ASSOCIAZIONE NAZIONALE ITALIANA DI INGEGNERIA SISMICA ANIDIS
Italian National Association of Earthquake Engineering

XVIII CONVEGNO ANIDIS
L'ingegneria sismica in Italia
ASCOLI PICENO 2019
15-19 Settembre

Cold-Formed Steel lateral force resisting systems: proposal of seismic design criteria for 2nd generation of Eurocode 8

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Keywords: Eurocode 8, seismic design criteria, lightweight steel structures, lateral force resisting systems, cold-formed steel

ABSTRACT
The current edition of Eurocode 8, which provides the rules for the design of the building structures under the earthquake actions, does not cover the Lateral Force Resisting Systems (LFRS) common to Lightweight Steel (LWS) buildings made with Cold-Formed Steel (CFS) profiles. As part of the revision process of Eurocode 8, efforts are being carried out at University of Naples "Federico II" to make possible the availability of rules for the seismic design of LWS building to designers in the 2nd generation of Eurocode 8, which were developed and validated during the extensive research in the past decade. In particular, different types of LFRS would be listed in the second generation of Eurocode 8: CFS strap braced walls and CFS shear walls with steel sheet, wood or gypsum sheathing. This paper provides the background information on the research works and the reference design standards, already being used in some parts of the world, which formed the basis of design criteria for these LFRS systems. The design criteria for the LFRS common to LWS buildings would include rules necessary for ensuring the dissipative behaviour, appropriate values of the behaviour factor, guidelines to predict the design strength, geometrical and mechanical limitations.

1 INTRODUCTION

Lightweight Steel (LWS) buildings owing to their ability to meet growing market demands related to economic efficiency and ecological performance are becoming a popular alternative for low to mid rise constructions. A typical LWS buildings is fabricated with the Cold-formed steel (CFS) structural elements and sheathed with different types of panels to form an envelope. Their diffusion in the construction market of the earthquake prone regions of the world demands a more robust seismic design guidelines.

Eurocodes are European construction standards for structural and geotechnical design and the currently in use editions were published about ten years ago. However, some of their parts have become obsolete if analysed in the light of the intense scientific research and technological progress made over the last decade. Moreover, the current edition of Eurocode 8: EN 1998-l (CEN, 2004b), which provides the rules for the design of structures for earthquake resistance, does not cover LFRS's common to Lightweight Steel (LWS) buildings. These discrepancies have led to the process of revision of the Eurocode 8. In particular, the authors have been involved actively in this revision process and focused their efforts on devising a proposal of the rules for seismic design of LWS buildings in 2nd generation of Eurocode 8. This paper provides the background information on the research works and the reference design standards, already being used in some parts of the world, which formed the basis of design criteria for the proposal. The newer edition of EN 1998-1 (CEN, 2004b) will provide rules for LWS buildings laterally braced with four different types of LFRS's: CFS strap braced walls (Figure 1-1); CFS shear walls (Figure 1-2) with steel sheet sheathing, or wood sheathing, or gypsum sheathing. The paper is organized to provide a brief description of LFRS functioning followed by a short overview of the design framework of the newer edition of EN 1998-1, and the design rules for each system along with the relevant background information.

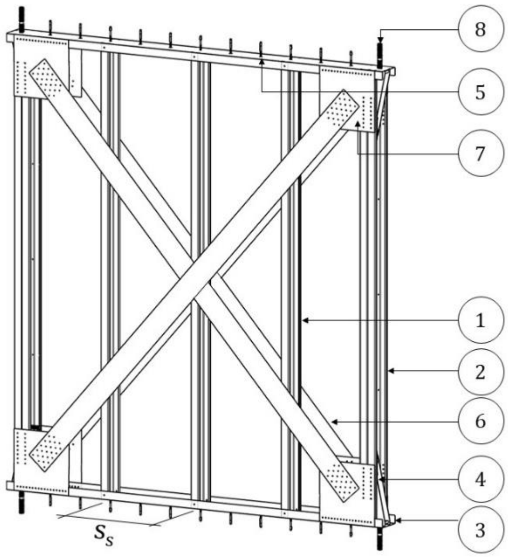

Figure 1-1 CFS strap braced wall: 1. stud; 2. chord stud; 3. track; 4. hold-down; 5. shear anchorage; 6. steel strap brace; 7. connection of strap brace; 8: tension anchorage; s_s: stud spacing.

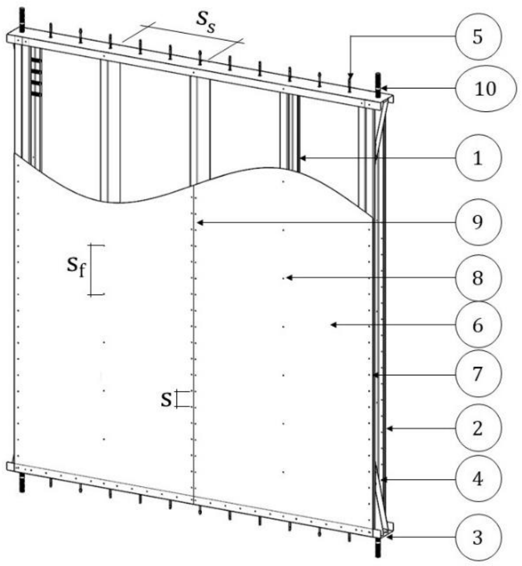

Figure 1-2 CFS shear wall: 1. stud; 2. chord stud; 3. track; 4. hold-down; 5. shear anchorage; 6. steel sheet or wood or gypsum sheathing; 7. screw at panel edge; 8: screw in the panel field; 9. sheathing joint; 10. tension anchorage; s_s: stud spacing; s_f: screw spacing in the panel field; s: screw spacing at panel edge.

2 SYSTEM CONCEPT

A LWS building is made of studs, i.e. vertical load bearing equally spaced members screwed at each end to tracks, which restrain the studs at their ends. The floors of the building are composed of equally spaced joists, on top of which the sheathing materials are connected by means of screws or the floor could be made by a composite steel concrete slab. The individual components of building walls and floors can be designed using the other relevant part of the Eurocodes 3, 4 and 5 (CEN, 2004a, 2005a, 2005b) against the gravity loads.

To resist earthquake loads, two different types of LFRSs are used in CFS constructions, strap braced walls and shear walls, also called sheathing-braced walls. In case of strap braced walls, lateral resistance is usually provided by thin steel straps acting as braces in an X configuration. While for shear walls steel sheets or panels generally provide the lateral bracing effect to withstand horizontal loads. Strap braced walls (Figure 1-1) is a solution designed to resist in-plane lateral forces mainly with tension-only steel straps applied diagonally. The mechanism allowing the dissipative behaviour in the walls is tension yielding of the strap braces. Contrarily, in shear walls (Figure 1-2) with steel sheets, or wood panels, or gypsum panels, the dissipative behaviour of walls is due to the member-to-sheathing connections (termed also as sheathing connections) and steel sheets in case of shear walls with steel sheets, and sheathing connections only in case of shear walls (Figure 1-2) with panels. Obviously, brittle failure mechanisms for all LFRSs should be avoided. In particular, the possible brittle failure mechanisms can be grouped as follows:

- strap brace connections in case of strap braced walls only;
- panels in case of shear walls with panels only;
- chord studs or other compressed vertical boundary elements at the ends of the wall;
- tracks;
- hold-downs, tension anchorages and their connections or other tensioned vertical boundary elements at the ends of the wall;
- shear anchorages;
- all other components and connections in the wall;

3 DESIGN FRAMEWORK OF NEWER EDITION OF EN 1998-1

The design of a building, equipped with any type of LFRS, according to the newer edition of EN 1998-1 will result in a structure with three different ductility levels:

- DC1: Low-dissipative structural behaviour;
- DC2 : Medium Dissipative structural behaviour.
- DC3 : High Dissipative structural behaviour.

The design of a structure with a particular ductility class (DC) can not be made above certain levels of seismic action for DC1 and DC2 Class structures, while there is no limit on the intensity of seismic action for DC3 Class structures.

In DC2 and DC3 Class structures, the capability of parts of the structure (dissipative components) to resist seismic actions through inelastic behaviour is taken into account. To ensure the ability to dissipate energy in plastic mechanisms for DC2 and DC3 Class structures, specific requirements are defined for the design of dissipative components and for the protection of non-dissipative components, which are also termed as capacity design rules. The design requirements for the dissipative components of CFS LFRSs belonging to both DC2 and DC3 Class structures are same and given in Section 4, whereas the capacity design rules DC2 and DC3 CFS LFRS Class structures are different, as discussed in Section 5 and Section 6.

In contrast to the DC2 and DC3 Class structures, the DC1 Class structures are not required to follow capacity design rules. This will results in the limited ductility capacity in DC1 Class structures and hence the Code proposes a lower value of the behaviour factor (q) equal to 1.5. Higher values of the q are proposed for DC2 and DC3 Class structure owing to their high energy dissipative structural behaviour as shown in Section 5 and Section 6. The values of the behaviour factors for DC2 and DC3 Class structures are derived from the studies conducted following the FEMA P695 methodology (FEMA, 2009). FEMA P695 methodology follows an iterative approach by assuming an initial value of q for the design of set of archetypes, whose performance is quantified through nonlinear static analysis and nonlinear dynamic collapse simulations under a suite of earthquake records and their safety is evaluated in terms of acceptable collapse margin ratios. The behaviour factor for CFS strap braced walls and CFS shear walls with gypsum sheathing were evaluated by authors in their past studies (Fiorino at al., 2017; Shakeel et al., 2019) using a set of fourteen, 1 to 4 storeys residential and office type, archetypes designed to withstand low to high intensity earthquake loads. A similar study on CFS shear walls with wood sheathing is currently being conducted by authors to support the assumption of a behaviour factor for this LFRS. For CFS shear walls with steel sheet sheathing, the study conducted by Shamim et al. (Shamim & Rogers, 2015) for the archetypes designed according to Canadian approach (*National Building Code of Canada*, 2005) is considered as reference.

Apart from providing special requirements for DC2 and DC3 Class structures, the newer edition of EN 1998-1 will also provide some general rules that will govern the proper functioning of the LFRS and will list all the important design considerations for each components of the LFRS. Moreover, these rules also impose the limits on the aspect ratio (height-to-length ratio) of the wall, which is fixed equal to 2.0 for all type of LFRSs. To have a sufficient deformation capacity of connections in the walls, the new version of EN 1998-1 would require the design shear resistance of the screws to be greater than 1.2 times the design bearing resistance of the steel structural member, or the design embedment resistance of wood or gypsum panels (in case of shear walls with panels), or the design net area resistance of the strap brace (in case of strap brace walls). This rule has been derived from the already existing guidelines in EN 1993-1-3 (CEN, 2006) for the shear design of connections made with screws.

4 DESIGN REQUIREMENTS FOR DISSIPATIVE COMPONENTS

In addition to rules explained in Section 3, special rules are also given for the dissipative component of each type of LFRS.

For strap braced walls, the yield resistance ($N_{\mathrm{pl,Rd}}$) of the gross cross-section of the strap braces should be greater than the design value of the axial force in the strap brace in the seismic design situation and the design net area resistance ($N_{\mathrm{u,Rd}}$) of the strap brace. This requirement ensures the formation of plastic mechanism in steel straps before the net section failure happens in strap connection to the wall frame. The values of $N_{\mathrm{pl,Rd}}$

and $N_{u,Rd}$ can be obtained from other relevant parts of Eurocode 3 (CEN, 2006).

In case of shear walls with steel sheet sheathing, the in-plane lateral resistance ($R_{c,Rd}$) corresponding to the strength of the sheathing connection within the effective sheathing strip should be greater than the design value of the lateral force acting on the shear wall in the seismic design situation, but it should be less than, the design yielding resistance of the effective sheathing strip ($R_{y,Rd}$). Fulfilling this requirement will allow the failure of the shear wall governed by the resistance of sheathing connections. $R_{c,Rd}$ and $R_{y,Rd}$ are evaluated based on the effective strip method (ESM) proposed in (Yanagi & Yu, 2014). ESM was calibrated based on the large amount of steel sheathed shear walls tested in USA and Canada over recent years. In the original proposed ESM (Yanagi & Yu, 2014), $R_{c,Rd}$ is calculated according to Equation (2), which relies on the formulation of North American specification for the design of CFS structural members AISI S100 (AISI, 2016) for the calculation of single sheathing connection bearing strength ($F_{b,Rd}$).

$$R_{c,Rd} = 1.33 \text{ n } F_{b,Rd} \cos(\arctan\left(\frac{h}{w}\right)) \qquad (2)$$

where $F_{b,Rd}$ is the bearing resistance of the sheathing connection, *n* is the number of sheathing connections in the effective with, *h* is height of wall and *w* is the length of wall. AISI S100 provides a different relationships for $F_{b,Rd}$ than the one provided in Eurocode 3 (CEN, 2006). Authors in their recent study (Campiche et al., 2019) checked the validity of the ESM for the design according to Eurocodes and found that the $R_{c,Rd}$ calculated according to Eurocode 3 (CEN, 2006) formulations for $F_{b,Rd}$ would give an underestimation of wall strength by 50%. As a result, in the new version of EN 1998-1, formulations to compute $F_{b,Rd}$ are provided based on the equations given in AISI S100 (AISI, 2016). Additionally, the use of the ESM in the new version of EN 1998-1 is only limited to walls with aspect ratio between 1.0 and 2.0 and a maximum steel frame thickness of 1.35 mm. This lower limit on aspect ratio is proposed based on the geometry of the walls used to calibrate the ESM (Yanagi & Yu, 2014), where walls always had an aspect ratio greater than 1.0.

For CFS shear walls with wood or gypsum sheathing, the in-plane lateral resistance ($R_{c,Rd}$) corresponding to the strength of the sheathing connection should be greater than the design value of the lateral force acting on the shear wall in the seismic design situation. In addition to the rules explained above, the new version of EN 1998-1 will also provide geometrical and mechanical requirements for the components and parts of the shear walls, which must also be fulfilled to achieve desired energy dissipation response in the walls. The requirements are defined based on the already existing geometrical and mechanical limitations on the permitted wall configurations given in AISI S400 (AISI, 2015). In particular, these requirements provide limits on thickness, dimensions and strength of the panels; thicknesses and strength of the frame elements; spacing of the sheathing connections; screw diameters; edge distances of the sheathing connections; and spacing of the studs.

5 OVERSTRENGTH REQUIREMENTS FOR DC2 CLASS STRUCTURES

The overstrength requirements for the DC2 Class structures ensure the formation of desired energy dissipating mechanism in the dissipative components of LFRS. In DC2 Class structures, the overstrength factor (Ωov) is used for the application of the hierarchy of resistances and it accounts for both the overdesign of the dissipative zones and the increase of seismic induced effects in the non-dissipative elements. These overstrength factors are applied using Equation (1) on the non-dissipative components of the LFRS of a DC2 Class structure to verify their strength and stability against the most unfavourable combination of the axial force N_{Ed}, bending moments M_{Ed} and shear force V_{Ed}.

$$\begin{aligned} N_{Ed} &= N_{Ed,G} + \Omega_{ov} N_{Ed,E} \\ M_{Ed} &= M_{Ed,G} + \Omega_{ov} M_{Ed,E} \\ V_{Ed} &= V_{Ed,G} + \Omega_{ov} V_{Ed,E} \end{aligned} \qquad (1)$$

where: $N_{Ed,G}$, $M_{Ed,G}$ and $V_{Ed,G}$ are the axial force, bending moment and shear force in the non-dissipative member due to the non-seismic actions included in the combination of actions for the seismic design situation; and $N_{Ed,E}$, $M_{Ed,E}$ and $V_{Ed,E}$ are the axial force, bending moment and shear force in the non-dissipative member due to the design seismic action. The overstrength factor for different types of LFRSs is listed in Table 1 along with their behaviour factor.

Table 1 Behaviour and overstrength factors for DC2 Class structures

LFRS	q	Ω_{ov}
Strap braced walls	2.0	1.5
Shear walls with steel sheet sheathing;	2.0	1.5
Shear walls with wood sheathing	2.0	2.0
Shear walls with gypsum sheathing	1.7	1.3

6 OVERSTRENGTH REQUIREMENTS FOR DC3 CLASS STRUCTURES

The designer will need to follow different overstrength requirements to design a DC3 Class structure than the requirements for DC2 class structure according to new version of EN 1998-1.

For strap braced walls, the brittle components listed in Section 2 should be designed with overstrength computed according to (4).

$$R_d \geq F_{Ed,G} + \gamma_{ov}\, \gamma_{sh}\, R_{fy} \tag{4}$$

where, R_d is the resistance of the component; $F_{Ed,G}$ is the action in the component or connection in the wall due to the non-seismic actions included in the combination of actions for the seismic design situation, γ_{ov} is the overstrength factor accounting for variability of the steel yield strength in the dissipative zones and ranges from 1.20 to 1.45 for lower to higher steel grades, γ_{sh} is the overstrength factor accounting for the hardening in the dissipative zones and is equal to 1.1, and R_{fy} is the plastic resistance of the gross cross-section of the strap braces based on the design yield stress of the material obtained from Eurocode 3 (CEN, 2005a).

For shear walls with steel sheet sheathing, a different Equation (5) is proposed, which ensure the overstrength in their britlle components.

$$R_d \geq F_{Ed,G} + \gamma_{ov}\, R_{c,Rd} \tag{5}$$

where γ_{ov} is the overstrength factor equal to 1.40, and $R_{c,Rd}$ is the design in-plane lateral resistance of the shear wall.

For CFS shear walls with with gypsum or wood sheathing, Equation (6) is used to provide overstrength in their britlle components.

$$R_d \geq F_{Ed,G} + \gamma_{ov}\, R_{c,k} \tag{6}$$

where γ_{ov} is overstrength factor equal to 1.50, and $R_{c,k}$ is the characteristic in-plane lateral resistance of the shear wall. Table 2 summarizes the values of γ_{ov} used for the overstrength of DC3 Class structures along with the values of the behaviour factor for different LFRSs.

Table 2 Behaviour and overstrength factors for DC3 Class structures

LFRS	q	γ_{ov}
Strap braced walls	2.5	1.20 to 1.45
Shear walls with steel sheet sheathing	2.5	1.40
Shear walls with wood sheathing	2.5	1.50
Shear walls with gypsum sheathing	2.0	1.50

7 CONCLUSIONS

This paper presents the design criteria for cold-formed steel lateral force resisting systems, starting from the past research carried out on these systems and the existing design standards currently in practice in some parts of the world. The design criteria would be the part of next generation of Eurocodes, which are currently being prepared. The design criteria covers four different types of LFRS's: CFS strap braced walls, CFS shear walls with steel sheet or wood or gypsum sheathing, which can be used to achieve three levels of ductility classes in a building. The three levels of DC's differ with each other in terms of their energy dissipating capacities. Special capacity design rules and limitations on the geometrical and mechanical properties are required to be followed for DC2 and DC3 Class structures, while DC1 Class structure requires no specific capacity design rules and limitations. Different values of the behaviour factors are also proposed for DC2 and DC3 Class structures, which are based on the studies conducted following the methodology of FEMA P695 on range of building archetypes equipped with a particular LFRS. One of the major task for future studies on this topic to further validate the design criteria through series numerical and experimental researches, hence making it more clear to practitioners and engineers the differences between among different types of ductility class structures. The design criteria currently also lacks the limitations on the heights of building and its relation to different ductility classes, which would also be explored in future. Once approved by the relevant authorities, there would also be need to have detail design examples for engineers to better understand the design guidelines, as the use of these systems under seismic actions is currently quite rare in the European continent.

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