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Lateral strength and deflection of cold-formed steel shear walls using corrugated sheathing

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ABSTRACT

Recent research has proven that cold-formed steel shear wall with corrugated steel sheathing is a promising lateral force resisting system in high wind and seismic zones. Extensive experimental investigations, including monotonic and cyclic tests on cold-formed steel shear walls with corrugated steel sheathing, were recently completed at University of North Texas. This paper summarizes previous and newly conducted tests and presents finite element analyses in order to establish a set of nominal shear strengths for the corrugated steel sheathed shear walls. In addition, a design method for determining the deflection of the corrugated steel sheathed shear walls under in-plane lateral loading was proposed based on the experimental results and nonlinear regression analyses. The deflections obtained from the proposed design equation were compared with the test deflections and good agreement was obtained.

1. Introduction

Cold-formed steel (CFS) shear wall with corrugated steel sheathing is a newly proposed lateral resisting system from recent research. Extensive experimental research has been done on the shear resistance of cold-formed steel framed shear walls under both monotonic and cyclic loading (Fülöp and Dubina [1], Stojadinovic and Tipping [2], Yu et al. [3]). The experimental results revealed that CFS framed shear walls using corrugated steel sheathing yielded higher strength, greater initial stiffness with similar ductility under cyclic loading compared to the CFS walls using conventional sheathing materials such as flat steel sheets. It is also worth noting the major difference between the CFS shear wall with the steel plate shear wall (SPSW) which has been relatively well studied and its design procedure is included in the current American Institute of Steel Construction (AISC)'s "Seismic Provisions for Structural Steel Buildings" (AISC 341-10 [4]). The major differences between those two shear wall systems are the boundary members and the connection method. The CFS shear wall system uses CFS thin-walled steel members and self-drilling screws are commonly used for attaching the sheathing (flat steel sheets or corrugated steel sheets) to the frame. While the SPSW system employs hot-rolled steel members for the frame and the sheathing is typically welded to the boundary elements via welds to form continuous attachment along the edge of the sheathing. Due to those differences, the SPSW's shear strength is controlled by the yield strength of the steel sheathing and it is capable of providing appropriate shear strength for high-rise buildings. The CFS shear wall's shear strength is generally limited by the screw connections between the sheathing and the framing members. The CFS shear wall is suitable for low- and mid-rise buildings. In terms of the seismic energy dissipation mechanism, the SPSW mainly relies on the strain deformation in the steel plate via tension filed action. The CFS shear wall is commonly considered to dissipate seismic energy by deformation of the screw connections on both the sheathing and the framing.

The objective of this paper is to provide the designers and engineers with guidance for the shear resistance and the design deflection calculation of CFS shear walls with corrugated steel sheathing. The previous test results completed at University of North Texas were summarized and complementary tests were conducted. Finite element analyses using ABAQUS were performed in order to supplement and complete the test data. Recommended shear resistance and a proposed design equation for determining the deflection of the corrugated steel sheathed shear walls under lateral loading were presented.

2. Experimental program

2.1. Test specimens

All the relevant tests from Yu et al. [3, 5, 6], Mahdavian [7], Zhang et al. [8, 9] as well as the complementary tests are summarized in Table 1.

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Table 1
Shear wall details.

Source	Test label	Width (m)	Loading method	Gravity load	Stud	Track	Sheathing	Screw	Screw spacing at panel edges/field (mm)
Yu et al. [3]	8	1.22	M	–	350S162-68	350T150-68	Vulcraft 0.6C, 0.69 mm (22 ga)	#12	64/127
Yu et al. [5, 6]	2	1.22	M	–	350S162-68	350T150-68		#12	64/127
	12	1.22	M	–	350S162-68	350T150-68		#12	64/127
	7	1.22	C	–	350S162-68	350T150-68		#12	64/127
	19	1.22	C	–	350S162-68	350T150-68		#12	64/127
Mahdavian [7]	54	1.22	M	–	350S200-68	350T150-68	Verco Decking SV36, 0.69 mm (22 ga)	#12	76/152
	2	1.22	C	–	350S162-68	350T150-68		#12	76/152
	5	1.22	C	–	350S162-68	350T150-68		#12	76/152
	32	0.61	C	–	350S162-68	350T150-68		#12	76/152
	62	1.22	C	–	350S162-54	350T125-54	Verco Decking SV36, 0.46 mm (24 ga)	#10	76/152
	63	1.22	C	–	350S162-54	350T125-54		#10	76/152
Zhang et al. [8]	SW-M1	1.22	M	Yes	350S200-68	350T150-68	Verco Decking SV36, 0.69 mm (22 ga)	#12	76/152
	SW-M2	1.22	M	Yes	350S200-68	350T150-68		#12	76/152
	SW-C1	1.22	C	Yes	350S200-68	350T150-68		#12	76/152
Zhang et al. [9]	1	1.22	C	–	350S200-68	350T125-68		#12	76/152
	2	1.22	C	–	350S200-68	350T125-68		#12	76/152
Complementary tests	C-1	0.61	C	Yes	350S200-68	350T150-68		#12	76/152
	C-2	1.83	C	Yes	350S200-68	350T125-68		#12	76/152

In this research, all shear wall specimens were of 2440 mm (8 ft.) height and the width of the walls varied from 610 mm to 1830 mm (2–6 ft.) to provide an aspect ratio of 4:1, 2:1 and 4:3. The framing members used Steel Studs Manufacturers Association (SSMA) structural studs and tracks. The boundary studs were fastened together back-to-back with No.12 × 25.4 mm (1 in.) hex head self-drilling screws with 152.4 mm (6 in.) distance on center. The field stud used a single C-shaped member. Two Simpson Strong-Tie hold-downs S/HD15S were used for each specimen, where one was attached to the inside of the boundary studs by No.14 × 25.4 mm (1 in.) Hex Washer Head (HWH) self-drilling screws.

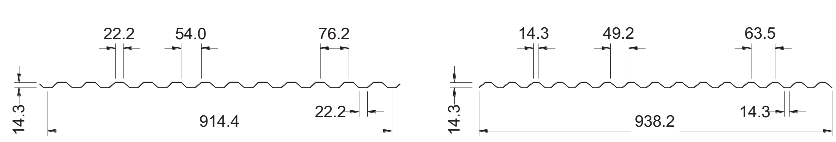
The sheathing consisted of three corrugated steel sheets and was applied on one side of each specimen using No.12 × 25.4 mm (1 in.) HWH self-drilling screws. It should be noted that the 0.69 mm (22 gauge) low-profile corrugated decking used two different specifications from two manufactures. The profile dimensions of these two decks are shown in Fig. 1 and the section properties are listed in Table 2. The 'p' and 'n' in the Table 2 represent the crest and trough (positive and negative) of the section profile. Due to the corrugation profile, the screw spacing was limited to 64 mm (2.5 in.) module for Vulcraft decking and 76 mm (3 in.) module for Verco decking. As can be seen from Table 2, the difference of the section properties between the two decks is not significant, less than 10%. Therefore the influence of the section properties is neglected and the difference between the shear resistances is deemed to be caused by the screw spacing only. The detailed wall configurations are illustrated in Fig. 2. Table 3 summarizes the material properties reported in the references that were used in this work.

2.2. Complementary test results

The complementary test contains two shear wall specimens with different aspect ratios. The test setup and loading condition of the commentary tests was the same as in Zhang et al. [8]. Combined lateral and gravity loading was applied. The gravity loads were calculated using the tributary areas theory of a typical 2-story office building and included the dead load plus 25% of live load. The applied gravity load was 32.0 kN for the 2.44 m × 1.83 m (8 ft. × 6 ft.) shear wall and 12.0 kN for the 2.44 m × 0.61 m (8 ft. × 2 ft.) shear wall. The failure of the 2.44 m × 1.83 m (8 ft. × 6 ft.) shear wall specimen was governed by shear buckling of the corrugated steel sheathing which resulted to screw pulling over the sheathing. By the end of loading protocol, the 2.44 m × 1.83 m (8 ft. × 6 ft.) shear wall suffered screw failures at the horizontal seams, represented by unzipping of connection along the entire seam, which led to the detachment of the bottom sheathing from the frame. The deformations of the 2.44 m × 1.83 m (8 ft. × 6 ft.) shear wall are shown in Fig. 3. The failure of the 2.44 m × 0.61 m (8 ft. × 2 ft.) shear wall specimen was governed by screw pulling over at the bottom sheathing. Bottom track distortional buckling was noticed after peak load was obtained. No obvious sheet buckling was observed during the loading process. The deformations of 2.44 m × 0.61 m (8 ft. × 2 ft.) shear wall are shown in Fig. 4. Hysteresis responses of these two wall specimens are shown in Fig. 5.

2.3. Strength and displacement data

The performance parameters obtained from each wall specimen are provided in Table 4. The results include the test peak load, P_{max}, lateral displacement at peak load, initial stiffness, and the ductility factor. The initial stiffness and the ductility factor were calculated using the Equivalent Energy Elastic Plastic model (EEEP) according to North American Standard for Seismic Design of Cold-Formed Steel Structural Systems AISI S400 [10]. The initial stiffness refers to the secant stiffness at $0.4P_{max}$. The ductility factor is defined as the ratio of $\Delta_{0.8u}$ to Δ_y, where is $\Delta_{0.8u}$ the displacement at 80% post ultimate load, and Δ_y is the displacement at yielding. For the hysteresis curve, backbone curves in both the positive and negative displacement regions were first identified by plotting locus of all the peak force points at the first cycle of the same displacement amplitude cycles. Then the parameters were determined from the back-bone curves.



(a) Verco Decking SV36, 0.69 mm (22 Ga) (b) Vulcraft 0.6C, 0.69 mm (22 Ga) and 0.46 mm (24 Ga)

Fig. 1. Corrugated steel sheet profiles.