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Seismic risk assessment of cold-formed steel shear wall systems

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ABSTRACT

This paper presents the probabilistic seismic performance and risk assessment of cold-formed steel (CFS) sheathed shear wall panel (SWP) structures adopting conventional steel moment-resisting frame (MRF) systems as a benchmark with the aim of exploring the viability of using CFS-SWP as a new structural solution in seismic prone regions. A set of 12 building structures of both systems, with 2-, 4- and 5-storey, have been designed for two seismic intensity levels. To simulate their nonlinear behaviour, the structures were modelled adopting recently developed deteriorating hysteresis models. Based on probabilistic seismic hazard analyses (PSHA), a site-specific selection of ground motion records for Incremental Dynamic Analyses (IDA) has been carried out adopting the Conditional Mean Spectrum (CMS) as a more realistic target response spectrum. Subsequently, the seismic risk was evaluated over the structure lifetime (*i.e.*, 50 years) in terms of the annual probability of exceeding the Damage Limitation, No-Local Collapse and Near Collapse limit states. The importance and usefulness of the risk metrics are highlighted and adopted as an indicator to explore the behavioural features of both structural systems. Overall, the assessment procedure showed that both systems present an acceptable seismic performance and therefore the CFS-SWP can be seen as a reliable structural solution to achieve performance-based objectives in seismic regions.

1. Introduction

In constructional steel practice, conventional steel moment-resisting and concentrically-braced frames (MRFs and CBFs, respectively) represent the most common solutions for buildings to withstand lateral loads (wind and earthquake). The reliability of these lateral load resisting systems was confirmed and improved from the performance observed in past earthquake events and also from significant past research activities that have culminated in detailed seismic design provisions adopted worldwide. In recent years, new innovative systems to ensure high structural and environmental performance have emerged. Among others, cold-formed steel (CFS) shear wall panel (SWP), represents an effective structural system to resist lateral loads for low- and medium-rise CFS buildings, offering a potential benefit from using lightweight framing components, thus, limiting the seismic mass. Nevertheless, conventional steel MRFs and CBFs are still preferred due to the more complex analysis and design procedures required when dealing with thin-walled CFS framing members, which develop local instabilities and several failure mechanisms. Besides, the fact that there is no prescription in the European seismic code, Eurocode 8 (EC8) [1], for the design of CFS-SWP, hinders the use of this lateral load resisting system in construction practice.

A number of research activities on CFS have been carried out in North America by Branston et al. [2], Yu [3], Yu and Chen [4], Balh et al. [5], DaBreo et al. [6] and Liu et al. [7] through quasi-static tests on SWPs as well as a dynamic test program conducted by Shamim et al., [8]. Many experimental and numerical research activities were also undertaken in Europe with the aim of gaining a deep understanding of the behaviour of CFS components and broaden their use as a new structural solution. Landolfo et al. [9], Iuorio et al. [10] and Fiorino et al. [11] performed monotonic and cyclic tests on different configurations of sheathed SWPs and diagonal strap-braced walls. Fülöp and Dubina [12], Della Corte et al. [13] and Vincenzo et al. [14] conducted numerical and theoretical studies on sheathed SWPs and diagonal strap-braced walls. Fiorino et al. [15], Landolfo et al. [16], Fiorino et al. [17] and Fiorino et al. [18] proposed a seismic design method for 1-storey CFS buildings. Ultimately, the main outcomes of these studies served for the characterization of the monotonic and the cyclic nonlinear behaviour of CFS sheathed SWPs and diagonal strap-braced walls, and allowed to establish a design procedure of these structural components. It is noteworthy that all the above-described works addressed the sub-system level behaviour of the CFS lateral load resisting system. As far as the full structure behaviour is concerned, Peterman [19] has conducted shake table tests on two full-scale CFS framed 2-storey buildings. The results highlighted the adequate structural performance under seismic loads where the buildings showed to be stiffer and stronger than what they were designed for (sub-system level design).

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A subsequent numerical study performed by Leng [20] which addressed the advanced 3D modelling of 2-storey CFS buildings' structure adopting experimental data spanning from fastener to full scale shake table level tests. Based on fragility analyses, similar conclusions as extracted experimentally by Peterman [19] have been drawn regarding the structural performance where acceptable levels of collapse safety were achieved. Although a major understanding of the behaviour of CFS structures under seismic loading conditions has been learned, the potential of CFS systems has not been fully evaluated yet in terms of risk assessment, based on a probabilistic method incorporating uncertainties that arise from the occurrence and intensity of earthquakes for limit states probability of exceedance (PoE). Therefore, it is deemed necessary to incorporate much of the previous research findings into the context of structural reliability to identify the performance of CFS buildings in seismic regions lacking proper standardised specifications for seismic design and verification.

In this paper, a probabilistic framework for structural performance assessment of CFS-SWP system with reference to conventional steel MRFs in terms of seismic risk, is presented. For this purpose, 2-, 4- and 5-storey buildings of each structural system have been designed for two seismic intensity levels and then modelled using the OpenSees finite element (FE) software [21]. Incremental Dynamic Analyses (IDA) were performed to generate the required data for the development of fragility curves adopting the Conditional Mean Spectrum (CMS) [22] to select site-specific ground motion records. Subsequently, in order to provide insights into the relative performance of both structural systems, the seismic risk is evaluated over the structure lifetime (*i.e.*, 50 years) in terms of the annual probability of exceeding the Damage Limitation (DL), No-Local Collapse (NLC) and Near Collapse (NC) limit states following the probabilistic SAC/FEMA closed-form framework [23], assuming a biased hazard fitted with a second-order power-law function [24].

2. Seismic design provisions

2.1. Sheathed CFS-SWP system

The CFS-SWP using wood or steel sheathing boards is a code approved lateral load resisting system for low- and medium-rise CFS buildings in North America, Australia and New Zealand. It is composed of CFS C-shaped framing members (chord studs, studs and tracks as shown in Fig. 4a) attached to sheathing boards using screw fasteners. In addition to gravity load resistance, this structural system dissipates energy by taking advantage of the inelastic behaviour that develops in the connection zone between the CFS frame and the sheathing board whilst failure of chord studs and Hold-Down elements is prevented through capacity design. Given the fact that EC8 does not provide guidance on the design of CFS-SWP lateral load resisting system, in this study the design of this structural system is carried out following a design procedure tailored to the framework of EC8 [25], adopting a behaviour or response modification factor (q) equal to 2. As for the design of non-dissipative elements (track, stud, chord stud and Hold-Down), capacity design rules for thin-walled members provided in Part 1.3 of Eurocode 3 (EC3) [26] are adopted herein. Since the CFS profiles are made of slender cross-sections (Class 4 according to EC3 classification), local buckling is expected to occur before the attainment of the yield stress in one or more parts of the cross-section. Either the Effective Width Method (EWM) or the more accurate Direct Strength Method (DSM) [27] could be used to evaluate their axial and flexural design strengths in order to take into account the strength reduction resulting from the development of buckling effects (local, distortional and global). Further details on the seismic design procedure for CFS-SWP system and its performance factors can be found in the work of Kechidi et al. [25].

2.2. Steel moment-resisting frames

The response of a conventional steel MRF depends on the characteristics of its fundamental components namely the columns, beams and connections (*e.g.*, beam-to-column connection). In this structural system, the shear yielding of the panel zone as well as the flexural yielding of the beams represent the main source of energy dissipation. In this study, the MRFs have been firstly designed to resist gravity loads in accordance with the provisions of EC3 [28] for sectional resistance, stability checks and deflection serviceability limits. Afterwards, seismic design was performed in accordance with the EC8 provisions considering a behaviour factor (q) equal to 4. Two limit states were verified, namely damage limitation and ultimate limit states. Although it is not specifically defined in EC8 [1], it is considered by the authors that the first step of the design process should be the DL limit state checking, particularly in the case of flexible structures located in moderate-to-high seismicity regions (see Section 3). Regarding the NC limit state, the design process consists of checking the dissipative elements followed by capacity design of non-dissipative elements. The capacity design of the non-dissipative members was conducted according to the EC8 criteria with the modifications proposed by Elghazouli [29]. As for the design of the panel zone, a "balanced" design approach was adopted in this study [30] which establishes that panel zones should be proportioned such that yielding of these elements occurs at similar load levels that develop flexural plastic hinges in the beams. The potential influence of second-order P-Δ effects should be checked through the calculation of the inter-storey sensitivity coefficient θ. In this study, the θ coefficient was limited to 0.2, meaning that an amplification of the lateral load had to be performed during the design process [1]. Furthermore, the DL performance requirement was considered in the seismic design by limiting the inter-storey drift ratio during a frequent earthquake event to 1% of the storey height.

3. Selection and design of the buildings

Two-, 4- and 5-storey CFS-SWP and MRF buildings have been selected and then designed. Table 1 summarizes the parameters used to describe the design space where two sites located in Portugal, namely Porto (north) and Lagos (south) were considered to reflect, respectively, low and moderate-to-high seismicity regions. The acceleration and displacement elastic response spectra are plotted in Fig. 1. The storey heights of CFS-SWP and MRF systems are 2.74 m and 3.50 m, respectively, with a MRF first floor height equal to 4.50 m.

A simple floor plan was selected for the buildings studied herein (Fig. 2). For the CFS-SWP system, rectangular buildings with perimeter shear walls that resist lateral forces for each direction intending to represent a typical CFS structure where the length of the lateral load resisting system is proportional to the lateral demand (Fig. 2a). As for the MRF system, the structural configuration in plan is shown in Fig. 2b. The buildings consist of three MRFs spaced at 6 m. Resistance to seismic loads is provided by the three frames in the longitudinal

Table 1
Parameters of the design space for CFS-SWP and MRF systems.

Building	Number of storeys	Design load level	
		Occupancy	Seismicity
1	2	Residential	Low (PGA = 0.8 m/s^2, soil class B)
2	4		
3	5		
4	2		Moderate-to-high (PGA = 2.5 m/s^2, soil class C)
5	4		
6	5		