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# Seismic design procedure for cold-formed steel sheathed shear wall frames: Proposal and evaluation

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ABSTRACT

A seismic design procedure for CFS structures employing sheathed shear wall panels (SWP), compatible with the framework of the Eurocodes, is proposed in this paper. In order to assess the structural behaviour and generate the required data for the appraisal of the seismic design procedure, the OpenSees finite element environment was used to simulate the nonlinear behaviour of CFS-SWP adopting a novel deteriorating hysteresis model. Nonlinear static (pushover) and incremental dynamic analyses (IDA) have been carried out on 54 CFS-SWP frames having 2-, 4- and 5-storeys designed with varying seismic intensity levels. Fragility curves based on buildings collapse probability have been developed following the FEMA P695 methodology. Based on the defined design requirements, the CFS structural system evaluated in this study is shown to meet the acceptance criteria for a behaviour factor ($q$) equal to 2 for low- and moderate-seismicity. Furthermore, the results reveal that the lateral overstrength has a relevant influence on the probability of collapse and that an improved performance could be achieved if continuity of the CFS-SWP chord studs along the height is enforced.



## 1. Introduction

New innovative systems to ensure high structural performance have emerged in recent years in constructional steel practice. Among others, cold-formed steel (CFS) structures which offer some advantages over conventional structural system counterparts, such as high strength-to-weight ratio, controlled material quality and sustainability. The current version of the European code for seismic design, Eurocode 8 (EC8) [1], does not provide any guidance for CFS shear wall panel (SWP) system, which limits the use of this lateral load resisting system in construction practice. The North American Standard code of practice for Seismic Design of Cold-Formed Steel Structural Systems AISI S400-15 (2015) [2] represents the main reference for the lateral design of this type of structures. Since there is a significant difference between the European and the North American approaches in structural design regulations, a new seismic design procedure should be defined for CFS structures based on existing information, but tailored to fit the Eurocode (EC) requirements and typical European design practices.

Over recent years, researchers have carried out several experimental and numerical studies aiming at evaluating the collapse safety of CFS structures designed according to specific provisions. By conducting nonlinear dynamic analyses, Morello (2008) [3] validated seismic reduction factors and height limits provided in the AISI S213-07 (2007) [4] (AISI S400-15 [2] today) for wood-sheathed CFS framed SWP with and without gypsum sheathing board. Four-, 6- and 7-storey buildings have been designed for two different cities in Canada implementing the equivalent static force method with the ductility (Rd) and the overstrength (Ro) modification factors taken equal to 1.5 and 1.7, respectively. Using the ATC-63 Federal Emergency Management Agency methodology FEMA P695 (2009) [5], the outcomes showed that the modelled structures exhibited an acceptable seismic performance. Balh (2010) [6] adopted the FEMA P695 methodology to assess a seismic design procedure for steel-sheathed CFS-SWP frames; it has been shown that the initial test-based seismic force modification factors were not able to provide an acceptable level of safety against collapse. Subsequent analyses conducted by the same author resulted in a recommendation of Rd. value of 2.0 and Ro value of 1.3. A maximum height limit of 15 m was also recommended. DaBreo (2012) [7] carried out dynamic analyses on a 2-storey CFS building model to validate the test-based seismic force modification factors for ductility, Rd = 2.0, and for overstrength, Ro = 1.3 following a methodology adopted from FEMA P695, where the acceptance criteria set, given in this document for assessing response modification factors, were not met. More recently, and based on shake table test results, a numerical study has been undertaken by Shamim and Rogers (2015) [8] to evaluate the seismic performance of 2-, 4- and 5-storey CFS buildings with steel-sheathed SWP. The authors did not account for the strength deterioration due to repeated cycles in the modelling of the CFS-SWP, which led to recommended values of Rd = 2 and Ro = 1.3. Further investigation on the inclusion of non-structural gypsum sheathing boards

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showed that they could increase the collapse capacity of buildings. It is worth noting that all the above-described studies have been carried out to provide seismic design provisions for CFS-SWP frames specific to Canada. Vigh et al. (2013) [9] evaluated the seismic performance factors for a newly proposed CFS corrugated steel-sheathed SWP for use in midrise residential and commercial CFS buildings. The archetype buildings evaluated in the study, which were designed according to ASCE 7-10 [10] with a response modification factor (R) equal to 4, met the FEMA P695 acceptance criteria. In Europe, many experimental and numerical research activities on CFS structures were undertaken. Landolfo et al. (2006) [11], Iuorio et al. (2014) [12] and Fiorino et al. (2016) [13] performed monotonic and cyclic tests on different configurations of sheathed SWPs and diagonal strap-braced walls. Fülöp and Dubina (2004) [14], Corte et al. (2006) [15] and Vincenzo et al. (2014) [16] conducted numerical and theoretical studies on sheathed SWPs and diagonal strap-braced walls. Fiorino et al. (2009) [17], Landolfo et al. (2010) [18], Fiorino et al. (2012) [19] and Fiorino et al. (2014) [20] proposed a seismic design method for 1-storey CFS building. Although, some of these studies focused on the seismic behaviour of CFS sheathed SWPs and diagonal strap-braced walls, whilst others allowed the sheathed SWP components to be designed using sub-system level criteria. However, more research work on the use of advanced analysis methods for frames and further investigation of seismic design at the global building level as opposed to simply SWPs, is deemed necessary.

The main objective of this paper is to propose a seismic design and verification procedure for CFS buildings employing sheathed SWP that can integrate the current seismic design framework of EC8. The approach adopted in this research comprises the definition of a set of design criteria, the selection and design of a set of archetype buildings, the development of nonlinear building models in OpenSees [21] followed by the conduction of nonlinear static (pushover) and incremental dynamic analyses (IDA) of the archetype buildings following the FEMA P695 methodology. In order to validate the proposed seismic design procedure and to examine whether the adopted behaviour factors could provide a sufficient margin against collapse under maximum considered earthquake (MCE) ground motions, fragility curves based on buildings probability of collapse are subsequently developed. The seismic performance assessment of the archetype buildings for other limit states is available elsewhere [22].

## 2. Definition of design provisions and guidelines for CFS structures

In CFS structures, SWP is the primary lateral load resisting system; it is composed of CFS C-shaped framing members (studs and tracks) attached to steel/wood sheathing using screw fasteners. The inelastic behaviour that develops in the connection zone between the CFS frame and the sheathing board, resulting from bearing between the sheathing and the fasteners and tilting of the fasteners themselves, is the main mechanism of energy dissipation, providing that inelastic behaviour of the chord studs is prevented through capacity design. This structural component should be designed to provide adequate lateral shear strength and stiffness to the global structure.

Given the fact that EC8 does not provide guidelines for design of CFS-SWP system, in this study the latter is designed, in terms of strength criterion, in accordance with AISI S400-15 [2] adopting Load Resistance Factor Design (LRFD) method, which requires that this system have to resist the shear demand according to the following expression:

$$\phi R_n \geq \text{lateral design factored loads applied to SWP} \quad (1)$$

where:

$\phi$ Resistance factor;
$R_n$ Nominal shear capacity of the SWP.

A reliability analysis was carried out to assess the resistance factor for the ultimate limit state design with a target reliability index, $\beta$, of 2.5 following the provisions given in Chapter F of AISI S100-12 (2012) [23]. For this purpose, the results of 106 wood-sheathed CFS-SWP tests carried out by Branston et al. (2006) [24] have been adopted and the resulted value of the resistance factor, $\phi$, was 0.74. On the other hand, the values provided by AISI S400-15 [2] ($\phi = 0.65$ and 0.60 for wind and seismic design, respectively) are deemed conservative since several research findings, such as those reported by Yanagi and Yu (2014) [25] and Balh et al. (2014) [26], confirmed this conservatism. On the basis of these two works, the authors recommended a value of $\phi$ equal to 0.70 for sheathed CFS-SWP. Moreover, given the fact that Eurocodes do not provide guidance on the design of CFS-SWP lateral load resisting system, from the authors' perspective, it would be more consistent and accurate if the AISI S100-12 standard approach is adopted in calculating the value of $\phi$ using substantial experimental data rather than directly adopting the standard values of AISI S400-15.

The EC8 seismic design provisions require that the designed structure, when subjected to earthquake events, meets strength, drift and stability criteria [27]. According to the European seismic code, two limit states should be verified, namely the damage limitation and the ultimate limit states. With regard to the former limit state, EC8 establishes that interstorey drifts occurring for a frequent earthquake event should comply with the following expression:

$$d_r \upsilon = \psi h \quad (2)$$

where $d_r$ refers to the interstorey drift developing for the earthquake intensity corresponding to the ultimate limit state; $\upsilon$ is a reduction factor applied for the smaller, more-frequent, earthquakes associated with serviceability; $\psi$ is suggested as 0.5%, 0.75% and 1.0% for brittle, ductile and non-interfering non-structural components, respectively, and $h$ refers to the interstorey height [28]. As for the ultimate limit state, in addition to strength design check, second-order stability effects need to be addressed using the following expression proposed in EC8:

$$\theta = \frac{P_{tot}\, d_r}{V_{tot}\, h} \quad (3)$$

In the above expression, $P_{tot}$ and $V_{tot}$ are the total cumulative gravity load and seismic shear applied at the storey under consideration; $h$ is the interstorey height; and $d_r$ is the design interstorey drift. In case $\theta < 0.1$, second order effects could be neglected. However, if $0.1 < \theta < 0.2$, the second-order effect may be approximately taken into account by multiplying the relevant seismic action effects by a factor equal to $1 / (1 - \theta)$ and, in no case, the value of $\theta$ shall exceed 0.3 [1]. In this study, the $\theta$ coefficient was limited to 0.2.

When the SWP selection satisfies the strength, drift and stability criteria, the latter should likewise meet the overstrength regularity condition in order to obtain a uniform dissipative behaviour along the structure's height. This proposal is similar to that prescribed in EC8 for concentrically and eccentrically brace frames. In case of buildings with >2-storey, EC8 requires that the maximum overstrength factor does not differ from the minimum one by >25%, which directly affects the design of the lateral load resisting system. However, this condition in some cases is seldom satisfied since the shear demand that develops in archetype buildings' top storey is relatively smaller in comparison to the one acting in intermediate storeys. A less stringent limit was set as follows:

$$\frac{\Omega_{\max}}{\Omega_{\max}} - 1 \leq 0.50 \quad (4)$$

where $\Omega_{\max}$ and $\Omega_{\min}$ are respectively, the maximum and the minimum values of the structural overstrength factors for SWPs.

The design of the non-dissipative elements (track, stud, chord stud, and hold-down) was carried out according to the prescriptions of Parts 1–3 of Eurocode 3 (EC3) [29], applicable to thin-walled members. Cross-section design checks were performed for the vertical members