SEISMIC BEHAVIOUR OF "ALL-STEEL" CFS STRUCTURES:
Design criteria

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INTRODUCTION

The search for innovative building methods to ensure high structural, technological and environmental performance is promoting the development of light gauge steel structural systems. Among them, stick-built constructions realized with Cold-Formed Steel (CFS) profiles are attracting considerable interest in the construction sector and in the recent research studies. Nevertheless, the main European structural code for seismic design, the Eurocode 8 part 1 (EN 1998-1) [1], does not provide any prescription for the seismic design of CFS structures. Presently, the "North American Standard for Cold Formed Steel Framing - Lateral Design" AISI S213-07 [2] represents the only reference for the design of this structural typology under seismic actions. This document is developed by the American Iron and Steel Institute Committee on Framing Standards and it codifies the design of different seismic resistant CFS systems for Canada, Mexico and United States. As an effort to define the seismic design criteria for such structures, an extended theoretical and experimental study aimed to investigate the seismic behaviour of strap-braced stud shear walls has been carried out within RELUIS –DPC 2010-2013 research project. The research included a wide experimental campaign as well as theoretical analyses to define criteria for the seismic design of strap braced CFS structures. Among the different steel seismic-resistant systems regulated by the EN 1998-1, traditional concentrically braced frame (X diagonal) represents the closest system to the investigated one. In this paper a critical analysis of the current standards is illustrated, with particular reference to the analysis and comparison of the existing provisions for the two similar structural typologies (traditional concentrically braced frames and strap braced CFS system). Based on the results of that critical analysis, the design hypotheses for the definition of a case study have been defined. In addition, on the basis of the adopted design assumptions and the experimental results [3], guidelines for the seismic design of strap braced CFS structures are proposed.

1 CFS VS TRADITIONAL BRACED SYSTEM IN CURRENT SEISMIC CODES

1.1 Basis of the comparison

The applicability of a structural system in a seismic area is related to the clarity and the interpretation of technical prescriptions. In order to identify the peculiarities of the seismic design of the investigated system, the prescriptions provided by the AISI S213 for Canada have been examined. The AISI prescriptions have been compared with those provided by EN 1998-1 for traditional X-braced frames. In the following sections, the comparison of the design prescriptions provided by the two examined codes is illustrated.

1.2 Behaviour factor and height limits

To develop seismic design rules to be adopted in a seismic code, the behaviour factor is fundamental issue to analyze. In the case of regular buildings in elevation, the behaviour factor provided by EN 1998-1 for traditional X-braced systems is equal to 4. On the other hand, the AISI S213 defines the behaviour factor for CFS buildings, namely force modification factor, 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. For the first one, called "Limited ductility braced wall", the capacity based design approach is applied by assuming that the braces act as the energy-dissipating element (gross cross-section yielding). For the latter one, called "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 a behaviour factor equal to 2.5 (R_o= 1.3 e R_d = 1.9) while, for "Conventional construction" category, the behaviour factor is equal to 1.6 (R_o= 1.3 e R_d = 1.2). In addition, the code provides building height limitations, depending on seismic intensity, for both building categories. In particular, in the case of "Limited ductility braced wall", this limit is equal to 20 m for any type seismic intensity, while "Conventional construction" is allowed only for medium-low seismic load and the building height should not exceeding 15 m.

1.3 Slenderness limits and diagonals design

In the case of traditional X-braced frames, EN 1998-1 prescribes that the seismic force has to be absorbed only by the tension diagonal. In order to reduce local buckling phenomena, the cross-sectional class for the seismic resistant elements should be 1 or 2. In addition, in the case of building having more than two storeys, the normalized slenderness of the diagonal members has to be limited in a given range ($1.3 \leq \bar{\lambda} \leq 2$). The AISI S213 does not provide prescriptions concerning the cross-sectional class and the limits for diagonal slenderness, because studs (columns) and tracks (beams) of the considered system are generally made of slender CFS profiles (class 4), while diagonals are straps which do not resist to any compression loads.
As far as the design rules for diagonal members are concerned, in order to ensure a ductile behaviour, the EN 1998-1 requires that, according to EN 1993-1-1 [4], the design yielding strength of the diagonal cross section has to be less than the ultimate design strength of the net cross section at fasteners holes. This condition can be expressed as follow:

$$\frac{A_{net}}{A} \geq 1.1 \cdot \frac{\gamma_{M2}}{\gamma_{M0}} \cdot \frac{f_{yk}}{f_{tk}} = \alpha \cdot \frac{f_{yk}}{f_{tk}} \text{ with } \alpha = 1.1 \cdot \frac{\gamma_{M2}}{\gamma_{M0}} = 1.38 \qquad (1)$$

where A_{net} is the net area of the cross-section at the fasteners holes; A is the gross cross-section area; γ_{M2} = 1.25 is the partial safety factor for the tensile resistance of net sections; γ_{M0}= 1.00 is the partial safety factor for yielding resistance of gross cross-section; f_{yk} is the nominal yield strength; f_{tk} is the nominal ultimate strength.
On the other hand, AISI S213 provides a verification for dissipative elements in tension similar to the Eq. (1), in which the expected yield strength has to be lower than the expected tensile strength of the diagonal strap bracing member:

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

where F_y is nominal yield strength; F_u is nominal ultimate tensile strength; R_y e R_t are the coefficients for expected yield and tensile strength, respectively. These coefficients are provided by the standard as function of the yield strength (F_y). Table 1 shows the values of β coefficient, obtained by Eq.(2), for the different steel grade. The results show that the coefficient α=1.38 represents an upper limit and it is conservative with regard to the coefficient β values, which ranges from 1.00 to 1.27.

Table 1: β values for steel grades provided by AISI S213

Steel grade (f_y in MPa)	β
33 ksi (230)	1.25
37 ksi (255)	1.27
40 ksi (275)	1.18
50 ksi (340)	1.00

1.4 Global mechanism and capacity design rules

In general, for both CFS and traditional X-bracing system, the most ductile failure mechanism consists of the yielding of the tension diagonal, which can be ensured by providing an adequate overstrength to non-dissipative elements, i.e. connections, beams and columns.

In particular, according to the EN 1998-1, the connections have to be designed by considering the following force:

$$1.1\cdot\gamma_{ov}\cdot R_{fy}=1.1\cdot\gamma_{ov}\cdot\frac{A\cdot f_{yk}}{\gamma_{M0}}=\delta\cdot A\cdot f_{yk}\ \text{ with }\ \delta=1.1\cdot\frac{\gamma_{ov}}{\gamma_{M0}} \tag{3}$$

where γ_{ov}=1.25 is the material overstrength factor; $R_{fy}=A\cdot f_{yk}/\gamma_{M0}$is the design plastic resistance of the connected dissipative member.

In addition, in order to provide an adequate deformation capacity and to avoid the brittle failure of the fasteners, the EN 1993-1-3 [5] provides the following equations for screwed connections:

$$F_{v,Rd}\geq 1.2F_{b,Rd}\ \text{ or }\ \Sigma F_{v,Rd}\geq 1.2F_{n,Rd} \tag{4}$$

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

In the case of beams and columns, subjected mainly to axial forces, the following condition should be satisfied:

$$N_{pl,Rd}\left(M_{Ed}\right)\geq N_{Ed,G}+1.1\cdot\gamma_{ov}\cdot\Omega\cdot N_{Ed,E} \tag{5}$$

where $N_{Ed,G}$ and $N_{Ed,E}$ are the design axial forces due to non-seismic and seismic loads, respectively; Ω is the minimum value of the overstrength factor evaluated for each diagonal defined as Ω_i = $N_{pl,Rd,i}$ / $N_{Ed,i}$; and $N_{pl,Rd}(M_{Ed})$ is the design plastic resistance evaluated by considering the interaction with the bending moment (M_{Ed}), that is generally null for the examined systems. Therefore, considering the ith diagonal and the relevant Ω_i, the fulfilment of Eq. (5) consists in designing the non-dissipative elements (studs, tracks and anchorages) for a force equal to the plastic resistance of the tension diagonal in the same way of Eq. (3) for design of connections.

In order to obtain a uniform dissipative behaviour and to promote a global mechanism, in the case of buildings with more than two floors, EN 1998-1 requires that the maximum overstrength factor (Ω_i) does not differ from the minimum by more than 25%.

In order to ensure an adequate overstrength of the non-dissipative elements, the AISI S213 requires that these elements have to resist the force corresponding to the expected yield strength of the diagonal, evaluated by the following equation:

$$R_y\cdot A\cdot F_y \tag{6}$$

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 addition, no specific prescriptions for the connections design are provided.

It has to be noticed that the meaning coefficient δ of Eq. (3) is the same of the coefficient R_y in Eq. (6). The values of these two coefficients are compared in Table 2.

Table 2: Comparison between δ and Ry

Yield strength (f_y in MPa)	Steel grade AISI S213 (f_y in MPa)	R_y	Steel grade EN 1998-1 (f_y in MPa)	γ_{ov}	δ
230 ÷ 235	33 ksi (230)	1.5	S235 (235)	1.25	1.38
250 ÷ 255	37 ksi (255)	1.4	-		
275 ÷ 280	40 ksi (275)	1.3	S275 (275)		
340 ÷ 355	50 ksi (340)	1.1	S355 (355)		

In particular, the coefficient δ is constant and equal to 1.38, while R_y depends on yield strength of steel (f_y). The comparison of the two coefficients shows that the coefficient R_y decreases with the

increasing of the yield strength and it is higher (conservative) than δ for low values of yield strength (230÷255MPa).
In the comparison of the design prescriptions, it can be noticed that both codes are inclined to promote a global failure mechanism. The EN 1998-1, attempts to obtain a global behaviour by the prescription on the distribution of the overstrength factors (Ω), which directly affects also the design of the diagonal members. The AISIS213 does not clearly provide a prescription for promoting the global mechanism, but the capacity design rules consider that, at each storey, the diagonals are simultaneously yielded.

2 CASE STUDY

In order to plan the experimental campaign and to define the configurations of diagonal 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, two and three storeys. These buildings have been designed considering the environmental loads of two different Italian locations: Rome and Potenza, that are characterized by a peak ground acceleration equal to 0.11g (medium-low seismicity) and 0.20g (medium-high seismicity), respectively. In particular, the seismic action has been defined by assuming the design spectra provided by the Italian code [6]. In Table 3 the main parameters for the calculation of the seismic load for the Life Safety limit state are summarized. The design of the seismic-resistant systems has been carried out through a linear dynamic analysis. The selected diagonal strap braced wall configurations have dimension 2400 mm x 2700 mm. They have been designed by adopting two different approaches: elastic and dissipative. The three configurations obtained according to the adopted different hypotheses are illustrated in Fig. 1.

Table 3: Parameters for the definition of seismic action

	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

a_g= peak ground acceleration; F_o= the spectrum amplification factor; T^*_c= starting period of the constant speed branch of the horizontal spectrum; S_s= stratigraphic amplification factor; S_T= topographic amplification factor.

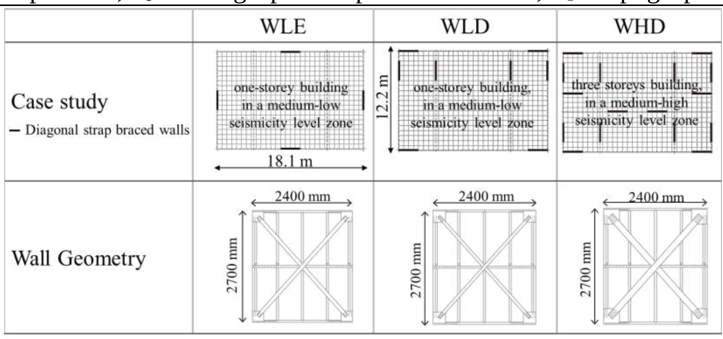

Fig. 1: Diagonal strap braced wall configurations

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 (q=1). In this case, all wall elements are made of steel S350GD+Z 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. (4) 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 (q=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 case of the three-story building, the strap braced walls dimensions and materials are the same on each storey. In the design of dissipative walls, the yielding of the tension diagonal has been considered as the weakest failure modes, without any control on the distribution of the overstrength factors (Ω). For these reason, the connection between the diagonal brace and the gusset plate, with particular reference to the net area fracture, has been calculated by satisfying the Eq. (1). This condition implied a particular care in the definition of the connection details and in the choice of the steel grade for diagonal straps. In fact, the diagonals are made of S235 steel, while all the other elements are made of S350GD+Z steel. In addition, in order to fulfil the capacity design rules, all the non-dissipative elements (studs, tracks, connections and anchorages) have been designed through the Eq. (3). This way corresponds to the prescription given by the AISIS213 in terms of global mechanism control and it is equivalent to adopt the relevant overstrength factor (Ω_i) at each storey. For connections, also Eq. (4) has been satisfied. Table 4 shows the criteria adopted for the design of the three different wall configurations.

Table 4: Design criteria

Configuration walls	WLE	WLD	WHD
Q	1 (EN 1998-1)	2.5 (AISI S213)	2.5 (AISI S213)
H_d	50kN	40kN	80kN
Eq (1): $\frac{A_{net}}{A} \geq 1.1 \cdot \frac{\gamma_{M2}}{\gamma_{M0}} \cdot \frac{f_{yk}}{f_{tk}}$	NO	OK	OK
Eq (3): $1.1 \cdot \gamma_{ov} \cdot R_{fy}$	NO	OK	OK
Eq (4): $F_{v,Rd} \geq 1.2 F_{b,Rd}$ and $\Sigma F_{v,Rd} \geq 1.2 F_{n,Rd}$	OK	OK	OK

3 EXPERIMENTAL VALIDATION OF THE DESIGN CRITERIA

In order to validate the design criteria for CFS structures in seismic area, the prescriptions and requirements of Eurocodes and AISI S213 have been also evaluated on the basis of the experimental data [3]. In particular, for each wall configuration defined in Section 2, the behaviour factor has been evaluated by considering the results of both monotonic and cyclic test results. The behaviour factor has been defined by the R_d (ductility) and R_o (overstrength) factors, as given in Uang [7]. The ductility-related force modification factor R_d can be evaluated as follows:

$$R_d = \sqrt{2\mu - 1} \text{ with } \mu = \frac{d_{max}}{d_y} \tag{7}$$

where μ 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 is the wall height): 1.5%, 2% and 7%. 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 those provided by FEMA 356 [8] for traditional concentrically braced structures at the Life Safety and Collapse Prevention limit states, respectively. On the other hand, the limit of 7% is the maximum displacement capacity obtained by shaking table tests [9] on wooden shear walls, which represent a system similar to the investigated one. The overstrength-related force modification factor R_o can be evaluated through the following formulation:

$$R_o = R_{sd} \cdot R_{\Phi} \cdot R_{yield} \cdot R_{sh} \tag{8}$$

where R_{sd}= H_c/H_d, with H_c and H_d design wall resistance and seismic demand, respectively; R_Φ= 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. Tables 4 and 5 show the values of the behaviour factor obtained by the experimental results. In particular, for WLE walls d_{max}/h result always less than 1.5%, so the evaluation of q is limited to the case $d=d_{max}$. In the case of WLE walls (Table 5), it can be noted that the behaviour factor values proposed by AISI S213 for Conventional construction category (q=1.6) is always smaller than those experimentally obtained (q=2.0÷ 2.2). As far as WLD and WHD walls are concerned, the value provided by AISI S213 in case of Limited ductility braced walls (q = 2.5) represents a lower limit of the obtained behaviour factors (q=2.5÷3.0 for 1.5%, q=3.0÷4.3 for 2%, q=6.4÷8.2 for 7%) (Table 6).

Table 5: Behaviour factor for WLE

Test	R_d	R_o	q
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 6: Behaviour factor for WLD and WHD

	1.5% interstorey drift			2% interstorey drift			7% interstorey drift		
Test	R_d	R_o	q	R_d	R_o	q	R_d	R_o	q
WLD-M1	2.18	1.42	3.09	2.58	1.43	3.68	5.08	1.53	7.76
WLD-M2	2.28	1.40	3.20	2.70	1.43	3.87	5.29	1.56	8.24
WLD-C1	2.18	1.51	3.29	2.58	1.50	3.88	5.08	1.53	7.75
WLD-C2	2.39	1.53	3.65	2.82	1.51	4.26	4.77	1.64	7.83
WHD-M1	1.89	1.38	2.60	2.26	1.38	3.11		(1)	
WHD-M2	1.83	1.37	2.51	2.19	1.41	3.08	4.40	1.46	6.40
WHD-C1	1.96	1.45	2.84	2.33	1.46	3.41	4.64	1.51	7.02
WHD-C2 (Pull)	2.12	1.41	2.99	2.52	1.44	3.63		(2)	
WHD-C2 (Push)	1.98	1.41	2.80	2.36	1.44	3.39	4.69	1.41	6.64

(1) The test was interrupted because of the occurrence of local buckling of the tracks;
(2) The diagonal net area collapse before reaching the limit of 7%.

An important finding is represented by the design formulation (Eq.1), aimed at preventing the failure of the diagonal net area at fastener holes, that is not always effective as demonstrated by the experimental evidence. In fact, the experimental tests on dissipative systems showed that, in all connection tests and cyclic wall tests, the collapse is always due to the net area fracture, while the wall collapse in monotonic tests is due to the yielding of the tension diagonals without rupture in the field of the investigated displacements. The difference between the observed failure mechanisms in the connections and monotonic wall tests can be explained by comparing the deformations reached in the two cases (Fig. 2). In fact, the deformations recorded at ultimate condition in the case of connections (ε_c) are significantly higher than those obtained in wall tests (ε_w). Therefore, the connection behaviour at failure is conditioned by the hardening that makes the force in the gross section ($A\cdot\sigma_c$) greater than the net section resistance ($A\cdot\sigma_c > A_{net}\cdot f_t$). On the contrary, in the case of walls, the maximum force in the diagonal gross section is not enough to entail the failure of the net area ($A\cdot\sigma_w < A_{net}\cdot f_t$). The occurrence of net sections failures, observed in the cyclic wall tests, can be caused by low cycle fatigue phenomena amplified by the stress concentrations due to fastener holes.

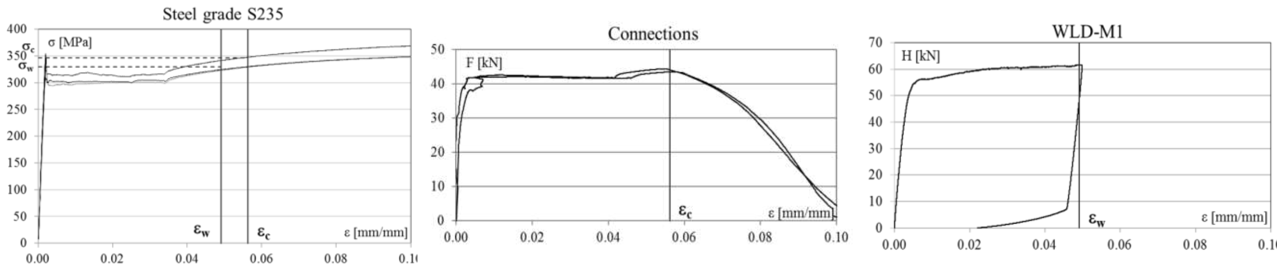

Fig. 2: Comparison of deformation related to testing of diagonal strap braced walls and connection

As far as the capacity design criteria are concerned, the experimental results showed that the adopted formulation (Eq. 3) 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. 4). In fact, no shear failure of the screws occurred in both connections and wall tests. The experimental results do not allow to make any consideration about the global mechanism because the performed tests on walls are representative of only one storey.

4 CONCLUSION

This paper presents a critical analysis of the seismic design criteria for strap braced CFS systems. In particular, on the basis of prescriptions given by the American code AISI S213 for CFS structures and those provided by Eurocodes for traditional concentrically braced frames, seismic design criteria for strap braced CFS structures are proposed. The experimental results allowed the validation of assumed design hypotheses. The behaviour factor values provided by AISI S213 are widely confirmed by the experimental tests and, the code values represents lower limits of the one obtained experimentally. In addition, the requirements concerning the capacity design given in the Eurocodes, for traditional systems, are also reliable for the CFS structures. As a further development, an extended numerical study including non-linear dynamic analysis should be performed for a more accurate estimation of the behaviour 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.

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