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A PROPOSAL OF SEISMIC DESIGN RULES FOR CFS SHEAR WALLS WITH WOODEN PANELS

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Abstract

Currently, European structural codes for seismic design [1] do not provide specific criteria for cold-formed steel (CFS) shear walls sheathed with wooden panels, limiting their application as lateral force-resisting systems in CFS structures. To address this regulatory gap, a study was conducted at the University of Naples "Federico II." The main objectives were to validate the method used to assess the shear resistance of the wall and to define seismic design parameters in accordance with the European standard format. The results indicate that the approach proposed by Easley et al. (1982) offers a reliable prediction of wall shear resistance, provided that the shear strength of screw connections between steel profiles and wooden panels is determined using reliable formulations. For design according to the limit state method, a safety factor of 1.30 can be applied to evaluate the nominal wall resistance. Additionally, for non-ductile components designed following the capacity design principle, an overstrength factor of 1.70 can be adopted.

Keywords: CEN EN1998, Cold-Formed Steel, Overstrength, Seismic Design, Shear Walls, Wooden Panels.

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1 INTRODUCTION

The revision process of the Eurocodes is nearing completion, and the latest edition of the second-generation Eurocodes is almost finalized. As part of this update, the new version of the European seismic design standards, specifically the next edition of Eurocode 8 [2], incorporates provisions for Lateral Force Resisting Systems in lightweight cold-formed steel (CFS) buildings, including strap-braced walls and shear walls sheathed with steel sheets, wooden, or gypsum boards [3]. These systems are in addition to the other steel systems that are already present in current European standards and have been studied in all their aspects by many researchers over the years [4-14].

For shear walls with wooden or gypsum boards, behavior factors were estimated based on research by Shakeel et al. (2020) [15], which followed the methodology outlined in FEMA P695 [16]. Overstrength factors were mainly determined based on empirical data, as theoretical models for evaluating the lateral resistance of these walls remain limited under current limit state design (LSD) principles. To address this gap, specific research has been conducted at the University of Naples "Federico II" to define seismic design provisions for CFS shear walls with wooden panels (SWWSs), following Eurocode methodologies.

The initial section of this paper provides an overview of the research background. Subsequently, it explores the distinctive seismic behavior of SWWSs, focusing on the shear resistance of connections between wooden panels and steel plates, which significantly affects overall wall performance. Lastly, a summary of the design requirements for SWWSs in the latest version of the second-generation Eurocode 8 [2] is presented. The latter part of the study examines the calibration of safety factor for design strength estimation and overstrength factor for non-ductile components.

2 SEISMIC BEHAVIOR OF SHEAR WALLS WITH WOODEN PANELS

Cold-formed steel (CFS) shear walls with wooden sheathing typically consist of CFS studs and tracks, covered with plywood or oriented strand board (OSB). In this structural system, gravity loads are supported by the vertical studs, while lateral in-plane forces are supported through the interaction between the frame and the wooden board, facilitated by screw connections. These connections play a crucial role in transferring forces between the wooden sheathing and the steel profiles. Additionally, horizontal steel straps can be installed on both faces of the wall at mid-height to enhance performance (Figure 1).

Steel profiles used in these walls generally range from 0.8 mm to 1.7 mm in thickness, while panel thickness varies between 9.5 mm and 12.5 mm for plywood and between 9 mm and 11 mm for OSB. The panels are typically fastened using self-tapping screws, with diameters between 4.2 mm and 4.8 mm and spacing equal to 50 mm to 150 mm along the perimeter, and 150 mm to 250 mm in the field. Although vertical orientation is the most common configuration for these panels, a horizontal arrangement is also possible.

Studies in literature indicate that, when the steel profiles, boards, and anchors are adequately oversized, the seismic performance of the wall is primarily controlled by the shear response of the wooden board-to-steel connections. Since this shear behavior significantly influences the overall lateral response, the spacing of these connections is a key parameter affecting wall performance. Various theoretical models have been developed for wood-framed shear walls, including the approach by Easley et al. (1982) [17] and the lower bound method proposed by Källsner and Girhammar (KGLB, 2009) [18], which estimate the lateral strength of the wall. The latter approach is considered in the most recent edition of the second-generation Eurocode 5 [19].

Assuming the symbols shown in Figure 2, which represents a wall segment having a vertical panel, three studs (two chord or end studs and one intermediate stud), and connections along the perimeter of the panel that are symmetrical and uniformly spaced, the following are reported relationships that allow calculation of the horizontal wall resistance, R_h, as a function of the connection shear resistance, F_v, according to KGLB (2009) method (Eq. 1) and Easley et al (1982) method (Eqs. 2 and 3).

$$R_h = \frac{b}{s} F_v \tag{1}$$

$$R_h = \frac{F_v b}{\sqrt{\left(\frac{b-2e_d}{n_r}\right)^2 + \left(\frac{h-2e_d}{\beta}\right)^2}} \tag{2}$$

with:

$$\beta = n_{ps} + \frac{4\sum_{i=1}^{n_r} x_i^2}{(b-2e_d)^2} \tag{3}$$

where n_r is the number of screw spacings along one track, n_{ps} is the number of screw spacings along one end stud, and x_i is the i^{th} screw abscissa.

Figure 1: Typical shear wall with OSB panels.

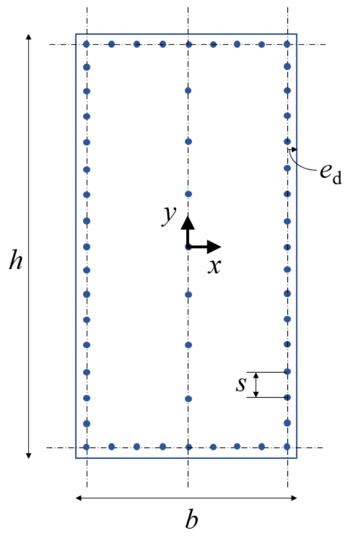


Figure 2: Geometrical parameters of the wall segment.

3 SHEAR RESPONSE OF THE CONNECTION BETWEEN WOODEN PANEL AND STEEL PLATE

When capacity design is applied to SWWSs, their seismic performance largely depends on the shear behavior of the connections between wooden panels and CFS profiles. Under shear forces, failure mechanisms involving the screw, panel, and steel plate can occur, including tilting, deflection, penetration, pull-out of the screw, tear-out of the board, and embedment of the CFS plate. If the minimal distance between the screw and the panel edge is maintained (typically more than 10 to 13 mm for board thicknesses ranging from 9 mm to 13 mm, and with screw diameters between 4.2 mm and 4.8 mm) tear-out is not a significant failure mode.

For typical configurations of fastener size, board thickness, and CFS plate thickness found in SWWSs, dominant failure mechanisms are screw tilting, screw penetration, and board embedment.

The latest edition of the second-generation Eurocode 5 [19] provides theoretical models for assessing the strength of connections between wooden components and steel elements. They are based on the "European Yield Model" (EYM), developed by Johansen (1949) [20]. According to this model, when the ratio of steel plate thickness to screw diameter is below 0.5, the plate is categorized as "thin." Given the panel thicknesses and screw diameters typically employed in SWWSs, the prevailing failure mode governing connection resistance is screw tilting. Therefore, the characteristic resistance of the connection, $F_{v,Rk}$, is given as a function of the screw diameter, d, the panel thickness, t, and the characteristic embedment strength of the panel, $f_{h,k}$:

$$F_{v,Rk} = 0.4 f_{h,k} t d \tag{4}$$

The characteristic embedment strength of the panel varies according to panel type. Specifically, for plywood it is a function of the characteristic density, ρ_k, for OSB it is a function of panel thickness, and for both materials it is a function of screw diameter:

$$f_{\mathrm{h,k}} = 0.11 \rho_{\mathrm{k}} d^{-0.3} \quad \text{for plywood panels} \tag{5}$$

$$f_{\mathrm{h,k}} = 65 d^{-0.7} t^{0.1} \quad \text{for OSB panels} \tag{6}$$

where $f_{h,k}$ is expressed in MPa, ρ_k in kg/m^3, t and d in mm.

4 SEISMIC DESIGN RULES IN THE LATEST EDITION OF THE SECOND-GENERATION EUROCODES

The latest edition of the second-generation Eurocode 8 [2] also includes SWWSs. In general, the standard allows the use of three Ductility Classes: Ductility Class 1 (DC1) for low dissipative structures; Ductility Class 2 (DC2) and Ductility Class 3 (DC3) for dissipative structures. Specifically, in DC1, energy dissipation capacity is not considered and a behavior factor q of 1.5 can be taken. In addition, no capacity design criteria are provided. For both DC2 and DC3, the energy dissipation capacity is considered, and the standard identifies the dissipative component of the system, represented for SWWSs by the connections between panels and profiles. For SWWSs in DC2, the behavior factor q is equal to 2.0 and the capacity design for the non-dissipative elements is adopted with the seismic action amplification coefficient (a coefficient that amplifies the effect on the non-ductile component of the design seismic action derived from the structural analysis), Ω_O, set equal to 1.5. For SWWSs in DC3, the behavior factor q is equal to 2.5, and the capacity design for the non-ductile components is applied through the overstrength factor, Ω_E, which amplifies the design resistance of the ductile component. According to the current edition of the second-generation Eurocode 8 [2], the horizontal design resistance of SWWSs, $R_{h,Rd}$, should be derived using the design provisions established in the latest edition of the second-generation Eurocode 5 [19], thus with the KGLB (2009) method (Eq. 7), where the design strength of the connection, $F_{v,Rd}$, should be calculated according to Equation 8:

$$R_{h,Rd} = \frac{b}{s} F_{v,Rd} \tag{7}$$

with:

$$F_{v,Rd} = k_{mod} F_{v,Rk}/\gamma_M = k_{mod} 0.4 f_{h,k} td/\gamma_M \tag{8}$$

where k_{mod} is a factor that depends on load duration and moisture content, and γ_M is the safety factor set equal to 1.2 for OSB and plywood boards [19].

For SWWSs in DC3, the overstrength of non-ductile elements can be calculated through the formula reported below:

$$E_{Ed} = E_{Ed,G} + \Omega_E\, E_{Rc,Ed} \tag{9}$$

where E_{Ed} is the total effect in the non-dissipative component, $E_{Ed,G}$ is the total effect due to the non-seismic actions, and $E_{Rc,Ed}$ is the effect of the seismic action in the non-dissipative component produced by the design resistance of the connections between panels and profiles, Ω_E is the overstrength coefficient amplifying the design strength of the connections, which is equal to 2.0.

5 CALIBRATION OF THE SAFETY FACTOR FOR RESISTANCE AND THE OVERSTRENGTH FACTOR

According to the LSD rules, if k_{mod}=1.0 is assumed, the horizontal design strength of the wall, $R_{h,Rd}$, can be assessed from the characteristic lateral strength of the wall, $R_{h,Rk}$, and the safety factor for strength, γ_M:

$$R_{h,Rd} = R_{h,Rk}/\gamma_M \tag{10}$$

from which γ_M can be obtained as:

$$\gamma_M = R_{h,Rk}/R_{h,Rd} \tag{11}$$

To calibrate γ_M, based on the statistical process provided by EN 1990 Annex D [21], it is first necessary to indicate a theoretical methodology for predicting the wall lateral strength. Therefore, the first step of the study was devoted to selecting the theoretical methodology correspondent to the best prediction. For this purpose, the lower bound method by Källsner and Girhammar (2009) and the method by Easley et al (1982) were applied to a large experimental database of shear walls with OSB or plywood boards [22-27] and connections [28]. Comparison of theoretical and experimental results demonstrates that the methodology by Easley et al (1982) provides a better prediction, with strength underestimation of 10% for shear walls with plywood panels and 7% for shear walls OSB panels. These results are obtained when the connection strength is set to the experimental value. Therefore, the methodology by Easley et al (1982) was considered for calibrating the safety factor for strength.

The second phase of this work was dedicated to theoretical prediction of connection strength. In this field, the comparison between the theoretical findings by EYM and the experimental results [28] showed a significant underestimation, with the ratio between experimental and theoretical strength equal to 3.4 for connections with plywood panel and 3.3 for connections with OSB panel. Therefore, to obtain a better correspondence between theoretical and experimental results, Equations (12) and (13) were proposed to be used as an alternative to EYM for the prediction of connection strength, F_v:

$$F_v = 0.149\, \rho d^{0.7} t \quad \text{for plywood panels} \tag{12}$$

$$F_v = 103\, d^{0.3} t^{1.1} \quad \text{for OSB panels} \tag{13}$$

and in terms of characteristic strength, $F_{v,k}$:

$$F_{v,k} = 0.149\, \rho_k d^{0.7} t \quad \text{for plywood panels} \tag{14}$$

$$F_{v,k} = 84.8\, d^{0.3} t^{1.1} \quad \text{for OSB panels} \tag{15}$$

For plywood panels, characteristic density, ρ_k, over average density, ρ, is taken equal to 0.823 [29], which corresponds to a coefficient of variation of 0.10 [30].

Equations (12) and (13) were obtained by imposing the average of the ratios of theoretical to experimental strengths equal to 1.0 for OSB and plywood panels.

The third step of the research regards the implementation of the method proposed by Easley et al (1982) with connection strength calculated using the proposed Equations (12) and (13). The results obtained showed a good result in terms of the ratio between experimental and theoretical resistance, with a value of 1.06 for plywood (Figure 3a) and 1.07 for OSB (Figure 3b).

The focus of the final phase of the study was the application of the process provided by EN 1990 Annex D [21] for calculating γ_M by considering the theoretical method of Easley et al (1982) with connection strength evaluated by Equations (12) and (13), obtaining a value of γ_M of 1.28 for plywood and of 1.23 for OSB. Consequently, a value of γ_M=1.3 can be used for shear walls with plywood or OSB panels. It should be noted that the latest edition of the second-generation Eurocode 5 [19] gives a γ_M of 1.2.

According to the approach provided by the latest edition of the second-generation Eurocode 8 [2], the design of non-dissipative elements depends on the overstrength factor that amplifies the design strength of the connection between panels and steel profiles (dissipative element), Ω_E, which can be defined as:

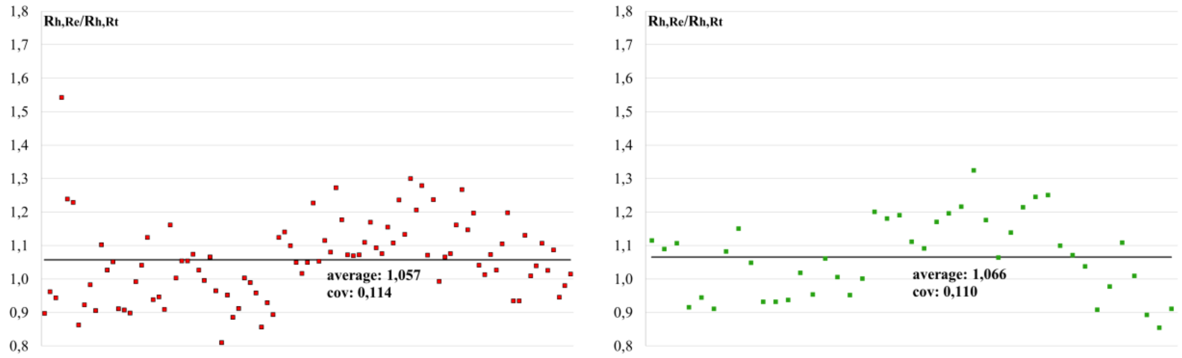
$$\Omega_E = R_{h,Re}/R_{h,Rd} = R_{h,Re}/(k_{mod} R_{h,Rk}/\gamma_M) \tag{16}$$

where $R_{h,Re}$, $R_{h,Rd}$, and $R_{h,Rk}$ are the expected resistance, design resistance and characteristic resistance of the wall, respectively.

For the evaluation of Ω_E, in Equation (16): the expected resistance was assumed to be equal to the experimental peak horizontal resistance; the characteristic resistance was derived using the method proposed by Easley et al (1982) (Eqs. 2 and 3) and the formulas for evaluating the characteristic strength of connections by Equations (14) and (15); γ_M was assumed to be equal to 1.3, in accordance with the results of the present study; k_{mod} was considered to be equal to 1.0.

The results showed that the ratio $R_{h,Re}/(R_{h,Rk}/\gamma_M)$ has an average equal to 1.59 for shear walls with plywood panels (Figure 4a) and equal to 1.74 for shear walls with OSB panels (Figure 4b). Consequently, an overstrength coefficient $\Omega_E=R_{h,Re}/(R_{h,Rk}/\gamma_M)=1.7$ can be used for shear walls with wooden panels. If $k_{mod}\neq1.0$, the overstrength coefficient to be used is:

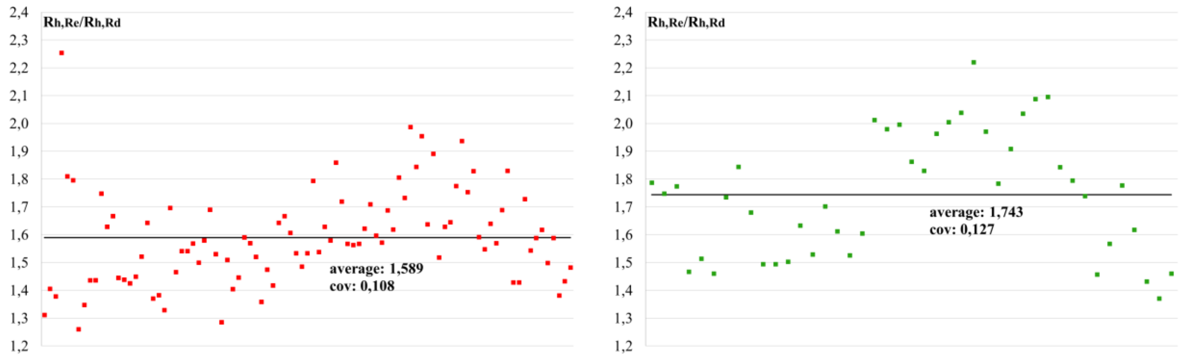
$$\Omega_E = 1.7/k_{mod} \tag{17}$$



(a) Shear walls with plywood panels

(b) Shear walls with OSB panels

Figure 3: Experimental strength over Theoretical strength by Easley et al (1982) method considering the connection strength proposed in this paper.



(a) Shear walls with plywood panels

(b) Shear walls with OSB panels

Figure 4: Ω_E for a unit k_{mod}.

6 CONCLUSIONS

Based on the results of this research, the following main conclusions can be drawn:

- Comparison between the KGLB (2009) method and the Easley et al (1982) method demonstrates that the best way to evaluate the lateral strength of shear walls with plywood or OSB boards is obtained with the second method;
- Application of the EYM (European Yield Model) exhibits a huge underestimation of the shear strength of connections between wooden boards and CFS sheets;
- The method by Easley et al (1982) is can be used for the evaluation of the lateral strength of shear walls with wooden panels, with the shear strength of connections between plywood or OSB panels and CFS profiles derived from the formulas proposed in this paper;
- Using the methodology proposed by Easley et al (1982) for the evaluation of lateral wall strength, a safety factor for strength, γ_M, of 1.3 and an overstrength coefficient, Ω_E, of $1.7/k_{mod}$ can be used for shear walls with wooden panels.

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