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SEISMIC RESPONSE OF CFS STRAP-BRACED STUD WALLS: THEORETICAL STUDY

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HIGHLIGHTS

- A critical analisys of design requirements for CFS strap-braced walls is discussed.
- The procedure for the evaluation of wall stiffness and resistance is illustrated.
- A case study involving the design of three residential buildings has been developed.

ABSTRACT

The use of cold-formed steel (CFS) profiles in low-rise residential buildings has increased in European construction sector. The reason of this interest is related to potentialities offered by this constructive system, which are the high structural performance, lightness, short construction time, durability and eco-efficiency. Nevertheless, the current structural codes, such as Eurocodes, do not provide enough information about the seismic design of this structural typology. In an effort to investigate the seismic response of CFS structures, a theoretical and experimental research has been carried out at University of Naples Federico II, with the main aim to support the spreading of these systems in seismic areas. This study focuses on an "all-steel design" solution in which strap-braced stud walls are the main lateral resisting system. In the present paper the outcomes of theoretical phase are shown with the aim of defining the criteria for the seismic design of such structures. In particular, a critical analysis of the requirements for CFS systems provided by the American code AISI S213 has been carried out by comparing it with those given by Eurocodes for traditional braced steel frames.

1 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 European construction sector and in the recent research studies [1]. This structural typology consists of a dry constructive system, in which both floors and walls are made with CFS profiles arranged with small spacing and completed at the end by means of track profiles. The seismic behaviour of this system is strictly related to the in-plane response of floors and walls, which represent the main seismic/lateral resistant system. In general, the design against seismic actions can be carried out by using two different approaches: “all-steel” and “sheathing-braced”. In the case of the “all-steel” approach, only steel elements are considered as part of the load-bearing structure and, in order to resist to lateral actions, the introduction of a bracing system, made generally with flat straps in X configuration, is required. Instead, in the “sheathing-based” design approach, the bracing contribution is provided by the interaction between the steel frame and the sheathing panels, generally wood or gypsum based.

Despite the several advantages related to their use, the main European structural reference code for seismic design, the Eurocode 8 part 1 (EN 1998-1) [2], 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 [3] 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 under wind and seismic loads of different lateral resistant CFS systems for Canada, Mexico and United States. Both sheathed shear walls and strap-braced systems are considered in the standard. In particular, special requirements for seismic design, such as the values of the behaviour factor (q) or the seismic response modification factor (R) using the American terminology, aspect ratio limitations, capacity design rules for non-dissipative elements, are provided for both systems. In the case of shear walls, a specific formulation for the calculation of wall deflection and tabulated values of wall resistance bases on experimental results are provided. The standard also provides the requirements for the seismic design of floor diaphragms made with CFS framing. In addition, in order to facilitate the use and the understanding of the code, a thorough commentary illustrates the research and scientific background of the standard. In particular, the design provisions for strap-braced walls in terms of force modification factor and capacity design approach are based on the research carried out by Serrette [4], Al-Kharat and Rogers [5-7], Comeau and Rogers [8] and Velchev and Rogers [9]. An evaluation of seismic requirements of AISI S213 was carried out by Velchev et al. [10]. Different configurations of strap-braced walls with diagonals connected by welds or screws, designed according to the capacity design rules provided by the code, were tested. The experimental results were used to measure the wall ductility and to determine test-based values of the behaviour factor. Further experimental studies and researches on the seismic response of strap-braced walls are presented in Section 2.

In the last decade, many research activities on the CFS structures were also 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. In particular, theoretical and numerical studies were carried out on the response prediction of sheathed shear walls [11-15], the evaluation of the behaviour factor [16-17] and the definition of specific design procedure [18-20]. In addition, the results of these studies have found a practical reflection in the design and execution of an important building in Italy [21].

As an effort to investigate the behaviour of such structures designed according to "all-steel" approach, 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 Italian 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. The present paper shows the results and findings of the theoretical phase of the research. In particular, the state-of-theart of the previous experimental researches carried out on diagonal strap-braced walls are presented in Section 2. The typical arrangement of diagonal strap-braced walls together with the methods for the prediction of lateral wall stiffness and resistance are illustrated in Section 3. Among the different steel seismic-resistant systems regulated by the EN 1998-1, traditional concentrically braced frame with X diagonal represents the closest system to the investigated one. In Section 4 a critical analysis of the AISI S213 and EN 1998-1 standards is illustrated, with particular reference to the analysis and comparison of the existing provisions for the two similar structural typologies (strap-braced CFS system and traditional concentrically braced). Based on the results of the critical analysis, the design hypotheses have been defined for the development of the design of case study buildings (Section 5), from which the wall configurations tested in experimental phase have been selected. The results of the experimental phase of the research are widely described and illustrated in the companion paper [22].

2 LITERATURE REVIEW

The unconventionality of CFS structures has motivated, in recent times, the experimental characterization by many international research groups. A rather large number of experimental programs, aimed at investigating the seismic performance of CFS strap-braced stud walls, have been carried out (Adham et al. [23], Serrette [24], Serrette [4], Serrette and Ogunfunmi [25], Fulop and Dubina [26], Tian et al. [27], Al-Kharat and Rogers [7], Casafont et al. [28], Moghimi and Ronagh [29] and Velchev et al. [10]). In particular, the studies were focused on the monotonic and cyclic response of these systems in order to evaluate the contribution provided to the wall shear resistance by flat strap braces combined with gypsum sheathing boards in some cases. The test typologies, the specimens with indications of the main wall components and the investigated parameters, affecting the wall seismic behaviour, are synthesized in Table 1 and Table 2 for each experimental research. The research objectives were to provide information about the wall behaviour in terms of lateral load capacity, stiffness, energy dissipation and failure modes. In particular, the effect of the following aspects on the wall lateral performance was investigated: (i) contribution of steel framing without any bracing system, (ii) steel flat strap X bracing behaviour (bracing side, strap dimensions, steel material properties), (iii) type of frame-to-strap connections (screws, bolts and welds), (iv) wall corner details, (v) contribution of gypsum sheathing boards, (vi) wall aspect ratio and (vii) loading type (monotonic and cyclic). The main outcomes of these researches, summarized below, have been considered for the planning and the evaluation of the experimental study presented in this paper.

The contribution of the steel framing without any bracing system to the wall lateral resistance is relatively small, as pointed out in the tests carried out by Serrette and Ogunfunmi [25] and Tian et al. [27]. Specifically, Tian et al. [27] estimated that the frame itself offers about 5% of the total strength of a braced frame. This result demonstrated the effectiveness to use the diagonal straps in CFS stud walls [25].

Studies concerning the steel flat strap X bracing behaviour include wall specimens realized with strap braces on one or both wall sides, different strap dimensions and steel material properties [4, 23, 25, 27, 28]. The experimental results highlighted that the compressed diagonal straps do not collaborate to the wall lateral strength [23]. Therefore, the design of the strap is a key issue in the seismic behaviour of CFS strap-braced stud walls and, for this reason, these studies were devoted to optimize the flat strap X bracing contribution in the wall lateral response. The walls braced with steel flat straps installed in an X configuration on both sides showed a better performance than one-side X-braced walls [23, 25, 27]. In particular, it was indicated that the one-side X-braced walls failed by excessive lateral deflection [25] and then the maximum load was reduced by more than 50% compared to two-sides X-braced walls [27]. In addition, Serrette [4] pointed out that the flat straps on one wall side may cause an eccentric loading on tracks and chord studs, which is particularly important for heavily loaded walls. This eccentricity may induce the local buckling phenomena in chord studs and tracks, due to combined bending and axial loads, and thus the premature wall failure before the development of the strap capacity. The effect of strap geometry on the wall behaviour was evaluated by Adham et al. [23], Serrette and Ogunfunmi [25] and Tian et al. [27] at varying of the strap width and thickness. The experimental results demonstrated that the use of wider straps allows the increment of the wall lateral resistance and stiffness and the added benefit to provide more room for connections. The steel material properties of wall frame were investigated by Serrette [4] and Casafont et al. [28]. In particular, Serrette [4] recommended that the chord studs, tracks and frame-to-strap connections must be designed for a brace force greater than the one corresponding to the minimum specified value of strap yield strength, since this last is usually smaller than the actual yield strength. Furthermore, Casafont et al. [28] pointed out that the adoption of a steel grade for the straps lower than for the other wall members (studs, tracks and gussets) increases the ductile behaviour and dissipation capacity of the tested walls.

The frame-to-strap connection behaviour highly influences the wall strength and ductility and, therefore, some experimental research [7, 10, 28, 29] were devoted to investigate this aspect. In particular, Casafont et al. [28] carried out an experimental campaign on the seismic behaviour of screwed frame-to-strap connection. This study indicated that the strap-braced stud walls should be designed in order to fail for effect of brace yielding followed by strap net-section failure, which is a preferable collapse mode that allows a good wall seismic performance. Therefore, it was recommended to use screwed frame-to-strap connections, because the small diameter of screws involves a net-section area greater than other fastener types (such as bolts) and this aspect increases the strap dissipative capacity. Furthermore, Velchev et al. [10] showed that the welded and screwed frame-to-strap connections exhibit similar inelastic behaviour if properly designed and detailed to avoid strap net section fracture before the brace yielding.

The studies carried out by Fulop and Dubina [26], Al-Kharat and Rogers [7] and Casafont et al. [28] highlighted that strengthening of the corner foundation anchorage details is crucial, because it affects considerably the lateral strength, stiffness and ductility of the wall system. In fact, the corner detail should be designed so that the force is directly transmitted from the brace to the anchoring, by means clip angles or hold-downs [26], in order to avoid the failure due to bending collapse and local buckling of the bottom tracks. Furthermore, the wall seismic performance could be improved by reinforcing the tracks, by selecting a thicker track section [7] and by reducing the eccentricity of the anchor bolt connection with respect to the strap axis [28].

The effect of gypsum sheathing boards, usually adopted as wall finishing, on the wall lateral performance was evaluated by Adham et al. [23], Serrette and Ogunfunmi [25] and Moghimi and Ronagh [29] at varying of their thickness. The results of these studies demonstrated a significant increment (about 130%) provided by sheathing panels to the wall resistance when they are applied on strap-braced stud walls [25]. In addition, if both strap and sheathing panels are considered in the lateral load-carrying capacity, then straps should be pretensioned in order to be effective on first loading, as demonstrated by Serrette and Ogunfunmi [25].

The effect of variation of the wall aspect ratio, defined as the height-to-length ratio, was investigated by Velchev et al. [10], which studied the behaviour of walls with the following aspect ratios: 1:1, 2:1 and 4:1. Specifically, the study argued that the use of aspect ratios greater than 2:1 should be avoided, because the 4:1 aspect ratio walls experienced combined axial compression and flexural failure of the chord studs with a significant reduction in the lateral stiffness.

The effect of loading type (monotonic and cyclic) on the wall lateral behaviour was investigated by Adham et al. [23], Serrette [24], Serrette [4], Serrette and Ogunfunmi [25], Fulop and Dubina [26], Tian et al. [27], Al-Kharat and Rogers [7], Casafont et al. [28], Moghimi and Ronagh [29] and Velchev et al. [10]. The studies on the monotonic and cyclic performance of the strap-braces walls revealed a satisfactory experimental behaviour in terms of energy dissipation, stiffness, strength and deformation capacity [10, 23, 25, 27, 28] when the walls are properly designed. In particular, Al-

Kharat and Rogers [7] recommended to apply the capacity design principles and to consider the strap material overstrength for the estimation of the brace yield capacity, in order to ensure a ductile wall failure governed by the strap yielding with minor damage of the other wall components (brace connections, tracks, studs, gusset plates and hold-downs). With reference to the wall cyclic response, these studies observed a symmetric behaviour characterized by a strong pinching of hysteresis loops larger than the one registered in similar walls braced with sheathing panels [26]. Furthermore, a small stiffness and strength degradation by increasing the cycle number was highlighted in Adham et al. [23] and Casafont et al. [28].

3 DIAGONAL STRAP-BRACED CFS WALLS

3.1 Description of the wall

In CFS stick-built constructions designed according to the “all-steel” approach, the diagonal strap-braced walls represent the main seismic resistant systems. The structure of a typical diagonal strap-braced wall is shown in Figure 1. In particular, the metal frame of the wall consists of stud members, having lipped channel section (C-shaped), generally spaced at 600 mm and connected at the ends by track members, made with unlipped channel sections (U-shaped). In order to provide the in-plane bracing to the metal frame, steel straps in X configuration are installed on one or both wall sides and are generally connected to the frame by means of suitable gusset plates. Because of the high slenderness of the steel straps used as bracing systems, they are considered active only in tension. Therefore, the lateral loads are fully absorbed by the diagonal in tension, which transmits significant compression force to the chord stud and the track. In order to avoid the local buckling due to compression transmitted by diagonals, the terminal fields of the track should be reinforced, e.g. by means C-shaped profiles, in such a way to obtain a built-up box profile [10]. For the same reason, the “back-to-back” coupled C-shaped profiles are generally used for chord studs. In order to improve the buckling behaviour of chord and interior studs by reducing their unbraced length, flat straps can be placed at the mid-height of the wall specimens and connected to blocking members at the ends of walls. At the ends of the chords studs, "hold-down" devices and tension anchors are generally used to transfer the uplift forces. In addition, mechanical anchors (shear anchors), placed along the tracks, are generally installed to resist against the wall slipping. All connections are usually made with self-drilling screws.

3.2 Evaluation of wall resistance

In general way, the design lateral resistance of CFS diagonal strap-braced walls can be evaluated as the strength associated to the weakest of the possible failure mechanisms for each wall components. Therefore, the design lateral wall resistance (H_c) can be written as follows:

$$H_c = \min\left(H_{c,d}; H_{c,c}; H_{c,g}; H_{c,s}; H_{c,t}; H_{c,a}\right) \qquad (1)$$

where $H_{c,d}$ is the lateral resistance due to tension failure of diagonal strap braces, $H_{c,c}$ is the lateral resistance due to the failure of diagonals connections, $H_{c,g}$ is the lateral resistance due to the net failure of the gusset plates, $H_{c,s}$ is the lateral resistance due to studs failure, $H_{c,t}$ is the lateral resistance due to track failure, and $H_{c,a}$ is the lateral resistance due to frame-to-foundations anchors failure.

In the case of the diagonal strap braces, the possible failure mechanisms are the yielding of the diagonals and the net section failure in correspondence to the fastener holes of diagonal-to-frame connection. Therefore, the lateral wall resistance associated to the diagonal failure can be evaluated by the following equation:

$$H_{c,d} = n_d \cdot \min(N_{pl,Rd}; N_{u,Rd}) \cos\alpha \qquad (2)$$

where n_d is the number of diagonals (1 for diagonals on one wall side only and 2 for diagonals on both sides), $N_{pl,Rd}$ is the design plastic resistance of the diagonal, $N_{u,Rd}$ is the design resistance of the net cross section at fasteners holes and α is the angle of the diagonal with respect to the horizontal.

In the case of the diagonal-to-frame connections made with self-drilling screws, the lateral wall resistance of the wall corresponding to the connection failure is given by:

$$H_{c,c} = n_d \cdot n_s \cdot \min\left(F_{b,Rd}; F_{v,Rd}\right) \cos\alpha \qquad (3)$$

where n_s is the number of screws in one diagonal-to-frame connection, $F_{b,Rd}$ is the design bearing resistance of the connected plates per one screw and $F_{v,Rd}$ is shear resistance of one screw.

In the case of gusset plates are used for the connections between the diagonal brace and the steel frame, also these elements must be checked and the corresponding lateral wall resistance can be evaluated as follows:

$$H_{c,g} = n_d \cdot F_{n,Rd} \cos\alpha \qquad (4)$$

where $F_{n,Rd}$ is the design resistance of the theoretical effective net cross-section area of the gusset plate at the end of the connection according to the well-know Whitmore section.

The failure of the metal frame under lateral load is generally related to the buckling due to compression of chord studs or tracks. Therefore, the lateral wall resistance associated to these elements can be evaluated with the following expressions:

$$H_{c,s} = \frac{\left(N_{s,Rd} - N_{Ed,G}\right)}{h} \cdot L \quad (5)$$

$$H_{c,t} = N_{t,Rd} \quad (6)$$

where h is the wall height, L is the wall length, $N_{Ed,G}$ is the acting axial force due to the gravity loads, $N_{s,Rd}$ and $N_{t,Rd}$ are the design buckling resistance of studs and tracks, respectively.

In the case of frame-to-foundations anchors, the failure can occur for overturning or slipping of the wall. The wall overturning involves the tension resistance of the anchors between the chord studs and the foundation, while the wall slipping involves the shear resistance of the anchors between the wall track and the foundation. The lateral wall resistance associated to the anchors can be evaluated as follows:

$$H_{c,a} = \min\left(\frac{\left(N_{a,Rd} + N_{Ed,G}\right)}{h} \cdot L; n_a \cdot V_{a,Rd}\right) \quad (7)$$

where $N_{a,Rd}$ is design resistance of each tension anchor; n_a is the number of shear anchors and $V_{a,Rd}$ is the design resistance of each shear anchor.

The resistance of the different wall elements (i.e., $N_{pl,Rd}$, $N_{u,Rd}$, $F_{b,Rd}$, $F_{v,Rd}$, $F_{n,Rd}$, $N_{s,Rd}$, $N_{t,Rd}$, $N_{a,Rd}$ and $V_{a,Rd}$) can be calculated through different available methodologies. In the presented research the methods given in EN 1993-1-3 [30] have been used.

3.3 Evaluation of wall stiffness

The lateral displacement (d) at the wall top under horizontal loads can be evaluated by taking into account the contributions due to main wall structural components (Fig. 2), such as diagonals in tension (d_d), connections between frame and diagonal braces (d_c) and the anchorages between frame and foundations (d_a). In particular, the lateral wall displacement can be evaluated as follows:

$$d = d_d + d_c + d_a \quad (8)$$

This equation is valid for the cases in which there are no slipping displacements between the wall and the foundation. In the real cases, this type of displacement is generally negligible.

In this way, the wall can be considered as a system of elastic springs in series corresponding to the different structural components. Therefore, the wall lateral stiffness can be evaluated with the following equation:

$$K = \frac{1}{\frac{1}{K_d} + \frac{1}{K_c} + \frac{1}{K_a}} \quad (9)$$

The stiffness contribution related to the axial deformability of the diagonals in tension can be obtained by the following expression:

$$K_d = \frac{n_d \cdot E \cdot A_d \cdot \cos^2 \alpha}{L} \qquad (10)$$

where n_d is number of diagonals in tension (1 for diagonals on one wall side only and 2 for diagonals on both sides), A_d is diagonal cross section area, α is the angle of the diagonal with respect to the horizontal, E is steel Young's modulus and L is the wall length.

The stiffness due to the deformability of the diagonal-to-frame connections can be obtained with the following expression:

$$K_c = \frac{n_d \cdot n_s \cdot k_s \cdot \cos^2 \alpha_d}{2} \qquad (11)$$

where n_s is the number of screws in one diagonal-to-frame connection and k_s is the shear stiffness of a single screw connection that is generally obtained on the basis of experimental data. In their calculation Velchev and Rogers [9] assumed k_s = 1.8 kN/mm on the basis of experimental tests on a connection representing those used for wall specimens. In this study, as discussed in the companion paper [22], test results of elementary and diagonal-to-frame connections are considered, where the obtained values of the shear stiffness for a single screw ranges from 3.8 to 4.6 kN/mm or from 3.8 to 6.0 kN/mm on the basis of test results on elementary connection or frame-to-strap connections, respectively. Useful information about fasteners stiffness used for CFS systems can be found in ECCS Document No. 88 [31]. For screw diameters in the range between 4.1 and 4.8 mm and steel sheets thickness up to 1.20 mm, a stiffness value of 4.0 kN/mm is recommended in this document.

The lateral stiffness corresponding to the deformation of the anchors in tension can be calculated through the following expression:

$$K_a = \frac{k_a \cdot L^2}{h^2} \qquad (12)$$

in which h is the wall height and k_a is the axial stiffness of the anchorage system in tension. The values of k_a are generally given by manufacturers. For its products, Simpson Strong-Tie [32] provides the values of the total elongation of the anchorage system (fasteners, hold-down and anchor bolt) under an allowable load. This information allows to determine the stiffness k_a that, for different hold-down devices, stud thicknesses and fasteners types, ranges from 7 to 68 kN/mm. In particular, the stiffness for Simpson’s hold-down similar to those used for the tested wall prototypes are in the range between 15 and 38 kN/mm. The stiffness of the anchorage system used in the wall specimen tested in the presented research has been evaluated on the basis of the up-lift displacements measured during the tests, and the obtained value is about 20 kN/mm [22].

4 CFS VS TRADITIONAL BRACED SYSTEMS IN CURRENT SEISMIC CODES

4.1 Basis of the comparison

The applicability and the diffusion of a structural system in a seismic area are related to the clarity and the interpretation of technical prescriptions. In order to identify the peculiarities of the seismic design of strap-braced walls, the specific prescriptions for this system provided by the AISI S213 [3] have been deeply examined. The AISI prescriptions have been compared with those provided by EN 1998-1 [2] for traditional concentrically braced frames with X diagonals. This comparison aims to define the design peculiarity of the examined seismic resistant system and to individuate the similarities with the design rules of a traditional steel systems provided by Eurocodes, with the objective of defining specific prescriptions for strap-braced walls according to the European design philosophy. In the following sections, the comparison of the prescriptions provided by the two examined codes is discussed in terms of behaviour factor, design of diagonal members and capacity design rules.

4.2 Ductility classes, behaviour factor and height limits

In general, seismic codes classify buildings on the base of the ductility requirements and the dissipation capacity of a given seismic resistant system. The behaviour factor q is the main design parameter that quantifies the inelastic capacity of the structural system and it represents a fundamental issue to deepen when design prescriptions for a new seismic resistant system are going to be proposed.

For seismic resistant steel buildings, the EN 1998-1 defines three structural ductility classes: low (DCL), medium (DCM) and high (DCH). The DCL class structures have a low dissipative behaviour and their design is carried out without taking into account significant non-linear behaviour. In this case, the recommended value for the behaviour factor is 1.5. Structural systems belonging to DCM and DCH classes have a higher ability to dissipate energy and are designed to resist seismic actions taking into account their inelastic capacity. The design requirements of DCM and DCH differ for limitation in terms of class section of dissipative members and rotation capacity of connections. The EN 1998-1 considers the traditional X-braced steel frames as tension-only bracing systems and it currently does not differentiate between DCM and DCH ductility classes, except for the section classes of the dissipative members, as detailed in the following. Although the

EN 1998-1 considers the DCM and DCH classes for X-braced steel frames, the prescribed value of the behaviour factor is always 4 for both of them.. In the case of non-regular buildings in elevation, the behaviour factor has to be reduced by 20%.

On the other hand, the AISI S213 for Canada defines two categories for diagonal strap-braced wall. 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. The behaviour factor is named force modification factor by AISI S213 and it is defined as the product of ductility related factor, R_d, and overstrength related factor, R_o. In particular, 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 exceed 15 m.

In the case of United States, the seismic modification factor (R) should be taken equal to or less than 3 according to the applicable building code for non-detailed systems, while greater values can be taken for structures designed through the capacity design approach. For the latter ones, the American code ASCE-07 [33] provides a seismic modification factor equal to 4.

4.3 Slenderness limits and diagonals design

The seismic design of traditional X-braced frames according to EN 1998-1 is performed by considering that the seismic forces have to be absorbed only by the tension diagonals. In the case of building having more than two storeys, the code prescribes that the normalized slenderness of the diagonal members has to be limited in a given range ($1.3 \leq \bar{\lambda} \leq 2$). The upper limit has the aim to ensure a good dissipative behaviour by reducing the pinching of the hysteretic cycles and to avoid the oligocyclic fatigue fracture due to occurrence of local buckling as well as the excessive out-of-plane distortions due the buckling of the diagonal in compression. The lower limit is related to the structural scheme with only active tension diagonals, assumed for the ultimate condition, and it aims to avoid the columns overloading in pre-buckling phase. In addition, in order to ensure an adequate ductility by reducing local buckling phenomena, the cross-sectional class of the seismic resistant dissipative elements for DCM structures should be 1 or 2 while, in the case of DCH structure, only class 1 sections can be used.

On the contrary, the slenderness limits imposed for traditional X-braced systems are not relevant for diagonal strap-braced walls because, in this case, the diagonals are straps are not able to resist to any compression loads. Therefore, since the initial stage, the seismic force is really absorbed only by the tension diagonals. For this reason, the AISI S213 does not provide any prescriptions about the diagonal slenderness and it expressly allows slenderness values for strap members exceeding 200. Also for the cross-sectional class of the members, the AISI S213 does not provide any limitations because studs (columns) and tracks (beams) of the considered system are generally made of slender CFS profiles (class 4).

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 [34], the design plastic resistance of the diagonal cross section ($N_{pl,Rd}$) has to be less than the ultimate design resistance of the net cross section at fasteners holes ($N_{u,Rd}$):

$$N_{pl,Rd} \leq N_{u,Rd} \tag{13}$$

with

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} \tag{14}$$

$$N_{u,Rd} = 0.9\frac{A_{net}\, f_u}{\gamma_{M2}} \tag{15}$$

where A is the gross cross-section area; f_y is the characteristic yield strength; γ_{M0}= 1.00 is the partial safety factor for yielding resistance of gross cross-section; A_{net} is the net area of the cross-section at the fasteners holes; f_u is the characteristic ultimate strength; and γ_{M2} = 1.25 is the partial safety factor for the tensile resistance of net sections.

A similar prescription for the design of strap bracing members is provided by the AISI S213, in which the expected yield strength has to be lower than the expected tensile strength of the net cross section:

$$A_g R_y F_y \leq A_n R_t F_t \tag{16}$$

where A_g is the gross cross-section area; F_y is nominal yield strength; A_n is the net area of the cross-section at the fasteners holes; F_t is nominal ultimate tensile strength; R_y and R_t are the coefficients used for estimate the expected yield and tensile strength, respectively. These coefficients are provided by the standard as function of the steel grade.

The prescriptions provided by the two codes for diagonal design presents a conceptual difference. In particular, the EN 1998-1 considers the design values of the gross and net section resistances through the partial safety factors, while the AISI S213 uses the expected resistances by introducing the R_y and R_t coefficients, which are obtained starting from a survey of North American CFS producers. Despite this conceptual difference, these prescription can be compared by writing the Equation (13) and (16) in terms of ratio between the gross and the net section areas as follows:

$$\frac{A_{net}}{A} \geq 1.1 \cdot \frac{\gamma_{M2}}{\gamma_{M0}} \cdot \frac{f_y}{f_u} = \alpha \cdot \frac{f_y}{f_u} \text{ with } \alpha = 1.1 \cdot \frac{\gamma_{M2}}{\gamma_{M0}} = 1.38 \quad (17)$$

$$\frac{A_n}{A_g} \geq \frac{R_y}{R_t} \cdot \frac{F_y}{F_u} = \beta \frac{F_y}{F_u} \text{ with } \beta = \frac{R_y}{R_t} \quad (18)$$

In particular, the α coefficient, described in the EN 1998-1 prescription, depends only by the ratio between the partial safety factors, and its value is constant and equal to 1.38. Instead for the AISI S213 the values of β coefficient ranges from 1.00 to 1.27 depending on the ratio between R_y and R_t (Table 3), which are function of the steel grade. The results show that the α-value represents an upper limit for β-values (Fig. 3). Therefore the design prescriptions for diagonal design provided by EN 1998-1 are conservative respect to the AISI S213 ones.
In order to achieve the gross cross-section yielding prior than the net section failure, the AISI S213 provides a further suggestion, based on the experimental findings of Velchev and Rogers [9], which can be expressed as follows:

$$\frac{R_t \cdot F_u}{R_y \cdot F_y} \geq 1.2 \quad (19)$$

Table 4 and Figure 4 show the values of the ratio given in Equation (18) for the different steel grade provided by AISI S213.

4.4 Capacity design rules and global mechanism

In general, for both CFS and traditional X-bracing systems, the most ductile failure mechanism is the yielding of the tension diagonal, which can be ensured by providing an adequate overstrength to other possible mechanisms corresponding to the failure of non-dissipative elements, such as connections, beams and columns.
As far as the design of the connections for dissipative members is concerned, the EN 1998-1 prescribes that the following condition should be satisfied:

$$R_d \geq 1.1 \cdot \gamma_{ov} \cdot R_{fy} \quad (20)$$

where R_d is the connection resistance; R_{fy} is the design plastic resistance of the connected dissipative member that, in the examined case, can be evaluated through the Equation (13); γ_{ov} is the material overstrength factor, recommended equal to 1.25.

In addition, a specific prescription, even if not closely related to seismic design, is provided by EN 1993-1-3 [30] for self-drilling screws connections, which are the main connecting system used in CFS structures. In particular, in order to provide an adequate deformation capacity and to avoid the brittle failure of the fasteners, the following equations should be satisfied:

$$F_{v,Rd} \geq 1.2 F_{b,Rd} \text{ or } \Sigma F_{v,Rd} \geq 1.2 F_{n,Rd} \qquad (21)$$

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.

For the design of beams and columns (non-dissipative elements), subjected mainly to axial forces, the following condition should be satisfied:

$$N_{pl,Rd}\left(M_{Ed}\right) \geq N_{Ed,G} + 1.1 \cdot \gamma_{ov} \cdot \Omega \cdot N_{Ed,E} \qquad (22)$$

where $N_{pl,Rd}(M_{Ed})$ is the design buckling resistance of the beam or column evaluated by considering the interaction with the bending moment (M_{Ed}), that is generally null for the examined systems; $N_{Ed,G}$ 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 $\Omega_i = N_{pl,Rd,i} / N_{Ed,i}$ with $N_{pl,Rd,i}$ and $N_{Ed,i}$ the design plastic resistance and seismic axial force in i^{th} diagonal, respectively. Therefore, in this condition, the seismic forces acting in the non-dissipative elements are those corresponding to the first plastic event in the diagonals.

Taking into account the i^{th} diagonal and the relevant Ω_i, the fulfilment of Equation (22) consists in designing the non-dissipative elements for a force corresponding to the attainment of the plastic resistance of the tension diagonal. In this case, the application of Equation (22) for beams and columns would be the same as the use of Equation (20) for the design of connections. 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 EN 1998-1 requires that the maximum overstrength factor (Ω_i) does not differ from the minimum one 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:

$$A_g \cdot R_y \cdot F_y \qquad (23)$$

The resistance of the non-dissipative members as studs, tracks and connection should be calculated according the specification of AISI S100 [35]. 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.

In Figure 5, the two structural schemes for the distribution of the only seismic force on columns according to the capacity design rules provided by EN 1998-1 and AISI S213 are depicted. For the sake of completeness, the effects of gravity loads and possible eccentricities have to be also considered in the calculation of non-dissipative elements.

In order to compare the capacity design rules provided by the two codes, the Equation (20), assumed as general formulation for EN 1998-1, can be written as follows:

$$1.1 \cdot \gamma_{ov} \cdot R_{fy} = 1.1 \cdot \gamma_{ov} \cdot \frac{A \cdot f_y}{\gamma_{M0}} = \delta \cdot A \cdot f_{yk} \text{ with } \delta = 1.1 \cdot \frac{\gamma_{ov}}{\gamma_{M0}} = 1.38 \qquad (24)$$

It has to be noticed that the mathematical meaning of δ coefficient is the same of R_y in Equation (23). In particular, the δ coefficient is constant and equal to 1.38, while R_y depends on yield strength of steel (f_y) and ranges from 1.1 to 1.5 (Table 5).

The comparison of the two coefficients (Fig. 6) shows that the coefficient R_y decreases with the increasing of the yield strength and it is higher, then conservative, than δ for low values of yield strength (230÷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 (Ω_i), which directly affects also the design of the diagonal members. The AISI S213 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.

5 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 considered as case studies. They are designed according to different hypotheses about the design criteria and loads. The studied structures have all the same rectangular plan, which covers an area of 220 m^2, and they are constituted by one, two and three storeys, with a 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 on the walls (Fig. 7). Therefore, the buildings are regular, from seismic point of view, both in plan and elevation. These buildings have been designed considering the environmental loads of two different Italian locations: Rome and Potenza, which are characterized by medium-low and medium-high intensities of snow and seismic loads, respectively. In order to take into account the different possible technological and architectural configuration of the structural elements (flooring, claddings, insulating systems, etc.), a range of values has been assumed for the evaluation of dead loads, as shown in Table 6. In this way, each building has been designed by considering the minimum and maximum possible dead loads distribution. Live loads for residential buildings equal to 2.00 kN/m^2 have been considered for both floors and roofs. The snow loads have been calculated for the assumed geographic locations according to Italian construction technical code [36] and they are equal to 0.48 and 1.81 kN/m^2 for Rome and Potenza, respectively.

The seismic actions and the design spectra have been defined according to Italian construction technical code, which provides the reference peak ground acceleration on the basis of geographical position of the construction site. In particular, the peak ground acceleration corresponding to the selected geographical positions, Rome and Potenza, are equal to 0.11g (medium-low seismicity) and 0.20g (medium-high seismicity), respectively. The assumed foundation soil is type C. The main parameters for the calculation of the seismic action at Life Safety limit state are summarized in Table 7, while the assumed elastic acceleration spectra are shown in Figure 8.

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 2400 mm x 2700 mm. 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 lateral resistance of walls has been evaluated through the procedure explained in Section 3.2.The different design hypotheses assumed for the three selected wall configurations, together with the main design results are summarised in Table 8.

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 S350GD+Z (characteristic yield strength f_y=350 MPa and characteristic ultimate strength f_u=420 MPa) steel grade 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 Equation (21) 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 by considering the behaviour factor given by AISI S213 for "Conventional construction" in Canada (q=2.5) and by 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 (Rome), while the WHD corresponds to a three-storeys building in a medium-high seismicity level zone (Potenza). 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 (Ω_i) prescribed by EN 1998-1. 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 Equation (13). 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 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 (f_y=235 MPa and f_u=360 MPa) steel grade, 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. Figure 9 shows the diagonal connection details designed for each selected wall. The capacity design rules for all the non-dissipative elements (studs, tracks, connections and anchorages) have been applied by considering the Equation (20). This way 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 (Ω_i) at each storey. For connections, also Equation (21) has been satisfied. All geometrical dimensions and materials of the wall components designed for the investigated walls are presented in companion paper [22].

6 CONCLUSIONS

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 in terms of behaviour factor and capacity design rules for strap-braced CFS structures are proposed. Following the proposed design criteria, a case study consisting in the design of three residential building under different design approach (elastic and dissipative) and seismic scenarios (medium-low and medium high seismicity) has been developed. The designed structures are the basis for the definition of the extended experimental campaign presented in the companion paper.

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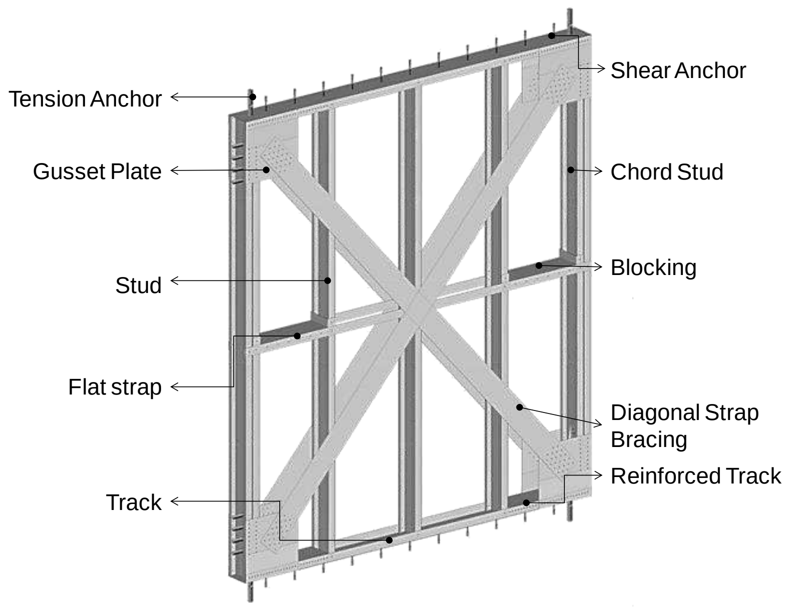

Figure 1: Typical diagonal strap-braced wall.

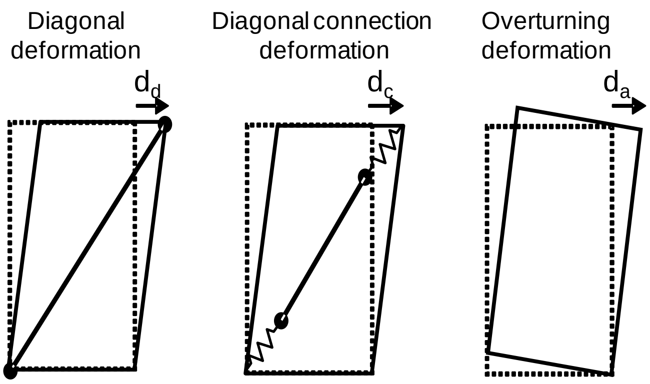

Figure 2: Contributions to deformations of a diagonal braced wall.

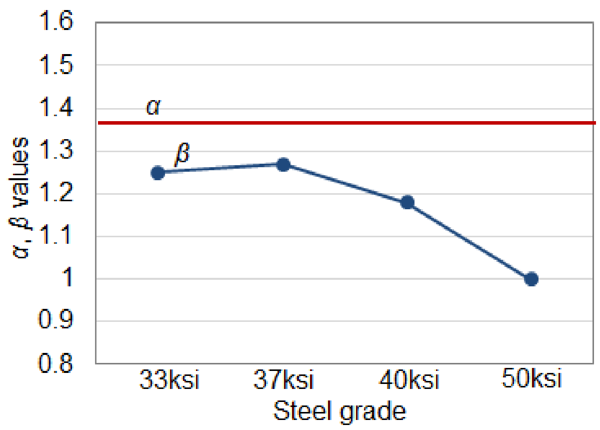

Figure 3: Comparison between α and β coefficient

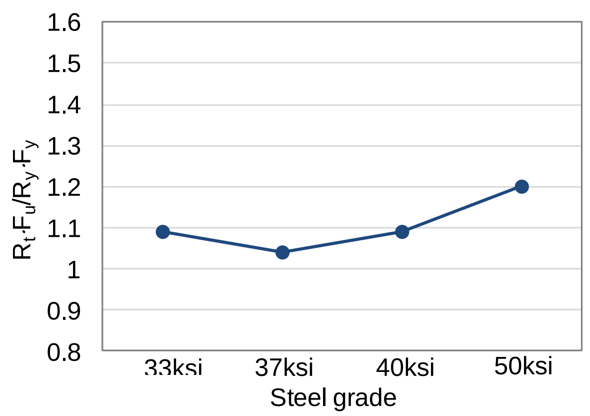

Figure 4: $R_t{\cdot}F_u/\ R_y{\cdot}F_y$ ratio values

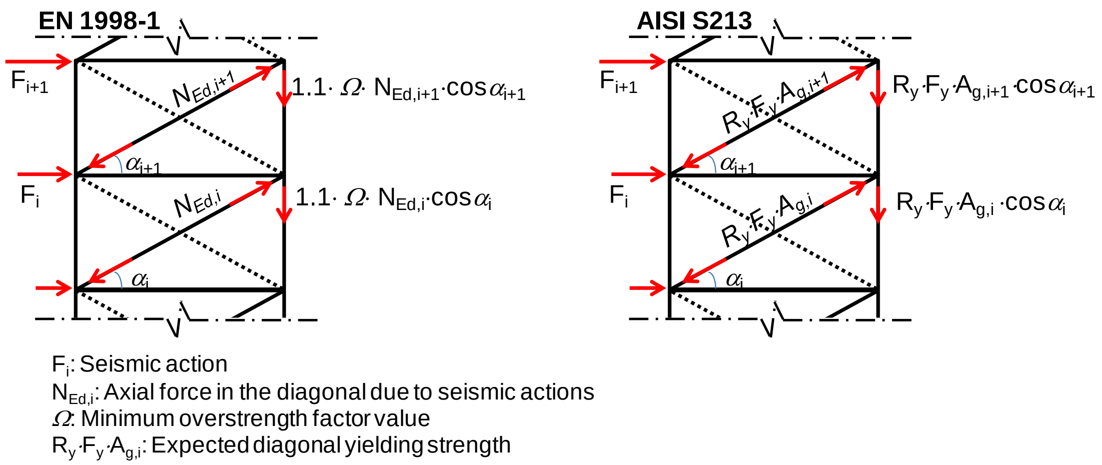

Figure 5: Schemes for only seismic forces according to capacity design rules of the two examined codes

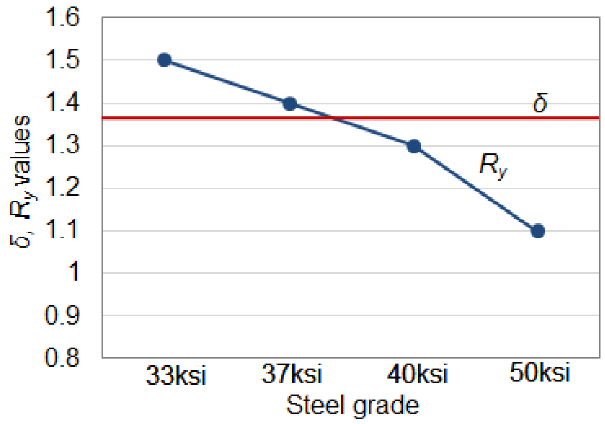

Figure 6: Comparison between δ and R_y coefficient

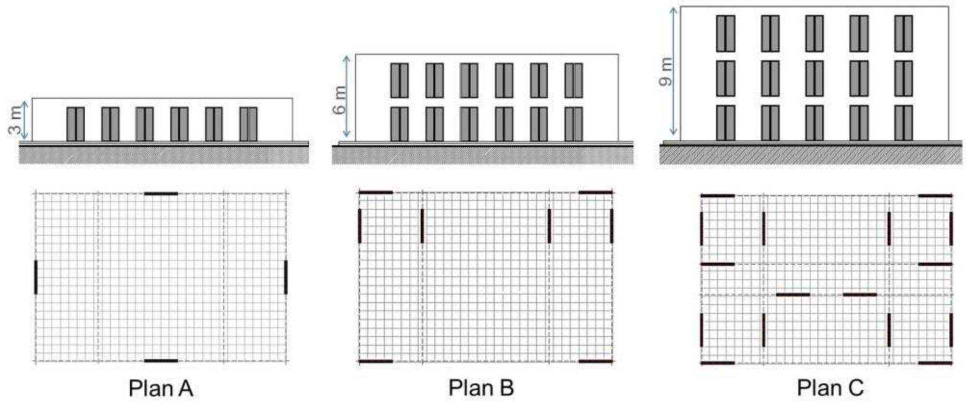

Figure 7: Schematic views of case study buildings and assumed plan distribution of walls.

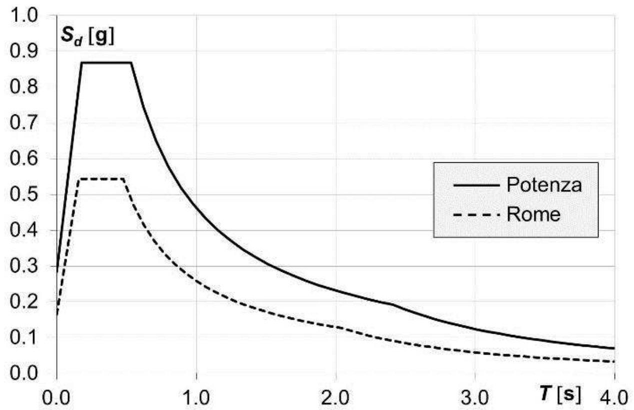

Figure 8: Elastic acceleration spectra.

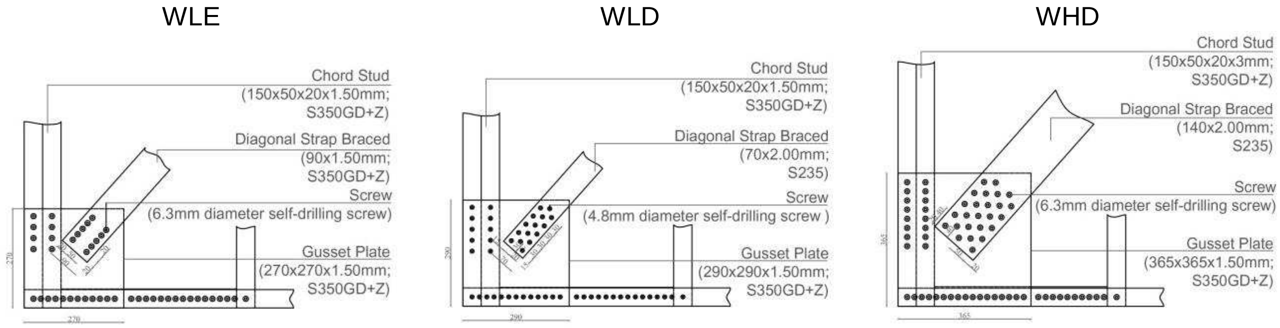

Figure 9: Diagonal connection details.

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Table 1. Literature studies matrix with indication of the test typology and specimens.

Author	Bracing system	Specimen number	Wall geometry	Tested specimens									Hold-down type
				Studs			Flat strap bracing			Frame-to-strap connection	Gypsum boards	Gusset plates	
			length x height [mm]	interior studs web depth/ thickness [mm]	yield stress [MPa]	chord studs no. x web depth/ thickness [mm]	(no. of tests) bracing side	width x thickness [mm]	yield stress [MPa]	type and diameter [mm]	thickness [mm]	dimensions [mm]	
Adham et al. (1990) ^{NA}	XB + GWB	5^{C}	2440 x 2440	C92/0.84	228	2xC92/0.84	(1) one (4) two	50.8x0.84 50.8x0.84 76.2x1.09 76.2x1.37	228	screws: d =4.2	15.9	305x305x0.84	steel angles (LC)
Serrette (1994) ^{NA}	XB	3^{M}	2438 x 2438	C152/0.84	228	2xC89/0.84	one	50.8x0.84	228	screws: d =4.2	no		hold-down^{2} (LC)
	XB + GWB	4^{C}									12.7		
Serrette and Ogunfunmi (1996) ^{NA}	XB	3^{M}	2440 x 2440	C152/0.84	228	2xC152/0.84	one	50.8x0.84	228	screws: d=4.2	no	254x254x0.84	steel angles (LC)
	XB + GWB	5^{M}					(4) one (1) two				12.5		
Serrette (1997) ^{NA}	XB	4^{M} + 4^{C}	1219 x 2438	C89/0.84 C89/1.09	228	2xC89/0.84 2xC89/1.09	one	114.3x0.84 190.5x0.84	228	screws: d=4.2	no		hold-down^{2} (LC)
Fulop and Dubina (2004) ^{E}	XB	1^{M} + 2^{C}	3600 x 2440	C150/1.50		2xC150/1.50	two	110x1.50		screws: d =4.8; d =6.3	no	no	no
Tian et al. (2004)^{E}	XB	5^{M}	1250 x 2450	C90/1.20	350	no	(1) one (4) two	60x1.00 60x1.20	280	rivets: d =5.0	no	no	steel angles (LC)
Al-Karat and Rogers (2006) ^{NA}	XB	9^{M} + 7^{C}	2440 x 2440	C92/1.22 C152/1.22 C152/1.22	230	2xC92/1.22 2xC152/1.52 2xC152/1.91	two	58.4x1.22 101x1.52 152x1.91	230	screws: d =4.8 weld weld	no	no 250x250x1.52 300x300x1.91	steel angles (LC, UC) or flat plates (LC, UC)
Casafont et al. (2007)^{E}	XB	2^{C}	1079 x 644	C102/2.00 + U108/2.00	350	no	two	65x0.80	250	screws: d =6.3	no	210x140x1.50	hold-down^{1} (LC)
Moghini and Ronagh (2009)^{A}	XB	15^{C}	2440 x 2440	C90/0.55	550	2xC90/0.55	(12) one (3) two	30x0.84	250 300	screws: d =4.8	no	no	steel angles (LC, UC)
	XB + GWB	3^{C}				no	one				10		
Velchev et al. (2010) ^{NA}	XB	27^{M} +17^{C}	2440 x 2440 1220 x 2440 610 x 2440	C92/1.09 C152/1.09 C152/1.09	230 230 230	2xC92/1.09 2xC152/1.37 2xC152/1.73	(1) one (26) two (1) one (16) two	63.5x1.09 69.9x1.37 101.6x1.73	230 340 340	screws: d =4.8 weld	no	no 152x152x1.37 203x203x1.73	hold-down^{2} (LC, UC) or U-shaped hold-down (LC, UC)

^{A} Australia, ^{E} Europe, ^{NA} North America;

XB: strap-braced stud walls, XB + GWB: strap-braced stud walls finished with gypsum sheathing boards;

^{M} monotonic test, ^{C} cyclic test;

C: C-section profile; U: U-section profile; 2xC= back-to-back double C-section profiles;

"no" stands for "not present";

LC: lower corners, UC: upper corners;

hold-down^{1}: special device of reinforced steel angles designed by the Authors; hold-down^{2}: devices provided by Simpson Strong Tie.

Table 2. Literature studies matrix with investigated parameters

Author	**Bracing system**	**Investigated parameters**										
		Studs				**Flat strap bracing**			**Frame-to-strap connection**	**Gypsum boards**	**Aspect ratio**	**Loading type**
		dimensions	steel grade	chord studs	bracing side	width	thickness	steel grade				
Adham et al. (1990) [NA]	XB + GWB	-	-	-	√	√	√	-	-	-	-	-
Serrette (1994) [NA]	XB	-	-	-	-	-	-	-	-	no	-	√
	XB + GWB									-		
Serrette and Ogunfunmi (1996) [NA]	XB	-	-	-	-	-	-	-	-	no	-	-
	XB + GWB				√					-		
Serrette (1997) [NA]	XB	√	-	√	-	√	-	-	-	no	-	√
Fulop and Dubina (2004) [E]	XB	-	-	-	-	-	-	-	-	no	-	√
Tian et al. (2004)[E]	XB	-	-	no	√	-	√	-	-	no	-	-
Al-Karat and Rogers (2006) [NA]	XB	√	-	-	-	√	√	-	√	no	-	√
Casafont et al. (2007)[E]	XB	-	-	-	-	-	-	-	√	no	-	-
Moghini and Ronagh (2009)[A]	XB	-	-	√	√	-	-	√	-	no	-	-
	XB + GWB			no	-					√		
Velchev et al. (2010) [NA]	XB	√	-	-	√	√	√	√	√	no	√	√

[A] Australia, [E] Europe, [NA] North America;
XB: strap-braced stud walls, XB + GWB: strap-braced stud walls finished with gypsum sheathing boards;
"no" stands for "non present".

Table 3: 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

Table 4: $R_t \cdot F_u / R_y \cdot F_y$ ratio values for steel grades provided by AISI S213

Steel grade (f_y in MPa)	$R_t \cdot F_u / R_y \cdot F_y$
33 ksi (230)	1.09
37 ksi (255)	1.04
40 ksi (275)	1.09
50 ksi (340)	1.20

Table 5: R_y and R_t values for steel grades provided by AISI S213

Steel grade AISI S213 (f_y in MPa)	R_y	R_t
33 ksi (230)	1.5	1.2
37 ksi (255)	1.4	1.1
40 ksi (275)	1.3	1.1
50 ksi (340)	1.1	1.1

Table 6: Dead loads.

Structural element	min	max
floors (kN/m^2)	0.60	1.50

walls (kN/m^2)	0.30	1.00

Table 7: 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: 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.

Table 8: Design hypotheses and results for selected wall configurations.

Wall configuration	WLE	WLD	WHD
Location	Rome	Rome	Potenza
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