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SEISMIC DESIGN RULES FOR LIGHTWEIGHT STEEL SYSTEMS IN NEXT EN1998-1-2

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Abstract

The seismic design of lightweight steel (LWS) structures is not covered by the current edition of Eurocode 8 (EN1998). In order to cover this structural typology in next Eurocode 8 and to reflect the findings of extensive research undertaken over the last years, the University of Naples Federico II took part in the development of the new rules for the design of LWS systems. This paper examines the Lateral Force Resisting System (LFRS) made of LWS profiles covered by next EN1998 - Part 1.2 in Section 11.14 and Annex (Normative) F, i.e. Strap-braced walls, Shear walls with steel sheet sheathing, Shear walls with wood sheathing, and Shear walls with gypsum sheathing and gives a summary of general design rules common to all systems and specific rules for each specific LFRS in case of dissipative structural behaviour.

Keywords: Cold-Formed Steel Structures, Earthquake Engineering, Eurocode 8, Lightweight Steel Structures, Shear Walls, Strap-Braced Walls, , Seismic Design.

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1 INTRODUCTION

Recent studies have highlighted the good seismic performances of traditional (heavy) steel constructions and Cold Formed Steel (CFS) systems [1-12], the latter even as nonstructural components. In particular, CFS buildings are generally framed structures having low to mid-rise height, consisting of walls built with vertically aligned studs that are evenly spaced and connected at both ends to tracks. The floors are constructed using joists placed at the same spacing as the studs. These structural elements are covered with sheathing materials such as steel sheet, wood, gypsum, or cement-based panels. Alternatively, steel-concrete composite slabs can serve as flooring systems.

To resist seismic forces, CFS buildings commonly incorporate either strap-braced walls or shear walls sheathed with steel sheet, wood, or gypsum panels as lateral force-resisting systems (LFRSs). Strap-braced walls function by employing diagonal steel straps that transfer loads through tension, forming a vertical truss system. In contrast, CFS shear walls face seismic loads through sheathing connections, i.e. connection between the sheathing and the steel frame. In strap-braced walls, energy dissipation occurs when steel straps yield under tensile forces. For shear walls sheathed with wood or gypsum panels, dissipation results from the tilting and bearing of screws against the sheathing. And for shear walls with steel sheet sheathing the energy dissipation primarily occurs through sheathing connections and the plastic elongation of tension fields within the sheets under lateral in-plane forces.

Both strap-braced walls and shear walls typically utilize studs and tracks formed from Class 4 cross-section CFS profiles. The outermost studs in these walls are known as chord studs. To face overturning effects, these chord studs are usually assembled using back-to-back coupled profiles, with hold-downs and tension anchorages installed at their endpoints. Shear anchorages are positioned along the top and bottom tracks to transfer in-plane shear forces.

Ensuring ductile behavior is essential in a dissipative design, and requires preventing brittle failure in various components of the LFRS. In aforementioned LFRSs, this includes the end connections of steel straps in strap-braced walls, the sheathing panels in shear walls, as well as the hold-downs, tension anchorages, their connections, chord studs, tracks, and shear anchorages in both strap-braced walls and shear walls.

For gravity load resistance, the structural design of CFS buildings follows the relevant sections of Eurocode 3 [13], 4 [14], and 5 [15]. However, in Europe, the current version of Eurocode 8 (EN 1998 [16]) does not provide specific seismic design guidelines for CFS buildings. This limitation makes it challenging to implement CFS structures in regions with moderate-to-high seismic activity. Under the European existing standards, only CFS buildings with strap-braced walls can be designed according to the principles of low-dissipative structures within Ductility Class Low (DCL), and their use is primarily advised for areas with low seismic risk.

In reality, the current edition of EN 1998 has been in place for over 15 years, making certain provisions outdated due to advancements in research and technology over the past decade. Additionally, some sections have been difficult to interpret and apply, as noted by engineers and practitioners based on their professional experience. At present, all European structural codes are under revision to bridge the gap between contemporary research, technological advancements, and regulatory standards while addressing these concerns. The CEN/TC250/SC8 committee is overseeing the comprehensive revision of all sections of EN 1998. Among its initiatives, SC8 is working on integrating new LFRSs—including CFS systems—into the next edition of EN 1998, aiming to promote wider adoption of CFS structures across Europe.

2 NORD AMERICAN STANDARD FOR SEISMID DESIGN OF CFS BUILDINGS

The seismic design of CFS buildings in North America is governed by the AISI S400 standard [17]. This regulation applies across the United States, Canada, and Mexico, outlining seismic design requirements for various lateral force-resisting systems (LFRSs). These include CFS special bolted moment frames, CFS light-frame shear walls with steel sheet sheathing, and CFS light-frame shear walls with wood structural panels. Other covered systems include shear walls sheathed with gypsum board or fiberboard panels, shear walls that combine wood-based structural panels on one side with gypsum board panels on the other, as well as CFS light frame strap braced wall systems, conventional construction CFS light frame strap braced wall systems.

For each of these systems, the standard specifies key seismic parameters required for structural design. In the United States and Mexico, compliance with ASCE 7: Minimum Design Loads for Buildings and Other Structures [18] requires values such as the response modification coefficient (R), overstrength factor (Ω), and deflection amplification factor (Cd). Meanwhile, in Canada, the standard follows the National Building Code of Canada (NBCC [19]) and defines the response modification factor (R), which results from the combination of the ductility-related factor (R_d) and the overstrength-related factor (R_O). Additionally, it includes the expected overstrength factor (Ω_E). Furthermore, the standard provides design strengths based on experimental data for predetermined LFRS configurations.

AISI S400 also establishes limitations on the geometric and mechanical properties of LFRSs. It incorporates the capacity design approach, ensuring that energy dissipation occurs in designated components by reinforcing the strength of non-dissipative elements within the LFRS. Additionally, ASCE 7 in the United States and NBCC in Canada set restrictions on building height and seismic intensity levels for locations where CFS structures may be implemented.

3 FRAMEWORK OF THE UPCOMING EN 1998 EDITION

The forthcoming edition of EN 1998 incorporates a well-established multi-performance approach, classifying building damage levels into four distinct limit states (LSs): Near Collapse (NC) LS, Significant Damage (SD) LS, Damage Limitation (DL) LS, and Fully Operational (OP) LS.

This new version also introduces three categories of ductility levels for building structures based on the type of LFRS used. These classifications are: DC1 (low-dissipative), which corresponds to low energy dissipation capacity; DC2 (dissipative), which represents a medium level of energy dissipation; and DC3 (dissipative), which indicates a high level of dissipative behavior.

In general, structures categorized under DC1 and DC2 are subject to a maximum allowable seismic intensity during the design phase, whereas DC3 structures do not have this restriction. The upper limit for seismic intensity is determined using a specific threshold value of S_δ, which represents the maximum response spectral acceleration (at 5% damping) within the constant acceleration range of the elastic response spectrum.

Regarding CFS systems, the current draft of the upcoming EN 1998 edition (as of March 2023) addresses this structural category in Section 11.14 (Design Rules for Lightweight Steel Systems) of Part 2 (prEN 1998-1-2: Design of Structures for Earthquake Resistance – Part 1-2: Rules for New Buildings [8]) and in Annex F (Steel Lightweight Structures).

The LFRSs included in the code are: (1) CFS strap-braced walls, (2) CFS shear walls with steel sheet sheathing, (3) CFS shear walls with wood sheathing, and (4) CFS shear walls with gypsum sheathing.

4 GENERAL REQUIREMENTS FOR CFS LFRSs

The forthcoming edition of EN 1998 establishes a set of general provisions applicable to all DC1, DC2, and DC3 Class CFS LFRSs. These guidelines are designed to ensure that key research findings supporting the proper performance of LFRSs are taken into account during the design process.

One of these prescriptions involves a fixed maximum aspect ratio (height-to-length ratio) of 2.0 for all LFRSs. This limitation is based on the observation that walls with an aspect ratio exceeding 2.0 tend to develop significant bending response, leading to excessive lateral deformation. Additionally, screws used in wall connections must possess adequate deformation capacity. To meet this requirement, the design shear resistance of the screws must exceed 1.2 times the design bearing resistance of the steel structural element, the design embedment resistance of wood or gypsum panels, or the design net area resistance of the strap brace. Moreover, the inter-story drift ratio at the Significant Damage (SD) limit state is restricted to a maximum of 1% for all CFS LFRSs.

4.1 Requirements for DC1 Class structures

For DC1 Class structures, there are no specific requirements related to overstrength or detailed seismic design. Their design can follow existing European standards such as EN 1993-1-3 [13], which is the primary guideline for CFS buildings, along with EN 1994-1-1 [14] for steel-concrete composite structures and EN 1995-1-1 [15] for timber structures. Additionally, no particular rules are specified for the behavior factor in DC1 Class structures, allowing the general value of $q = 1.5$ to be applied (Fig. 1). The maximum spectral acceleration (S_δ) is fixed equal to 5.0 m/s².

4.2 Requirements for DC2 and DC3 Class structures: design of dissipative elements

DC2 and DC3 Class structures must meet specific design criteria to ensure that their LFRS can withstand seismic forces through plastic deformation in dissipative components. The capacity design approach is used for this purpose, and the upcoming edition of EN 1998 provides comprehensive guidelines to ensure the energy-dissipative behavior of the LFRS.

To achieve effective energy dissipation in shear walls, the code specifies geometrical and mechanical requirements for individual components and wall assemblies. These requirements, which are based on limitations outlined in AISI S400, include restrictions on panel thickness, dimensions, and strength; frame element thickness and strength; sheathing connection spacing; screw diameters; sheathing edge distances; and stud spacing. However, strap-braced walls are not subject to these component limitations.

For strap-braced walls, the design resistance calculation is based on the yield resistance ($N_{pl,Rd}$) of the strap brace's gross cross-section. To ensure plastic mechanisms develop in the steel straps before failure occurs at the connection points, $N_{pl,Rd}$ must be not less than the net area resistance ($N_{u,Rd}$) of the strap brace. Both values can be determined using equations provided in EN 1993-1-1 [21], as summarized in Figure 2.

In the case of CFS shear walls with steel sheet sheathing, the wall's design resistance ($R_{c,Rd}$) depends on the strength of the sheathing connections within the effective strip of sheathing. To ensure failure occurs in the sheathing connections rather than in the sheathing material, $R_{c,Rd}$ must be lower than the yielding resistance ($R_{y,Rd}$) of the effective strip. The Effective Strip Method (ESM), developed by Yanagi and Yu [22], is used to compute these values. This method determines the shear resistance of a wall based on the effective strip width of the steel sheathing, which forms due to diagonal tension fields under lateral loading. The ESM approach, which takes into account both mechanical properties and geometrical characteristics of the shear wall, has been validated through extensive testing on various CFS shear wall specimens with steel sheathing in both the USA and Canada (Fig. 2).

System	DC1		DC2			DC3	
	q	S_δ m/s²	q	S_δ m/s²	Ω	q	S_δ m/s²
strap-braced walls	1.5	5.0	2.0	7.5	1.5	2.5	-
shear walls with steel sheet sheathing	1.5	5.0	2.0	7.5	1.5	2.5	-
shear walls with wood sheathing	1.5	5.0	2.0	7.5	1.5	2.5	-
shear walls with gypsum sheathing	1.5	5.0	1.7	7.5	1.3	2.0	-

Figure 1: Behavior factors and maximum spectral accelerations according to next prEN 1998-1-2 [20].

strap-braced walls

$$N_{Ed} \geq N_{pl,Rd};\ N_{pl,Rd} \geq N_{u,Rd};\ N_{pl,Rd} = \frac{Af_y}{\gamma_{M0}}\ \text{(A)};\ N_{u,Rd} = \frac{0.9A_{net}f_u}{\gamma_{M2}}$$

shear walls with steel sheet sheathing

$$F_{Ed} \geq R_{c,Rd};\ R_{c,Rd} \geq R_{y,Rd};$$

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

w_{eff}, see [10]

$F_{b,Rd,st}$, $F_{b,Rd,ss}$, $F_{b,Rd,sts}$, see AISI S100 [24]

shear walls with wood or gypsum sheathing

$$F_{Ed} \geq R_{c,Rd}$$

$$R_{c,Rd} = n_r F_{c,Rd}\ \text{(C)}$$ according to the lower bound method [23]

$$F_{c,Rd} = \frac{k_{mod}F_{c,Rk}}{\gamma_M}$$, for wood panels

N_{Ed} design value of axial force action in the strap brace under the seismic design situation
$N_{pl,Rd}$ design yield resistance of the gross cross-section of the strap braces
$N_{u.Rd}$ design net area resistance of the strap brace
A gross area of the steel straps
A_{net} net cross-sectional area of the steel straps
f_y yield strength of the steel straps
f_u ultimate strength of the steel straps
γ_{M0} partial safety factor for the resistance of the cross-section
γ_{M2} partial safety factor for the resistance of the net-cross-section
F_{Ed} design value of horizontal force action on the wall under the seismic design situation
$R_{c,Rd}$ design resistance of the wall
$R_{y,Rd}$ yielding resistance of the effective strip of sheathing
$w_{eff.}$ effective width of sheathing
s screw spacing at panel edge
α angle that the diagonal line of the sheet forms with the horizontal line
$F_{b,Rd,st}$ design bearing strength of a single sheathing connection between sheet and track
$F_{b,Rd,ss}$ design bearing strength of a single sheathing connection between sheet and stud
$F_{b,Rd,sts}$ design bearing strength of a single sheathing connection among sheet, track and stud
$F_{c,Rd}$ design shear strength of a single sheathing connection between panel, track track or stud
n_r number of fastener spacings along the top and bottom tracks
$F_{c,Rk}$ characteristic shear strength of a single sheathing connection between wood panel, track track or stud
k_{mod} coefficient dependent on loading duration and moisture content
γ_M partial safety factor considering the seismic actions as an accidental load combination

Figure 2: Equations for the design of dissipative elements according to next prEN 1998-1-2 [20].

For CFS shear walls with wood or gypsum sheathing, the design resistance ($R_{c,Rd}$) is based on the strength of the sheathing connections. In the case of wood-sheathed shear walls, the sheathing connection strength ($F_{Rd,c}$) can be determined using the design provisions specified in EN 1995-1-1 (Fig. 2) or through experimental testing. Once the sheathing connection strength is known, $R_{c,Rd}$ can be calculated using established methods from existing literature, such as the lower bound method introduced by Källsner and Girhammar [23] for timber shear walls (Fig. 2). However, a recent study [25] highlighted a significant underestimation of lateral in-plane resistance of CFS shear walls with wood sheathing obtained following the design provisions specified in EN 1995-1-1 for the evaluation of sheathing connection strength, by providing alternative design relationships.

4.3 Requirements for DC2 and DC3 Class structures: design of non-dissipative elements

In DC2 Class structures, the seismic action magnification factor (Ω) is utilized to account for overstrength, ensuring that the capacity design criteria are met. This factor considers both the overdesign of dissipative zones and the amplification of seismic effects in non-dissipative elements. To maintain the strength and stability of non-dissipative components within the LFRS, the overstrength factor is applied to these elements following Equation (1) in Figure 3.

The upcoming edition of EN 1998 introduces specific overstrength criteria for different LFRS types used in DC3 Class structures. For strap-braced walls, the overstrength of non-dissipative elements must be determined using Equation (2) in Figure 3. For shear walls with steel sheet sheathing, the overstrength of non-dissipative components is ensured through Equation (3) in Figure 3. Finally, for CFS shear walls sheathed with wood or gypsum panels, Equation (4) in Figure 3applies. Due to the underestimation of lateral in-plane resistance of CFS shear walls with wood sheathing obtained following the design provisions specified in EN 1995-1-1 for the evaluation of sheathing connection strength, the recent study [25] find a non-safe results if non-dissipative elements of CFS shear walls with wood sheathing are designed according the Equation (4), and propose a new expected overstrength factor

DC1 – All LFRSs

$$E_{Ed} = E_{Ed,G} + \Omega\, E_{Ed,E} \quad \textbf{(1)}$$

DC2 - Strap-braced walls

$$E_{Ed} \geq E_{Ed,G} + 1.1\omega_{rm}\, E_{Nfy} \quad \textbf{(2)}$$

DC2 - Shear walls with steel sheet sheathing

$$E_{Ed} \geq E_{Ed,G} + 1.4\, E_{Rc,Ed} \quad \textbf{(3)}$$

DC2 - Shear walls with wood or gypsum sheathing

$$E_{Ed} \geq E_{Ed,G} + 2\, E_{Rc,Ed} \quad \textbf{(4)}$$

E_{Ed} total action effect in the non-dissipative member
$E_{Ed,G}$ action effect in the non-dissipative member caused by non-seismic actions in the combination of actions for the seismic design situation
$E_{Ed,E}$ seismic action effect in the non-dissipative member due to the design seismic action
Ω overstrength factor (See Fig. 1)
E_{Nfy} action effect due to the yielding resistance Nfy of the gross cross-section of the strap braces based on the nominal yield stress of the material calculated using Equation (A) in Fig. 1, with γ_{M0}=1.0.
ω_{rm} overstrength factor accounting for the variability of steel yield strength in the dissipative zones ranging from 1.20 to 1.45
$E_{Rc,Ed}$ (For DC2 - Shear walls with steel sheet sheathing) action effect due to the design resistance of the member-to-steel sheathing connections within the effective sheathing strip calculated using Equation (B) in Fig. 1
$E_{Rc,Ed}$ (For DC2 - Shear walls with wood or gypsum sheathing) action effect due to the design resistance of the member-to- wood or gypsum sheathing connections. Calculated using Equation (C) in Fig. 1 for Shear walls with wood sheathing

Figure 3: Equations for the design of non-dissipative elements according to next EN 1998-1-2 [20].

5 CONCLUSIONS

This paper provides an overview of the seismic design regulations for CFS buildings outlined in the forthcoming edition of Eurocode 8. These regulations are derived from previous research carried out at the University of Naples "Federico II" and existing design standards. The framework includes four different types of Lateral Force Resisting Systems (LFRS): CFS strap-braced walls and CFS shear walls with steel sheet, wood, or gypsum sheathing. Additionally, three ductility classes are defined based on their ability to dissipate energy: DC1 (low-dissipative), DC2, and DC3 (dissipative). While DC2 and DC3 Class structures must adhere to capacity design principles and specific limits on geometrical and mechanical properties, DC1 Class buildings do not require such constraints. The proposed behaviour factor values for DC2 and DC3 structures are determined through studies applying the FEMA P695 [26] methodology to various building models. Overstrength provisions for DC2 and DC3 buildings are separately addressed to avoid brittle failures in non-dissipative components, even if a recent study [25] proposes an upgrading of seismic design rules for DC3 CFS shear walls with wood sheathing.

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