PROGETTO DI STRUTTURE SISMORESISTENTI IN PROFILI FORMATI A FREDDO IN ACCIAIO IN ACCORDO ALLA SECONDA GENERAZIONE DI EUROCODICI

DESIGN OF SEISMIC-RESISTING COLD-FORMED STEEL STRUCTURES ACCORDING TO 2ND GENERATION OF EUROCODES

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ABSTRACT

Nowadays the design of seismic-resisting cold-formed steel (CFS) structures are not explicitly covered by European structural standards, even if extensive studies has been carried out in the past decades on this topic and a solid background has long been available. On this basis, within the ongoing revision process European structural standards, rules for the seismic design of CFS buildings based on available background studies have been incorporated in 2nd generation of Eurocode 8. Different types of Lateral Force Resisting System (LFRS) are covered, namely CFS strap-braced walls and CFS shear walls with steel sheets, wood, or gypsum sheathing. Design rules include a set of provisions for achieving the dissipative behaviour of LFRS, together with behaviour factor values and geometrical and mechanical limitations. A summary of design rules for CFS structures incorporated in 2nd generation of Eurocode 8 and the reference to main background documents are presented in the manuscript.

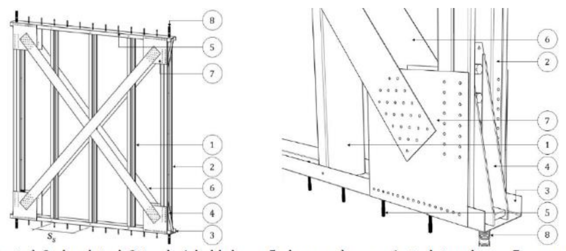
SOMMARIO

La progettazione di strutture sismoresistenti in profili formati a freddo in acciaio (CFS) non è esplicitamente coperta dale normative strutturali europee, anche se negli ultimi decenni numerosi e approfonditi sono stati gli studi condotti su questo argomento, che hanno da tempo reso disponibile solido background. Partendo da questo presupposto, nell'ambito del processo di revisione in corso per le norme strutturali europee, le regole per la progettazione sismica degli edifici in CFS sono state incorporate nella Seconda generazione dell'Eurocodice 8. In particolare, la nuova norma include le pareti controventate con piatti sottili di acciaio e le pareti di taglio controventate con

lamiere sottili di acciaio o con pannelli di legno o cartongesso. Le regole di progettazione includono prescrizioni per il comportamento dissipativo, valori dei fattori di comportamento e limitazioni geometriche e meccaniche. La memoria presenta una sintesi delle regole di progettazione per le strutture sismoresistenti in CFS presenti nella Seconda generazione dell'Eurocodice 8 e il riferimento ai principali documenti di background.

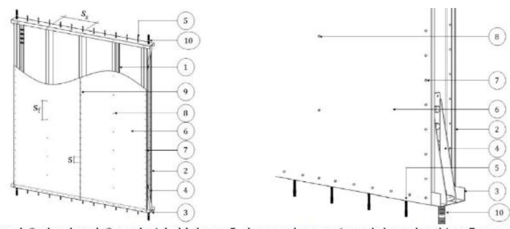
1 INTRODUCTION

Nowadays the design of seismic-resisting cold-formed steel (CFS) structures are not explicitly covered by European structural standards, even if extensive studies has been carried out in the past decades on this topic and a solid background has long been available. It is worth mentioning that design rules for CFS buildings are already available in North American Standard for seismic design of cold-formed steel structures (AISI S400 [1]), which is currently adopted in USA, Canada, and Mexico. AISI S400 follows the capacity design approach: the standard gives provision for the selection of energy dissipation mechanisms of dissipative components, provides overstrength requirements concerning non-dissipative components, and gives values of response modification factors accounting for system inherent overstrength and ductility.
Starting from the provisions given in AISI S400 and on the basis of available relevant literature and, this paper presents novel seismic design rules for LWS buildings, which are currently under consideration by CEN for inclusion in the upcoming edition of Eurocode 8.
Different types of Lateral Force Resisting System (LFRS) are covered, namely CFS strap-braced walls (Figure 1) and CFS shear walls (Figure 2) with steel sheets, wood, or gypsum sheathing. Design rules include a set of provisions for achieving the dissipative behaviour of LFRS, together with behaviour factor values and geometrical and mechanical limitations.



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.

Fig. 1. CFS strap-braced walls



1. stud; 2. chord stud; 3. track; 4. hold-down; 5. shear anchorage; 6. steel sheet 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.

Fig. 2. CFS shear walls

2 FRAMEWORK OF THE 2ND GENERATION OF EN 1998

The 2nd generation of Eurocode 8 would allow achieving three levels of ductility in building structures: (1) DC1 - Low Dissipative structural behaviour; (2) DC2 - Medium Dissipative structural behaviour; (3) DC3 - High Dissipative structural behaviour.

Usually, a structure cannot be designed beyond a certain limit of seismic load in the case of DC1 and DC2 Class structures while in the case of DC3 Class structures, there is no limit on the seismic action. Limits on the seismic action are defined in terms of threshold values of S_α, where S_α is maximum response spectral acceleration (5% damping) corresponding to the constant acceleration range of the elastic response spectrum. In the case of CFS buildings, the threshold value of S_α for DC1 and DC2 Class structures is 5.0 and 7.5 m/sec^2, respectively.

In DC2 and DC3 Class structures, the capability of parts of the LFRS (dissipative components) to resist the seismic actions through plastic behaviour in dissipative components is taken into consideration during the design, which would be a capacity design process. In contrast to the DC2 and DC3 Class structures, the DC1 Class structures would not be required to follow any specific design and overstrength requirements. The design of individual components of the DC1 Class structure can be carried out according to Eurocode 3 – Part 1-3 [2]. Therefore, DC1 Class structures would have a limited ductility capacity, and a lower value of behaviour factor (q) equal to 1.5 is proposed for them. Meanwhile, in the case of DC2 and DC3 Class structures, the ability of structures to dissipate energy through their plastic behaviour is accounted for, therefore higher values of q are proposed for them.

The values of the behaviour factor for DC2 and DC3 Class structures are derived from the studies [3,4,5,6] conducted following the FEMA P695 methodology [7], i.e., an iterative approach to evaluate the behaviour factor for any LFRS, based on nonlinear static analysis and incremental dynamic analysis under a suite of earthquake records. The behaviour factor for CFS strap-braced walls and CFS shear walls with gypsum or wood sheathing were evaluated in [3,4,5]. For steel sheathed shear walls, the study following the FEMA P695 methodology was conducted by Shamim et al. [6] and Kechidi et al. [8].

Apart from providing special design rules for DC2 and DC3 Class structures, the 2nd Edition of Eurocode 8 will also provide some general rules common to all DC1, DC2 and DC3 Class lightweight steel systems. These general rules are provided for the proper functioning of LFRS and ensure that all the important design considerations are not overlooked during the design process. These general rules include the limitation on the aspect ratio (height-to-length ratio) of the walls, which is fixed equal to 2.0 for all types of LFRS's. This limit reflects the tendency of walls with a greater than 2.0 aspect ratio to develop the bending moments in boundary frame elements (studs and tracks). In a such case, the design of boundary elements against the bending actions would be required. In addition, the lateral response of walls with a greater than 2.0 aspect ratio is characterized by excessive deformability. Nonetheless, it would not be permitted to exceed the aspect ratio of 2.0 in both strap-braced and shear walls.

To have a sufficient deformation capacity of connections in the walls, the 2nd Edition of Eurocode 8 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 prescriptions in Eurocode 3 – Part 1-3 [2] for the shear design of connections made with screws. Additionally, pull-out resistance of screws cannot be used to resist seismic forces.

3 DESIGN REQUIREMENTS FOR DISSIPATIVE COMPONENTS

The 2nd Edition of Eurocode 8 would provide specific rules to calculate the design strength of the LFRS in the case of DC2 and DC3 Class structures. In addition to these design rules, the Code will also provide geometrical and mechanical requirements for the components and parts of the shear walls, which must also be fulfilled to achieve the 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 [1]. 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. More details on these requirements are provided in Figure 3. There will be no limitation for components of strap-braced walls.

Component	Limit on thickness/diameter	limit on yielding strength	Spacing limit/ special requirement
CFS shear walls with steel sheathing			
Steel sheet	$0.4\ mm \leq t_p \leq 0.9\ mm$	$f_y \leq 350\ MPa$	$w \geq 300\ mm$
Studs and track	$t_f \geq 0.9\ mm$	$f_y = 220\ MPa, t_f < 1.4\ mm$ $f_y = 350\ MPa, t_f \geq 1.4\ mm$	$s_s \leq 600\ mm$
Screws	$4.2 \leq d \leq 4.8\ mm$	Not given	$50\ mm \leq s_{sc} \leq 150mm$ $s_{edge} \geq 13\ mm$
CFS shear walls with wood sheathing			
Wood panel	OSB ($9\ mm \leq t_p \leq 11\ mm$) Plywood ($9.5\ mm \leq t_p \leq 12.5\ mm$	Not given	$w \geq 300\ mm$
Studs and track	$t_f \geq 1.1\ mm$	$f_y = 220\ MPa, t_f < 1.4\ mm$ $f_y = 350\ MPa, t_f \geq 1.4\ mm$	$s_s \leq 600\ mm$
Screws	For $t_f < 1.4\ mm$ $d = 4.2\ mm, d_h \geq 7.2\ mm$ For $t_f \geq 1.4\ mm$ $d = 4.8\ mm, d_h \geq 8.5\ mm$	Not given	$50\ mm \leq s_{sc} \leq 150mm$ $s_{edge} \geq 13\ mm$
CFS shear walls with gypsum sheathing			
Gypsum panel	$t_p = 12.5\ mm$	Not given	$w \geq 300\ mm$
Studs and track	$t_f \geq 0.9\ mm$	$f_y = 220\ MPa, t_f < 1.4\ mm$ $f_y = 350\ MPa, t_f \geq 1.4\ mm$	$s_s \leq 600\ mm$
Screws	$d = 4.2\ mm, d_h \geq 7.2\ mm$	Not given	$50\ mm \leq s_{sc} \leq 150mm$ $s_{edge} \geq 12.5\ mm$
t_p thickness of the sheathing panel t_f thickness of the frame element w width of the sheet or panel f_y yielding stress of the component d nominal diameter of the screw d_h head diameter of the screw s_{sc} screw spacing at panel edges. s_{edge} end distance from screw centre to sheathing edge			

Fig. 3. Geometrical and mechanical requirements for shear walls

3.1 CFS strap-braced walls

For strap-braced walls, the design resistance of the wall is calculated as a function of the yield resistance ($N_{pl,Rd}$) of the gross cross-section of the strap braces. The value of the yield resistance should be greater than the design value of axial force action in the strap brace (N_{Ed}) under the seismic design situation. Additionally, $N_{pl,Rd}$ should also be greater than the design net area resistance ($N_{u,Rd}$) of the strap brace. This requirement will allow the formation of plastic mechanisms in the steel straps before the net section failure in the strap connections to the steel frame. Values of the $N_{pl,Rd}$, and $N_{u,Rd}$ can be calculated using Equations 1 and 2, respectively. These Equations are already listed in Eurocode 3 – Part 1-1 [9]:n

$$N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}} \tag{1}$$

$$N_{u,Rd} = \frac{0.9A_{net}f_u}{\gamma_{M2}} \tag{2}$$

where A and A_{net} are the gross area and net cross-sectional area of the steel straps, respectively; f_y and f_u are the yield strength and ultimate strength of the steel straps, respectively; and γ_{M0}=1.0 and γ_{M2}=1.25 are the relevant partial safety factors.

3.2 CFS shear walls with steel sheet sheathing

For CFS shear walls with steel sheet sheathing, the design resistance ($R_{c,Rd}$) of the wall is calculated as the function of the sheathing connections within the affective strip of sheathing. $R_{c,Rd}$ should be greater than the design value of lateral force acting on the shear walls in a seismic design situation, while it should be less than the yielding resistance of the affective strip of sheathing ($R_{y,Rd}$). This requirement ensures that the wall resistance is governed by the strength of the connections within the affective strip of sheathing.

Values of $R_{c,Rd}$, and $R_{y,Rd}$ can be calculated using the Effective Strip Method (ESM) proposed by Yanagi and Yu [10]. This method defines the shear resistance of the wall as the function of the effective strip width of the steel sheathing formed under the action of lateral loads. The effective strip is formed in the shear wall due to the diagonal tension field action. ESM defines the procedure for calculating the ESM (Figure 4), which is based on certain mechanical and geometrical characteristics of the shear wall. ESM was calibrated based on the large number of tests conducted on various types of specimens of CFS shear walls with steel sheathing in the USA and Canada. ESM [9] uses Equation (3) to calculate $R_{c,Rd}$, which is the function of design bearing resistances of the sheathing-to-track ($F_{b,Rd,st}$), sheathing-to-stud ($F_{b,Rd,ss}$) and sheathing to-track and stud connections ($F_{b,Rd,sts}$), calculated according to the formulations of the North American Specification for the design of CFS [11].

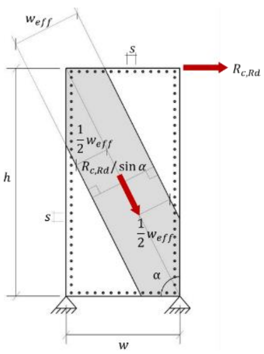


Fig. 4. Effective strip method [10]

$$R_{c,Rd} = \left(\frac{W_{eff}}{2s\ sin\ \alpha}F_{b,Rd,st} + \frac{W_{eff}}{2s\ cos\ \alpha}F_{b,Rd,ss} + F_{b,Rd,sts}\right) cos\ \alpha \tag{3}$$

where w_{eff} is the effective strip; s is the screw spacing at panel edges; α=arctg(h/w); h is the height of the wall; and w is the length of the wall.

The use of the ESM is only limited to the walls with an aspect ratio between 1.0 and 2.0 and a maximum steel frame thickness of 1.35 mm. This lower limit on the aspect ratio is proposed based on the geometry of the wall tests used to calibrate tthe ESM [10]. The wall tests used to calibrate the ESM had an aspect ratio greater than 1.0.

3.2 CFS shear walls with wood or gypsum sheathing

For CFS shear walls with wood or gypsum sheathing, the design resistance of the wall ($R_{c,Rd}$) is calculated as the function of the strength of sheathing connections, and it should be greater than the design value of lateral force (F_{Ed}) acting on the shear walls in the seismic design situation.

For CFS shear walls with wood sheathing the strength of the sheathing connection, ($F_{Rd,c}$), can be evaluated from the already existing design rules in [12]. Alternatively, the strength of the single sheathing connection can also be evaluated experimentally.

Once, the strength of the sheathing connection is obtained, $R_{c,Rd}$ can be calculated through various formulations in the literature [13,14,15]. All of these formulations relate the sheathing connection spacing and number of sheathed sides of the wall to the strength of a single sheathing connection in various ways and tend to give closely matched values of $R_{c,Rd}$. The lower bound method proposed by Källsner and Girhammar [13] for timber shear walls, which estimates the lateral strength ($R_{c,Rd}$) as the product of $F_{Rd,c}$ and number of fastener spacings along the top and bottom tracks, n_r, according to Equation (4), is the most simpler one.

$$R_{c,Rd} = n_r F_{Rd,c} \tag{4}$$

4 BEHAVIOUR FACTOR AND OVERSTRENGTH REQUIREMENT

4.1 General

The values of the behaviour factors proposed for 2nd Edition of Eurocode 8 are given in Table 1 for both DC2 and DC3 Class structures. Values for the DC3 Class structures are evaluated following the FEMA P695 approach [7] in past studies [3,4,5] considering several building archetypes representing various building heights, seismic hazards, and building occupancies. On the other hand, the values of the behaviour factor for the DC2 Class structure are proposed based on experience. These selected values for DC2 Class structure provide reasonable safety against the collapse as shown in [16].

To guard the non-dissipative components of the LFRS's against the failure, an overstrength would be provided in them for use in DC2 and DC3 Class structures, as described in the following Sections.

Table 1. Behaviour and overstrength factors

LFRS	DC2		DC3
	q	Ω	q
Strap-braced walls	2.0	1.5	2.5
Shear walls with steel sheet sheathing;	2.0	1.5	2.5
Shear walls with wood sheathing	2.0	1.5	2.5
Shear walls with gypsum sheathing	1.7	1.3	2.0

4.2 DC2 Class structures

In DC2 Class structures, the overstrength is applied using seismic action magnification factor (Ω) to develop 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 (5) on the non-dissipative components of the LFRS of a DC2 Class structure to verify their strength and stability against the total action effect E_{Ed}.

$$E_{Ed} = E_{Ed,G}\text{" + "}\Omega E_{Ed,E} \tag{5}$$

where: $E_{Ed,G}$ is the action effect in the non-dissipative member due to the non-seismic actions included in the combination of actions for the seismic design situation; and $E_{Ed,E}$ is the seismic action effect in the non-dissipative member due to the design seismic action. The overstrength factor for different types of LFRS's is listed in Table 1 along with their behaviour factor. It should be noted that the Equation (5) is valid for all four types of LFRS's.

4.3 DC3 Class structures

The overstrength requirements for DC3 Class structure are different for each type of LFRS. For strap-braced walls, the brittle components, i.e., connections of the steel straps, hold-downs, tension anchorages, and their connections or other tensioned vertical boundary elements at the ends of the wall, chord studs or other compressed vertical boundary elements at the ends of the wall, tracks and shear anchorages, should be designed with an overstrength computed according to Equation (6).

$$E_{Ed} \geq E_{Ed,G} + 1.1\overline{\omega}_{rm}E_{Nfy} \tag{6}$$

where, E_{Ed} is the total action effect in the non-dissipative brittle component; $E_{Ed,G}$ is the action effect in the non-dissipative member due to the non-seismic actions included in the combination of actions for the seismic design situation; ϖ_{rm} is the material overstrength factor accounting for the variability of the steel yield strength in the dissipative zones, i.e., ratio between the expected (average) and nominal yield strength, and ranges from 1.20 to 1.45 for lower to higher steel grades; E_{Nfy} is the action effect due to the yielding resistance N_{fy} of the gross cross-section of the strap braces based on the nominal yield stress of the material as defined in Eurocode 3 – Part 1-1 [9]. The factor of 1.1 accounts for the hardening in the dissipative zones.

For shear walls with steel sheet sheathing, Equation (7) is proposed, which ensures the overstrength in their brittle components.

$$E_{Ed} \geq E_{Ed,G} + 1.4E_{Rc,Rd} \tag{7}$$

where $E_{Rc,Rd}$ is the action effect due to the design resistance $R_{c,Rd}$ of the member-to-steel sheathing connections within the effective sheathing strip calculated according to Equation (3).

For CFS shear walls with wood or gypsum sheathing, Equation (8) is used to provide overstrength in their brittle components.

$$E_{Ed} \geq E_{Ed,G} + 2E_{Rc,Rd} \tag{8}$$

where $E_{Rc,Rd}$ is the is the action effect due to the design resistance $R_{c,Rd}$ of the member-to-wood or gypsum sheathing connections. The factor 2 being multiplied with $E_{Rc,Rd}$ is used to enforce more strict over strength requirements for DC3 class structures.

CONCLUSIONS

This paper presents the set of new seismic design rules for CFS buildings based on the past research carried out at the University of Naples Federico II as well as on the existing design standards currently adopted outside Europe. The design rules cover main CFS LFRSs and three levels of ductility Classes according to their energy dissipating capacities. Capacity design rules and limitations on the geometrical and mechanical properties are given for DC2 and DC3 Class structures, while no specific capacity design rules and limitations are required for DC1 Class. 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 a range of building archetypes. Overstrength rules are provided separately for DC2 and DC3 Class structures to safeguard against the brittle failure mechanism in the non-dissipative components. Furthermore, formulations to predict the design strength of the wall are also provided.

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KEYWORDS

Eurocode 8, seismic design rules, lightweight steel structures, lateral force resisting systems, cold-formed steel.