontal displacements and two LVDTs ($W_4$, $W_5$) for the vertical displacements. The local deformations of the diagonal straps have been recorded by means of two strain-gauges ($S_i$). A load cell has been used to measure the horizontal loads.

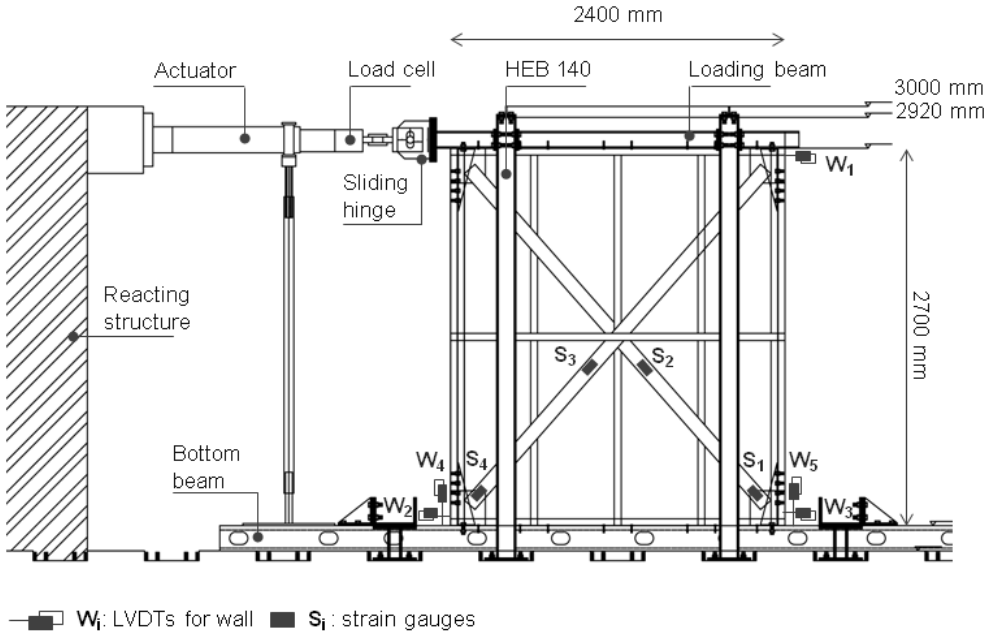



**Figure 3.** Test set-up and instrumentation

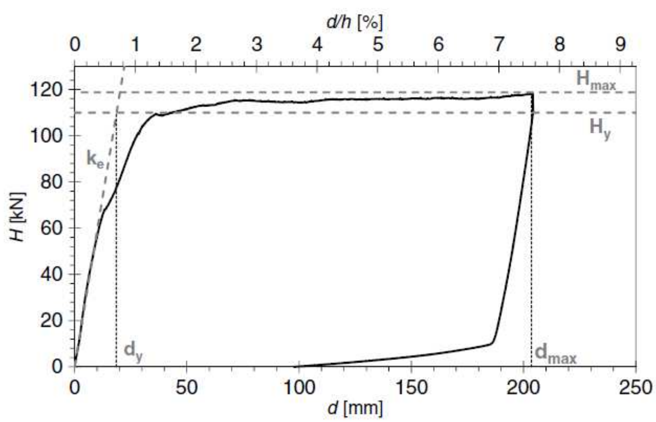



**Figure 4.** Test result assumed parameters

**LOADING PROTOCOLS**

In the monotonic loading regime, the tests have been performed by applying a loading protocol organized in two phases. In the first phase the wall specimens have been pulled and in the second phase they have been pushed. Both phases have been followed by the unloading of the wall prototypes in order to lead them back to the initial position. This testing protocol involved displacements at a rate of 0.10 mm/s.

The cyclic tests have been carried out by adopting a loading protocol known as "CUREE ordinary ground motions reversed cyclic load protocol" developed for wood walls by Krawinkler et al. (2001). The cyclic loading test protocol consists of a series of stepwise increasing deformation cycles with amplitudes equal to multiple of a reference displacement ($\Delta$), defined as $\Delta$ = 2.667$d_y$ according to Velchev et al. (2010), where $d_y$ is the yield displacement evaluated in the monotonic tests on identical wall specimens. The displacements was imposed with a rate of 0.5 mm/s, for displacements up to 9.97 mm, 7.36 mm e 7.27 mm for WLE, WLD and WHD walls respectively, and of 2.0 mm/s for displacement greater than those mentioned above. Therefore, these tests are quasi-static considering the assumed loading rates and are different from the Canadian tests by Velchev et al. 2010, which were performed with imposed frequency and high loading rates (about 100 mm/s).

**MONOTONIC TEST RESULTS**

Test results are expressed in terms of yield strength ($H_y$), maximum strength ($H_{max}$), yield displacement ($d_y$), maximum displacement ($d_{max}$), and conventional elastic stiffness ($k_e$), where this last is the secant stiffness at 40% of the maximum strength which are defined in Fig. 4. The test results together with the observed failure mechanisms are shown in Tab. 5. Moreover, the theoretical predicted values of the strength ($H_{y,p}$ and $H_{max,p}$) and stiffness ($k_{e,p}$), which are evaluated according to the EN 1993-1-3 (CEN, 2006b) through the methodology illustrated in Appendix 1 and by using the experimental measured material properties (S350GD+Z: $f_y$=355 MPa, $f_u$ =409 MPa; S235: $f_y$=302 MPa, $f_u$ =366 MPa), are provided. The experimental acting loads (H) vs top wall displacements (d) curves for the monotonic tests are provided in Fig 5. Also the interstorey-drift levels, defined as the ratio (d/h) between the top wall displacement and wall height set equal to 2700 mm are shown.

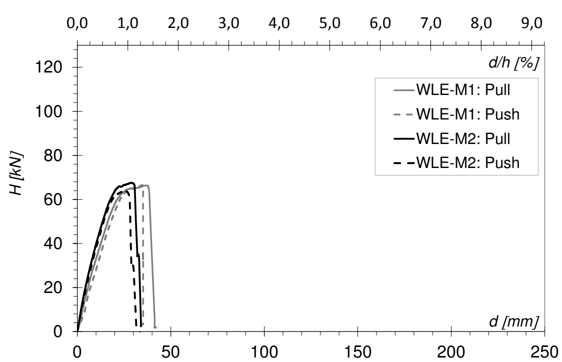
Test results reveal a reduction of maximum strength contained within 12% in the pushing phase with respect to the pulling phase. In general, the conventional elastic stiffness values are significant different between the two phases, with reductions up in the range from 7% to 72% because, in the first phase (pulling), some wall components deteriorated influencing the response of the second phase (push). For the WLE configurations the collapse was governed by the net section failure of diagonal straps (Fig. 6a), while WLD and WHD specimens showed the brace yielding without reaching the rupture, in accordance with the maximum stroke of the actuator (Fig. 6b). The results highlight variations up to 9% between the experimental and theoretical strengths, while the ratios between the average experimental and theoretical stiffness demonstrate that the experimental values are lower than the theoretical predictions, with variations ranging between 8% and 47%.

Concerning the reached interstorey-drift levels, the dissipative walls were displaced to significantly higher drift levels (in the range of 5.1% through 9.0% for WLD walls and in the range of 5.8% through 8.1% for WHD specimens) compared with elastic walls (in the range of 1.0% through 1.4%).

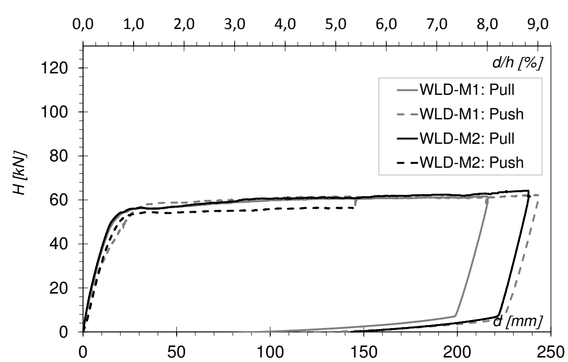
**Table 5.** Test results of monotonic tests

| Specimen | $H_y$ [kN] | | $H_{max}$ [kN] | | $d_y$ [mm] | | $d_{max}$ [mm] | | $k_e$ [kN/mm] | | Failure mode | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | pull | push | pull | push | pull | push | pull | push | pull | push | pull | push |
| WLE-M1 | 64.9 | 65.6 | 66.3 | 66.6 | 18.5 | 24.3 | 36.7 | 35.3 | 3.5 | 2.7 | NSF | NSF |
| WLE-M2 | 65.9 | 63.7 | 67.6 | 64.3 | 15.0 | 15.5 | 30.2 | 27.1 | 4.4 | 4.1 | NSF | NSF |
| th | - | - | 61.4 | 61.4 | - | - | - | - | 4.4 | 4.4 | NSF | NSF |
| $exp_{,AV}$ / th | - | - | 1.09 | 1.07 | - | - | - | - | 0.90 | 0.77 | - | - |
| WLD-M1 | 56.7 | 58.8 | 61.7 | 62.3 | 14.2 | 18.4 | 214.5 | 244.2 | 4.0 | 3.2 | BY | BY |
| WLD-M2 | 56.0 | 54.4 | 64.2 | 56.5 | 13.0 | 17.0 | 237.9 | 139.0 | 4.3 | 3.2 | BY | BY |
| th | 55.0 | 55.0 | - | - | - | - | - | - | 4.9 | 4.9 | BY | BY |
| $exp_{,AV}$ / th | 1.02 | 1.03 | - | - | - | - | - | - | 0.85 | 0.65 | - | - |
| WHD-M1 | 110.3 | 107.8 | 116.9 | 119.3 | 17.8 | 29.9 | 157.6 | 159.7 | 6.2 | 3.6 | BY | BY |
| WHD-M2 | 109.5 | 114.2 | 118.4 | 119.3 | 18.6 | 33.6 | 203.5 | 220.0 | 5.9 | 3.4 | BY | BY |
| th | 110.0 | 110.0 | - | - | - | - | - | - | 6.6 | 6.6 | BY | BY |
| $exp_{,AV}$ / th | 1.00 | 1.01 | - | - | - | - | - | - | 0.92 | 0.53 | - | - |

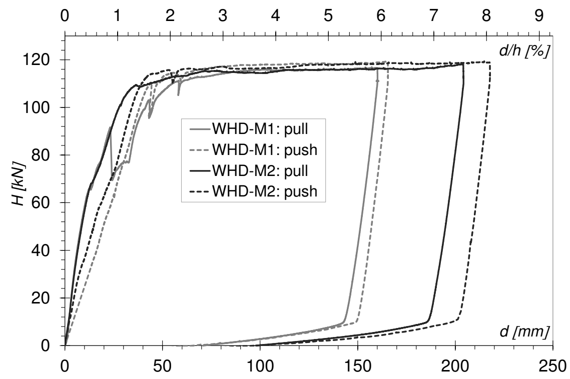
$exp_{,AV}$ : average experimental values; th: theoretical values; BY: brace yielding; NSF: net section failure of strap-bracing




WLE-M

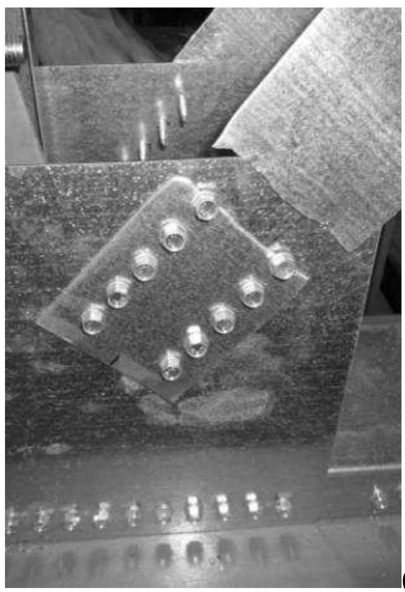



WLD-M



WHD-M

**Figure 5.** Experimental curves of monotonic tests


(a)

(b)

**Figure 6.** (a) Net section failure for WLE-M1 and (b) brace yielding for WLD-M1

## CYCLIC TEST RESULTS

Fig. 7 provides the acting loads (H) versus the measured displacements (d) curve for all the cyclic tests. The results of the cyclic tests are shown in Tab. 6, in which the theoretical values of the wall strength and stiffness are the same provided for the monotonic tests. The results show that the strength and stiffness recorded for the two loading directions have maximum differences of 4% and 18%, respectively, except a variation of 35% for the stiffness of WHD-C1 specimen. For WLE prototypes the observed collapse mode has been the net section failure of diagonal straps, whereas the WLD and WHD wall specimens showed the brace yielding (Fig. 8). In particular, in some configurations the net section failure occured for high level of drift (>2.5%). The ratios between the average experimental and theoretical values highlighted that the experimental strengths are higher than the theoretical predictions with maximum difference of 14%, while the measured stiffness values are lower than the predicted parameters with a variation up to 14%. As in the case of monotonic tests, the dissipative walls exhibited significantly higher drift levels (in the range of 5.2% through 6.5% for WLD walls and in the range of 2.5% through 8.2% for WHD specimens) compared with elastic walls (in the range of 1.0% through 1.4%).

The comparison between the monotonic and cyclic test results reveals that the average experimental shear strength values registered under monotonic loads are lower than the one recorded in the cyclic tests with maximum variations of about 10%. The maximum difference in terms of stiffness between the results of monotonic pulling phase and cyclic results is about 30%.

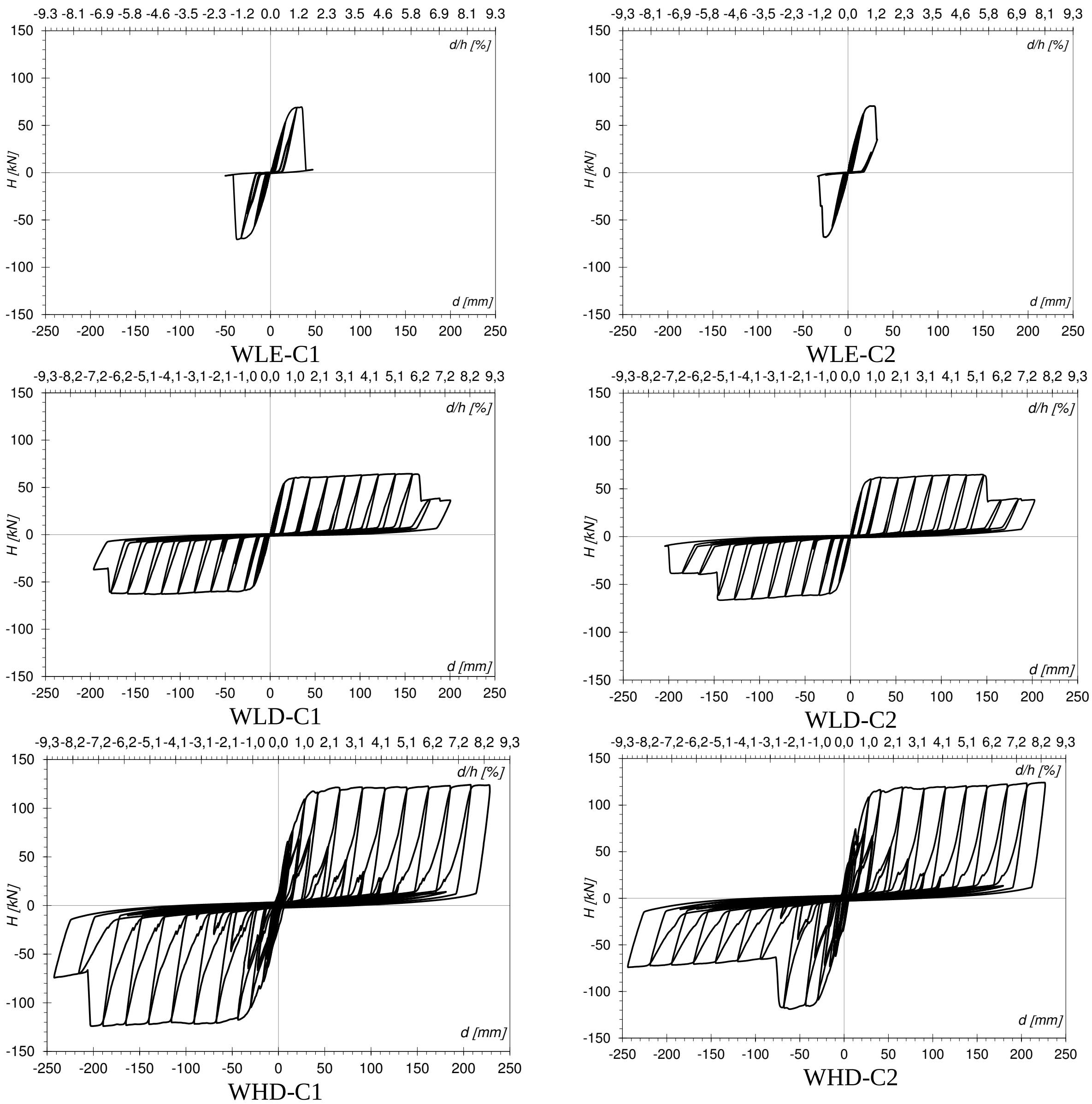
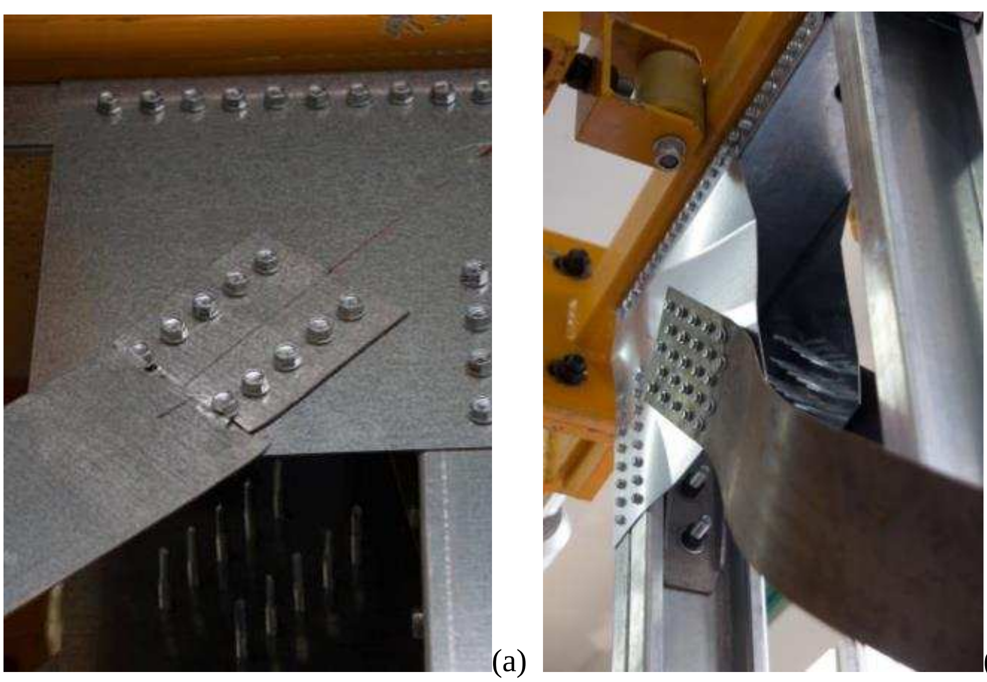



**Figure 7.** Experimental curves of reversed cyclic tests

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

| Specimen | $H_y$ [kN] | | $H_{max}$ [kN] | | $d_{max}$ [mm] | | $k_e$ [kN/mm] | | Failure mode | |
|---|---|---|---|---|---|---|---|---|---|---|
| | pull | push | pull | push | pull | push | pull | push | pull | Push |
| WLE-C1 | 69.6 | 68.9 | 70.6 | 69.4 | 38.1 | 35.7 | 3.7 | 3.4 | NSF | NSF |
| WLE-C2 | 68.0 | 69.9 | 68.3 | 70.5 | 26.5 | 31.3 | 4.0 | 4.7 | NSF | NSF |
| th | - | - | 61.4 | 61.4 | - | - | 4.4 | 4.4 | NSF | NSF |
| $exp_{,AV}$ / th | - | - | 1.13 | 1.14 | - | - | 0.88 | 0.92 | - | - |
| WLD-C1 | 58.7 | 59.8 | 63.1 | 64.4 | 176.2 | 165.5 | 3.8 | 4.0 | BY* | BY* |
| WLD-C2 | 58.7 | 60.0 | 66.6 | 64.9 | 141.2 | 144.8 | 4.6 | 4.5 | BY* | BY* |
| th | 55.0 | 55.0 | - | - | - | - | 4.9 | 4.9 | BY | BY |
| $exp_{,AV}$ / th | 1.07 | 1.09 | - | - | - | - | 0.86 | 0.87 | - | - |
| WHD-C1 | 116.7 | 116.0 | 124.0 | 124.2 | 197.0 | 221.0 | 5.7 | 7.7 | BY* | BY |
| WHD-C2 | 112.9 | 111.6 | 118.9 | 124.2 | 67.5 | 221.8 | 7.5 | 6.7 | BY* | BY |
| th | 110.0 | 110.0 | - | - | - | - | 6.6 | 6.6 | BY | BY |
| $exp_{,AV}$ / th | 1.04 | 1.03 | - | - | - | - | 1.00 | 1.09 | - | - |

$exp_{,AV}$ : average experimental values; th: theoretical values; BY: brace yielding; NSF: net section failure of strap-bracing; *net section failure occurred after the brace yielding for high levels of drift (>2.5%)



(a) (b)

**Figure 8.** (a) Net section failure for WLE-C2 and (b) brace yielding for WHD-C2

## VERIFICATION OF THE ASSUMED DESIGN CRITERIA

In order to validate the design criteria for CFS diagonal strap-braced walls in seismic area, the prescriptions and requirements of Eurocodes and AISI S213 have been also evaluated on the basis of the experimental data.

For each selected wall configuration, a preliminary evaluation at component level (single seismic resistant element) of the R factor based on the results of monotonic and cyclic wall tests has been carried out. This approach cannot consider the effects of the whole system overstrength. The R factor has been defined by the ductility-related ($R_d$) and overstrength-related ($R_o$) modification factors, as given in Uang (1991):

$$R = R_d \cdot R_o \tag{8}$$

Considering that the fundamental periods for this structural typology is generally ranging between 0.1 and 0.5 s, the $R_d$ factor can be evaluated as follows:

$$R_d = \sqrt{2\mu - 1} \quad \text{with} \quad \mu = \frac{d_{max}}{d_y} \tag{9}$$

where $\mu$ is the ductility; $d_{max}$ e $d_y$ are the maximum and the conventional elastic limit of the top wall displacement, respectively. The displacement $d_{max}$ has been defined as the displacement corresponding to the following limits of interstorey-drift (d/h, with h=2700 mm as wall height): 1.5%, 2% and 2.5%. For the cases in which the wall collapse occurred for displacement lower than the given limits, $d_{max}$ has been assumed as the displacement at the peak load. The limits of 1.5% and 2% are conservatively assumed equal to those provided by FEMA 356 (2000) for hot-rolled concentrically braced structures at the Life Safety and Collapse Prevention limit states, respectively, whereas the limit of 2.5% represents the minimum value of drift observed in dissipative tests (WLD and WHD) without any type of rupture.

The $R_o$ factor can be evaluated through the formulation provided by Mitchell et al. (2003):

$$R_o = R_{sd} \cdot R_{\phi} \cdot R_{yield} \cdot R_{sh} \tag{10}$$

where $R_{sd}$= $H_c/H_d$, with $H_c$ and $H_d$ design wall resistance and seismic demand, respectively; $R_{\phi}$= $H_{yn}/H_c$, with $H_{yn}$ nominal yielding resistance; $R_{yield}$= $H_y/H_{yn}$, with $H_y$ experimental yielding resistance (average); $R_{sh}$= $H_{\%}/H_y$, with $H_{\%}$ wall resistance at relevant inter-story drift.

Tab. 7 and 8 show the values of the R factor obtained by the experimental results. In particular, for WLE walls $d_{max}/h$ is always less than 1.5%, so the evaluation of R is limited to the case d= $d_{max}$. In the case of WLE walls (Tab. 7), it can be noted that the R factor values proposed by AISI S213 for Conventional construction category (R=1.6) is always smaller than those experimentally obtained (R=2.0-2.2). In particular, the obtained values of overstrength factor are very uniform ($R_o$=1.2) and slightly lower than the one provided by code ($R_o$=1.3). On the contrary, the measured ductility factors ($R_d$=1.7-1.8) are always greater than the provided value ($R_d$=1.2). As far as WLD and WHD walls are concerned, the value provided by AISI S213 in case of Limited ductility braced walls (R=2.5) represents a lower limit of the obtained R factors (R=2.5-3.7 for 1.5%, R=3.1-4.3 for 2%, R=3.6-4.9 for 2.5%) (Tab. 8). In this case, it can be noticed that the obtained values of both overstrength ($R_o$=1.4÷1.5) and ductility factor ($R_d$=1.9-4.9) are greater than AISI S213 values ($R_o$=1.3 and $R_d$=1.9), with the only exception of WHD-M2 case ($R_d$=1.9) for 1.5% drift limit. As it is well known, the methodology used to evaluate the behaviour factor does not explicitly take into account the load-deformation hysteresis "shape", which for the examined structural typology is characterized by a relevant pinching. Therefore, the obtained results in terms of q-values should be estimated using more advanced methods, such as non-linear time history dynamic analysis.

**Table 7.** R factor for WLE

| Specimen | $R_d$ | $R_o$ | R |
|---|---|---|---|
| WLE-M1 | 1.74 | 1.15 | 2.00 |
| WLE-M2 | 1.74 | 1.17 | 2.04 |
| WLE-C1 | 1.80 | 1.21 | 2.19 |
| WLE-C2 | 1.73 | 1.20 | 2.08 |

**Table 8.** R factor for WLD and WHD

| Specimen | 1.5% interstorey drift | | | 2% interstorey drift | | | 2.5% interstorey drift | | |
|---|---|---|---|---|---|---|---|---|---|
| | $R_d$ | $R_o$ | R | $R_d$ | $R_o$ | R | $R_d$ | $R_o$ | R |
| WLD-M1 | 2.2 | 1.4 | 3.1 | 2.6 | 1.4 | 3.7 | 2.9 | 1.5 | 4.3 |
| WLD-M2 | 2.3 | 1.4 | 3.2 | 2.7 | 1.4 | 3.9 | 3.1 | 1.5 | 4.5 |
| WLD-C1 | 2.2 | 1.5 | 3.3 | 2.6 | 1.5 | 3.9 | 2.9 | 1.5 | 4.5 |
| WLD-C2 | 2.4 | 1.5 | 3.7 | 2.9 | 1.5 | 4.3 | 3.2 | 1.5 | 4.9 |
| WHD-M1 | 1.9 | 1.4 | 2.6 | 2.3 | 1.4 | 3.1 | 2.6 | 1.5 | 3.7 |
| WHD-M2 | 1.9 | 1.4 | 2.5 | 2.2 | 1.4 | 3.1 | 2.5 | 1.5 | 3.6 |
| WHD-C1 | 2.0 | 1.5 | 2.9 | 2.3 | 1.5 | 3.4 | 2.7 | 1.5 | 4.0 |
| WHD-C2 | 2.1 | 1.4 | 2.9 | 2.5 | 1.4 | 3.5 | 2.8 | 1.5 | 4.1 |

It has to be noticed that the experimental evidence showed that the design formulation (Eq. 2), aimed at preventing the failure of the diagonal net area at fastener holes, is not always effective. In fact, even if the diagonal connections of dissipative configurations (WLD and WHD) were designed according to Eq. 2, only in the case of monotonic wall tests the yielding of the tension diagonals has been reached without ruptures in the field of the investigated displacements (drift higher than 5.1%), while the failure mechanism observed in all cyclic wall tests always corresponds to the net area fracture (drift higher than 2.5%). This difference would be caused by low cycle fatigue phenomena amplified by the stress concentrations at the fastener holes. However, the obtained drift levels are always larger than those typically occurring in real structures during a design level earthquake and they are greater than the drift limits of 1.5% and 2% provided in FEMA 356 (2000) for hot-rolled concentrically braced structures.

As far as the capacity design criteria are concerned, the experimental results showed that the adopted formulation (Eq. 4) is able to preserve the seismic-resistant system from undesirable brittle failures of connections, tracks, studs and anchorages. Similar considerations can be also made for the formulation used to provide an adequate deformation capacity to the connections (Eq. 8). In fact, no shear failure of the screws occurred in all performed wall tests. The experimental results do not allow to make any consideration about the global mechanism because the tests performed on walls are representative of only one story buildings.

## CONCLUSIONS AND FUTURE DEVELOPMENTS

This paper presents a critical analysis of the seismic design criteria for CFS diagonal strap-braced stud walls. In particular, on the basis of prescriptions given by the AISI S213 for CFS structures and those provided by Eurocodes for hot-rolled tension-only X-braced steel frames, a seismic design method for CFS strap-braced structures is proposed to be used for future European seismic codes. The experimental results allowed the validation of the assumed design hypotheses. The force modification factor values provided by AISI S213 are widely confirmed by the experimental tests and, the code values represent lower limits of the one obtained experimentally. In addition, the requirements concerning the capacity design given in the Eurocodes, for hot-rolled tension-only X-braced steel frames, are also reliable, with some modifications for the CFS diagonal strap-braced stud walls. As a further development, an extended numerical study including non-linear dynamic analysis should be performed for a more accurate estimation of the R factor. In addition, shaking table tests on 3D structures and tests on prototypes representative of multi-storey building could be carried out in order to obtain a complete overview of the seismic performance of the investigated structural typology.

## ACKNOWLEDGMENTS

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## APPENDIX 1

Wall resistance prediction:

Wall resistance: $H_c = \min\left(H_{c,d}; H_{c,c}; H_{c,s}; H_{c,t}; H_{c,a}\right)$ (11)

Diagonal contribution: $H_{c,d} = n_d \cdot \min(N_{pl,Rd}; N_{u,Rd}) \cos\alpha$ (12)

Connection contribution: $H_{c,c} = n_d \cdot n_s \cdot \min\left(F_{b,Rd}; F_{v,Rd}\right) \cos\alpha$ (13)

End stud contribution: $H_{c,s} = \dfrac{\left(N_{s,Rd} - N_{Ed,G}\right)}{h} \cdot L$ (14)

Track contribution: $H_{c,t} = N_{t,Rd}$ (15)

Anchor contribution: $H_{c,a} = \min\left(\dfrac{\left(N_{a,Rd} + N_{Ed,G}\right)}{h} \cdot L; n_a \cdot V_{a,Rd}\right)$ (16)

Wall stiffness prediction:

Wall stiffness: $K = \dfrac{1}{\dfrac{1}{K_d} + \dfrac{1}{K_c} + \dfrac{1}{K_a}}$ (17)

Diagonal contribution: $K_d = \dfrac{n_d \cdot E \cdot A_d \cdot \cos^2 \alpha}{L}$ (18)

Connection contribution: $K_c = \dfrac{n_d \cdot n_s \cdot k_s \cdot \cos^2 \alpha_d}{2}$ (19)

Anchor contribution: $K_a = \dfrac{k_a \cdot L^2}{h^2}$ (20)

## NOTATION LIST

The following symbols are used in this paper:

$A$: gross cross-section area

$A_d$: diagonal cross section area

$A_{net}$: net area of the cross-section at the fasteners holes

$E$: Young's modulus

$F_{b,Rd}$: design bearing resistance per one screw

$F_{C,Rd}$ is the connection resistance

$F_o$: spectrum amplification factor

$F_u$: nominal ultimate tensile strength

$F_{v,Rd}$ is shear resistance of one screw

$F_y$: nominal yield strength

$H_d$: wall design lateral strength

$L$: wall length

$N_{a,Rd}$: design resistance of a single tension anchor

$N_{Ed,G}$ and $N_{Ed,E}$: design axial forces in non-dissipative members due to non-seismic and seismic loads, respectively

$N_{Ed,G}$: acting axial force due to the gravity loads

$N_{pl,Rd}$ : design plastic resistance of the diagonal

$N_{pl,Rd}(M_{Ed})$: design buckling resistance of the beam or column considering the interaction with the bending moment ($M_{Ed}$), that is generally small for the examined systems having low aspect ratios

$N_{s,Rd}$ : design buckling resistance of the chord stud

$N_{t,Rd}$: design buckling resistance of the track

$N_{u,Rd}$ : design resistance of the net cross section at fasteners holes

$R_y$ and $R_t$: coefficients for expected yield and tensile strength

$S_s$: stratigraphic amplification factor

$S_T$: topographic amplification factor

$T^*_c$: starting period of the constant speed branch of the horizontal spectrum

$V_{a,Rd}$: design resistance of each shear anchor

$a_g$: peak ground acceleration

$f_u$ is the characteristic ultimate strength

$f_y$ is the characteristic yield strength

$h$: wall height

$k_a$: axial stiffness of the anchorage system in tension

$k_s$: shear stiffness of a single screw connection

$n_a$: number of shear anchors

$n_d$: number of diagonals (1 for diagonals on one wall side only and 2 for diagonals on both sides)

$n_s$: number of screws in one diagonal-to-frame connection

$\Omega$ is the minimum value of the overstrength factor $\Omega_i = N_{pl,Rd,i} / N_{Ed,i}$ with $N_{pl,Rd,i}$ and $N_{Ed,i}$ the design plastic resistance and seismic axial force in $i^{th}$ diagonal, respectively

$\alpha$: angle of the diagonal with respect to the horizontal

$\gamma_{M0}$= 1.00 partial safety factor for yielding resistance of gross cross-section

$\gamma_{M2}$= 1.25 partial safety factor for the tensile resistance of net sections

$\gamma_{ov}$ : material overstrength factor (recommended equal to 1.25)

$\bar{\lambda}$ : normalized global slenderness

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