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Higher strength cold-formed steel framed shear wall with sheet steel sheathing

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Abstract
The cold-formed steel (CFS) framed shear wall with sheet steel sheathing is a code approved lateral force resisting system in low-rise residential and commercial buildings in North America. Current codes for CFS framed shear walls with sheet steel sheathing, including the analytical method and the tabulated values, supports the frame member' thickness up to 54mil (1.372mm). This paper presents a research project aimed at improving the shear capacity for those shear walls by two methods: (1) thicker studs, (2) more field screws. Eight full-scale shear wall specimens are tested, including two different frame configurations and two test protocols. For each shear wall configuration, two specimens are tested. The results show that the CFS framed shear walls with sheet steel sheathing can significantly improve the shear capacity while using thicker studs and more field screws. In addition, the applicability of design provisions in AISI standards for higher strength CFS framed shear walls with sheet steel sheathing is analyzed.

1. Introduction

Cold-formed steel (CFS) shear wall is an important lateral resisting element in light-framed cold-formed steel structures. In North America, AISI S240 (2015) and AISI S400 (2015) provide the nominal strength of CFS shear wall for wind load and seismic load respectively, focusing on low-rise and mid-rise CFS constructions. The CFS shear wall configuration in AISI standards is a CFS framed shear wall with sheathing on one side, including CFS frame and various sheathings. The main sheathings contain plywood, oriented strand board and steel sheet. Among them, the CFS shear walls with sheet steel sheathing (CFS-SSSW) has a wide application prospects because of its non-combustibility. Based on researches carried out by Serrette (1997&2002), Balh (2011), Yu (2011), Yu and Chen (2011), Yanagi and Yu (2014), design provisions which have a maximum thickness limit (sheathing 33mil (0.838mm) and frame member 54mil (1.372mm)) for CFS-SSSW are formed. However, the nominal strength of CFS-SSSW in design provisions is difficult to meet the requirements of mid-rise and high-rise CFS constructions.

In recent years, some scholars conducted researches to improve the shear capacity of CFS shear walls by changing the shear wall configuration, like sheathing materials, sheathing on double

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sides, connection types and so on. For example, Fülop and Dubina (2004), Stojadinovic and Tipping (2008), Yu (2009) investigated the performance of CFS shear walls with corrugated steel sheathing. The research found that the CFS shear wall corrugated steel sheet sheathing had considerably higher in-plane strength and stiffness than the CFS-SSSWs, but demonstrated low ductility. Zhang and Yu (2018, 2019) improved ductility of CFS shear wall corrugated steel sheet sheathing while maintaining a high level of shear strength by a method of slit openings in the field of sheathing. Some experimental studies on double-sided steel sheathed CFS shear walls were carried out by Mohebbi (2015), Niari (2015) and Attari (2016). The results found that using double-sided sheathings increases the energy dissipation, shear strength and stiffness compared to those of single-sided sheathed walls. Xie (2018) conducted an experimental investigation of CFS-SSSWs with self-piercing riveted connections instead of self-drilling screws connections. Compared to CFS-SSSWs with self-drilling screws, CFS-SSSWs with self-piercing riveted had significantly improved shear strength and stiffness. An innovative configuration for CFS-SSSWs was developed by Rogers et al. (2018). This shear walls configuration comprised a sheathing placed at the mid-line of the framing. The center-sheathed shear wall represented a new category of CFS lateral system in terms of its vastly improved resistance and ductility.

Although a lot of researchers put forward new shear wall configuration to improve the performance of CFS-SSSWs. However, these shear walls are lack of systematic data and have not been included in the AISI standards, so the shear walls cannot be directly adopted by structure designers and engineers. Therefore, there is a demand to produce design provisions to support higher strength CFS-SSSWs in AISI standards. The objectives of this project are to achieve higher shear strength of the wall assembly, focusing on using thicker frame members (1.727 mm) and more sheathing screws (50.8mm), and expand the applicable range of design provisions for CFS-SSSWs. This paper presents a test study to provide the recommended nominal shear strength of CFS-SSSWs and analyzes the applicability of design provisions in AISI standards.

2. Test program

The test program was recently conducted at University of North Texas, including a total of eight shear wall tests. The shear wall specimens contained two wall configuration and two test protocols. For each wall configuration and test protocol, two identical shear wall specimens were tested.

2.1 Specimens details

The detailed information of the test specimens is summarized below in Table 1. All wall specimens were 1.22 m wide and 2.44 m high. A single stud was used for the interior stud and back-to-back studs were used for the boundary stud. Fig. 1 shows the dimensions and layout of the test specimens in the cyclic tests. For the monotonic tests, a hold-down was installed on the uplift side of the boundary studs. For the cyclic tests, two hold-downs were installed on both sides of the boundary studs.

2.2 Material properties

Coupon tests were conducted to obtain the material properties following ASTM A370 (2019). Three steel samples were randomly selected from each member, and the mean values of the material properties are given in Table 2.

Table 1: Test Matrix

Group No.	Test Label	Number of Blocking	Test Method	Number of Hold-Down
1	SW-N-M1	No	M	1
	SW-N-M2		M	1
2	SW-N-C2	No	C	2
	SW-N-C2		C	2
3	SW-W-M1	1	M	1
	SW-W-M2		M	1
4	SW-W-C1	1	C	2
	SW-W-C2		C	2

1. The overall dimensions, frame members, sheathing members and screws spacing kept the same for all test specimens.
2. “SW”-shear wall, “N”-shear wall without blocking, “W”-shear wall with blocking, “M”-monotonic loading, “C”-cyclic loading.
3. Stud: 350S162-68 (SSMA 2019), Track: 350T125-68 (SSMA 2019), Sheathing: 33mil (0.838 mm) thick standard steel sheet.
4. Boundary studs (back-to-back connection): No. 10-16×1in.(25.4mm) hex washer head self-drilling screws.
5. Stud-to-track and frame-to-sheathing connection: No. 10×3/4in.(19.1mm) modified truss head self-drilling screws.
6. Hold-down: Simpson Strong-Tie S/HD15S with No. 10-16×1in.(25.4mm) hex washer head self-drilling screws.
7. Screw spacing (edge/field): 2in. (50.8mm) / 12in. (304.8mm).

Table 2: Material properties

Sample	Uncoated thickness (mm)	Yield stress, F_y (Mpa)	Tensile strength, F_u (Mpa)	F_u / F_y	Elongation of 50.8-mm gauge length (%)
33mil steel sheet	0.886	300.6	374.6	1.25	37.5
68mil stud	1.772	406.6	522.0	1.28	29.0
68mil track	1.767	380.5	530.7	1.39	54.5

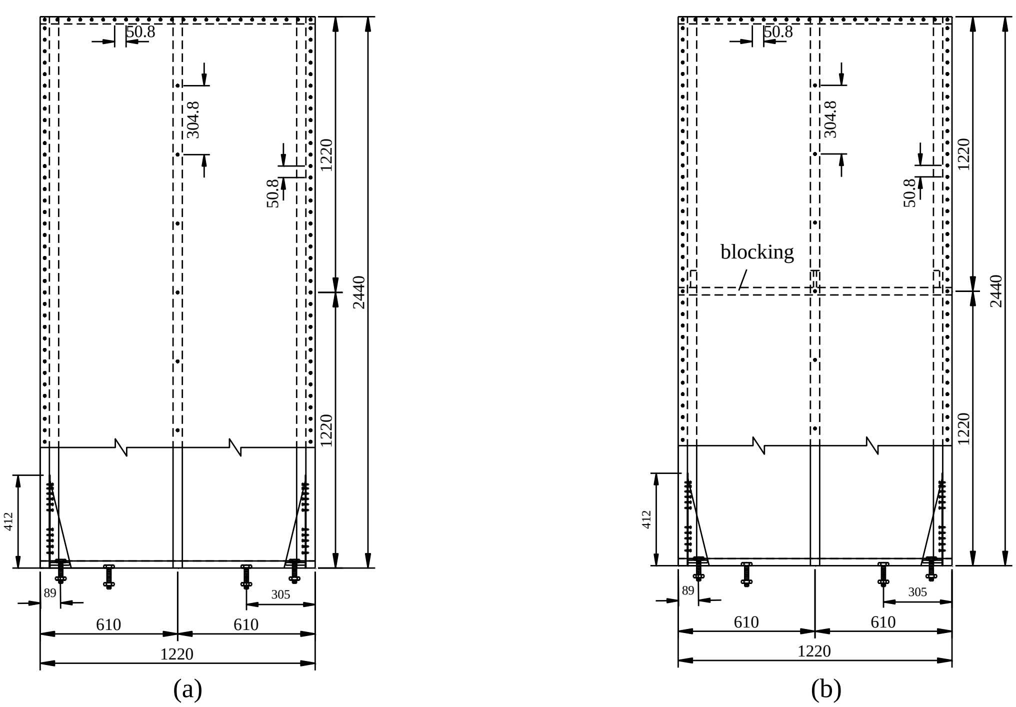


Figure 1: Shear walls configurations(mm), (a) Group 2; (b) Group 4

2.3 Test setup

The schematic diagram of the test setup, including test frame, base beam, hydraulic actuator, load cell, load beam, lateral supports, position of five displacement sensors (DP), is shown in Fig. 2. A 156kN hydraulic actuator and an 89kN load cell were utilized for these tests. The load cell was placed to pin-connect the actuator shaft and the load beam. The lateral force was applied horizontally through the load beam to the top of the shear wall specimen. No.12-14×1in. (25.4mm) hex washer head self-drilling screws were used to fix the load beam to the top track of shear wall. 5/8×5in. (127mm) G8 hold-down bolts and 5/8×2in. (50.8mm) G8 shear bolts were used to fix the bottom of the shear wall specimen to the base beam. The lateral supports were placed on both sides of shear wall to prevent out-of-plane movements. Five displacement sensors were employed to measure the deformation of the walls. DS1 was installed at the top of test frame to measure the horizontal displacement. DS2&DS3 and DS4&DS5 were used to measure horizontal displacements and the vertical displacements at the bottoms of the two boundary studs respectively.

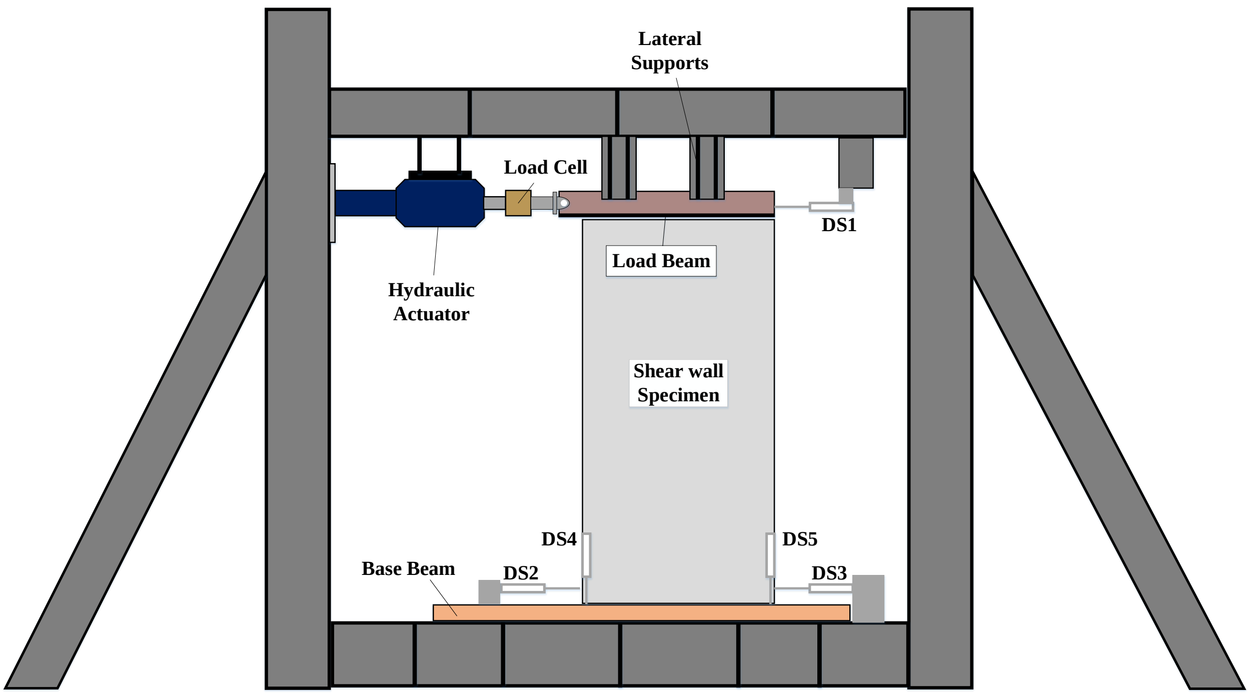


Figure 2: Test setup

2.4 Test protocols

In the monotonic tests, the top of the shear wall moved with a rate of 0.19 mm/s until the wall failed. The cyclic tests used the CUREE protocol in accordance with ASTM E2126 (2019). The CUREE test protocol includes 40 cycles, and this test protocol added 3 more cycles in order to investigate the post buckling behavior of the wall, as shown in Figure 3. The displacement amplitude was consistent for all specimens (Δ=57.2 mm).

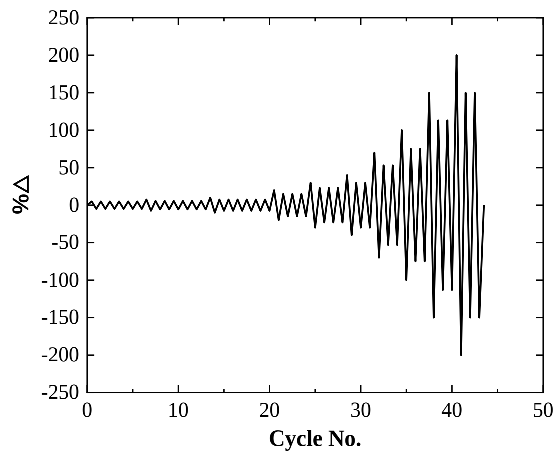


Figure 3: The CUREE test protocol

3. Test observations and results

3.1 Experimental results

Table 3 presents the experimental results for each specimen. In this study, the experimental results of the specimens were obtained by the Equivalent Energy Elastic Plastic (EEEP) method (1989). Peak load, $P_{\max}$, is the maximum horizontal force in the test curves. The displacement, drift ratio at peak point and yield load, P_{y}, are also listed. Nominal shear strength is given based on the 2.5% drift ratio limit in ASCE 7(2016). The peak load of specimens in each Group are within allowance stipulated in ASTM E564 (2018) and ASTM E2126 (2019).

Table 3: Experimental results for each specimen

Group No.	Test label	Peak load, $P_{\max}$ (kN)	Displacement at $P_{\max}$, $\Delta_{\max}$ (mm)	Drift ratio at $P_{\max}$, (%)	Yield load, P_y (kN)	Nominal shear strength, (kN/m)
1	SW-N-M1	36.62	93.14	3.8	33.03	28.13
	SW-N-M2	36.91	78.87	3.2	33.12	28.76
	Average	36.77	86.01	3.5	33.08	28.45
2	SW-N-C1	36.72	70.27	2.9	34.30	29.90
	SW-N-C2	37.68	74.43	3.1	34.54	30.20
	Average	37.20	72.35	3.0	34.42	30.06
3	SW-W-M1	47.81	97.37	4.0	42.59	35.84
	SW-W-M2	42.82	91.15	3.7	39.22	33.90
	Average	45.32	94.26	3.9	40.91	34.88
4	SW-W-C1	43.75	78.00	3.2	39.81	34.13
	SW-W-C2	44.08	76.96	3.2	40.26	34.40
	Average	43.92	77.48	3.2	40.04	34.27

3.2 Observed wall performance and discussion

The observed failure phenomenon in eight tests included: 1) sheet buckling, 2) screws pulling-over, 3) screws pulling-out, 4) buckling on boundary studs, 5) sheet tearing around screws, 6) buckling on interior studs.

3.2.1 Shear walls under monotonic loading

Specimens in Group 1 were shear walls without blocking under monotonic loading. The failure phenomenon of SW-N-M2 was similar to SW-N-M1. The main sheet buckling waves along wall diagonal direction appeared firstly (Fig. 4a). Then, the local buckling on the upper of boundary studs at compression side also emerged. As the displacement increased, the test reached the peak load and the sheet teared around sheathing screws at the upper compression corner (Fig. 4b). Moreover, more serious buckling on boundary studs at the compression side was observed (Fig. 4c). Different from SW-N-M1, screw pulling-over at the middle of the sheet was noticed for SW-N-M2 (Fig. 4d&4e). In addition, the damage on the boundary studs of SW-N-M2 was serious than SW-N-M1 (Fig. 4f). Fig. 4 shows the failure phenomenon. The failure mode of Group 1 specimens was the shear failure of the sheathing screw connections and buckling on boundary studs.

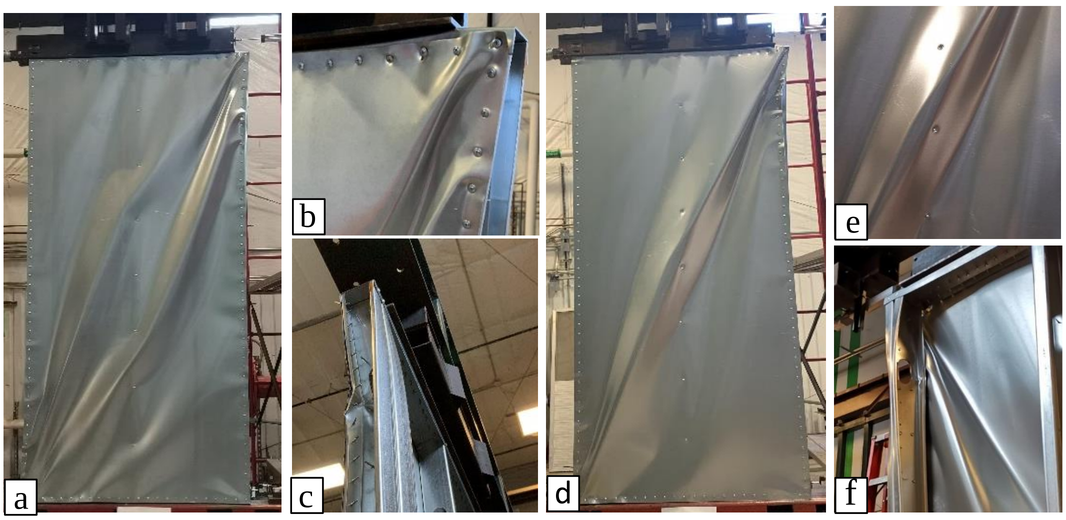


Figure 4: Failure phenomenon of Group 1 shear walls, (a) Sheet buckling at about peak point; (b) Sheet tearing; (c) Buckling on boundary studs; (d) Sheet buckling of SW-N-M2 and screws pulling-over; (e) Screws pulling-over; (f) Buckling on boundary studs of SW-N-M2

Specimens in Group 3 were shear walls with blocking under monotonic loading. SW-W-M1&M2 had blocking installed at the mid-height of the wall, 0.838 mm wide strap was installed to the blocking on each side of the wall with screw spacing 76.2 mm. Same with Group 1 shear walls, the main sheet buckling waves along wall diagonal direction also appeared firstly (Fig. 5a). However, the main failure mode of this Group shear walls was edge tearing of the bottom of sheathing at the tensile side (Fig. 5b). Buckling of the flange of the boundary studs (Fig. 5c) and the middle of the interior stud (Fig. 5d) was also noticed due to the constraint of the blocking. In addition, screws pulling-out also emerged in SW-W-M2 test (Fig. 5e). Fig. 5 shows the failure phenomenon.

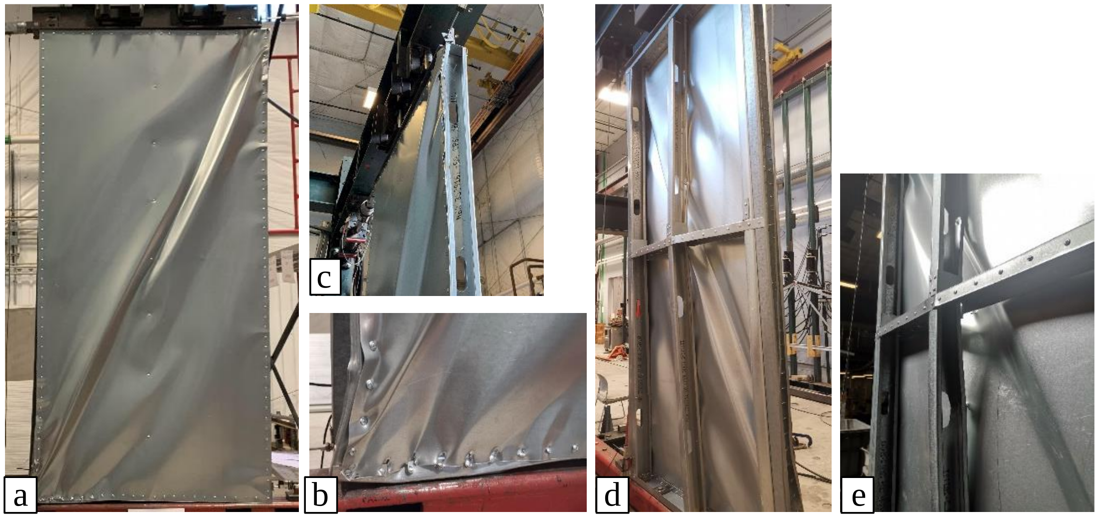


Figure 5: Failure phenomenon of Group 3 shear walls, (a) Sheet buckling at about peak point; (b) Sheet tearing; (c) Buckling on boundary studs; (d) Buckling on interior stud; (e) Screws pulling-out

The test curves of Group 1&3 specimens are shown in Fig. 6. CFS-SSSWs with blocking demonstrated the higher shear strength. The peak load and yield load of Group 1 shear walls were 36.77 kN and 33.08 kN averagely while the two values of Group 3 shear walls were 45.32 kN and 40.91 kN averagely. In the monotonic tests, the yield load and the peak load of CFS-SSSWs with blocking was improved by 23.3%, 23.67% respectively compared with CFS-SSSWs without blocking. For 1.727 mm-frame CFS-SSSWs under monotonic loading, installing blocking and straps to shear wall can improve the shear capacity.

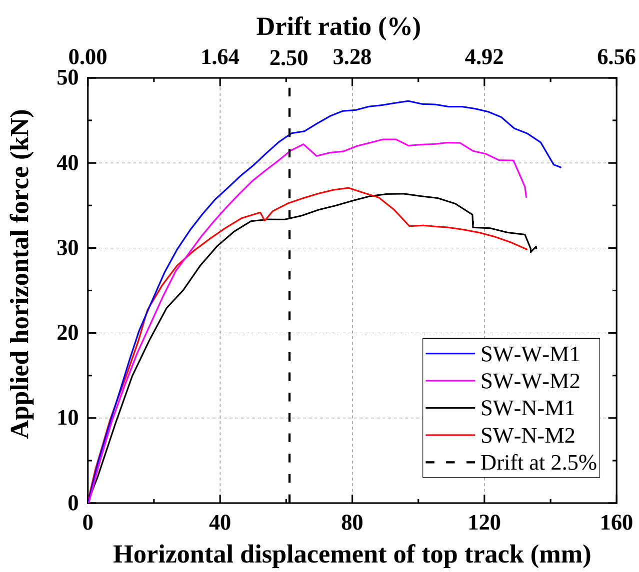


Figure 6: Test curves of shear walls under monotonic loading

The recommended nominal shear strength values of CFS-SSSWs for wind load are given in Table 4, along with the nominal shear strength in AISI S240 (2015). Compared to the 1.372 mm-frame CFS-SSSW obtained from AISI S240 (2015), the shear strength was improved by 4.3% for Group 1 shear walls, and 14.7% for Group 3 shear walls. Therefore, by increasing the thickness of frame members, the shear strength of the shear wall without blocking can be slightly improved and the shear strength of the shear wall with blocking can be significantly improved for wind load.

Table 4: Nominal shear strength for wind load (kN/m)

Assembly description	Aspect ratio (h:w)	Screws spacing: 50.8-mm / 304.8-mm	
		No blocking	Blocking
1.727mm studs, tracks 0.838mm sheet	2:1	28.45	34.88
1.372mm studs, tracks 0.838mm sheet (AISI S240)	2:1	27.27	30.41

3.2.2 Shear walls under cyclic loading

Specimens in Group 2 were shear walls without blocking under cyclic loading. The failure phenomenon of SW-N-C1 was the same as SW-N-C2's. The main failure mode of the Group 2 shear walls was the buckling on boundary studs and sheet tearing at corner screws. The deformation of the wall was not obvious at the beginning of loading. A major sheet buckling waves along diagonal directions started appearing at the 28^{th} loading cycle (Fig. 7a). In the 38^{th} cycle, sheet tearing around screws on the top of sheathing was noticed (Fig. 7b). Additionally, the screws pulled over at the middle of the sheet (Fig. 7c&7d) and distortional buckling on the upper boundary studs (Fig. 7e) were also observed. Compared with Group 1 shear walls, the failure of screw pulling-over was more obvious. The failure phenomenon is shown in Fig. 7 and the hysteresis response curves are shown in Fig. 8.

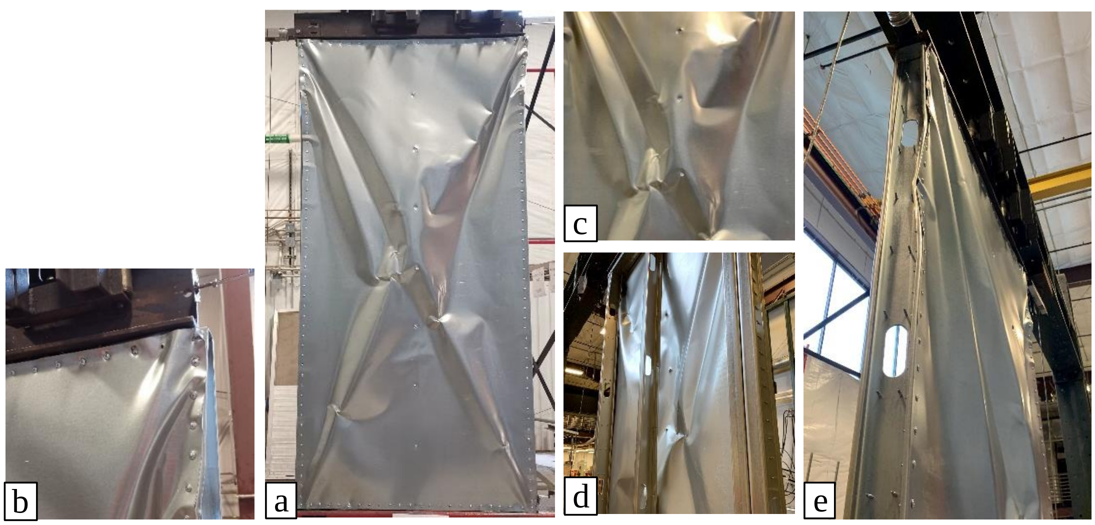


Figure 7: Failure phenomenon of Group 2 shear walls, (a) Sheet buckling after loading; (b) Sheet tearing; (c) Screws pulling-over; (d) Rear view of screws pulling-over; (e) Buckling on boundary studs

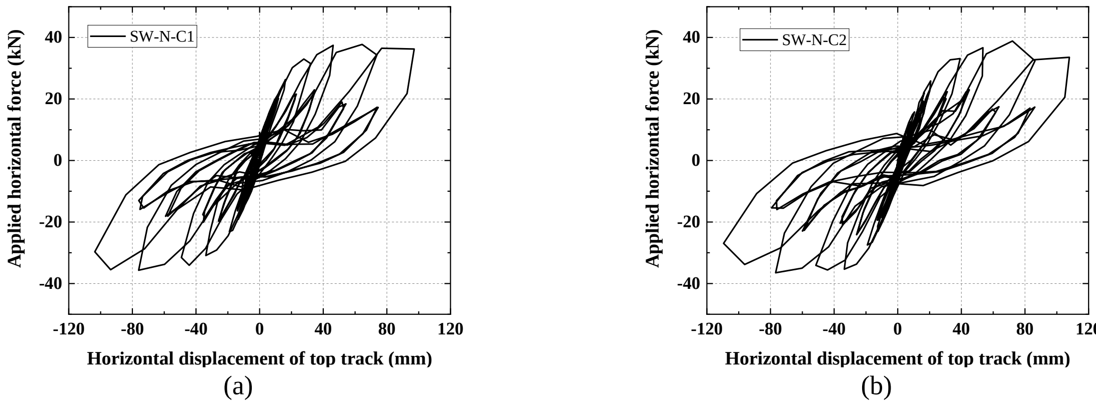


Figure 8: Hysteresis response curves, (a) SW-N-C1; (b)SW-N-C2

Specimens in Group 4 were shear walls had the same blocking as Group 3 shear walls installed but were tested under the cyclic loading. The behaviors of the two shear walls SW-W-C1&C2 were similar with the shear walls in Group 2. The failure phenomenon of SW-W-C1 specimen was as follows: The main sheet buckling waves along diagonal directions appeared firstly at the 25th loading cycle. The screws pulling-over at the middle of the sheet occurred at the 35th loading cycle (Fig. 10a). Slight sheet tearing at bottom corner screws was the dominated failure mode of this specimen that started at the 38th loading cycle. As the loading cycle increased, more severe sheet tearing at sheathing screws along the bottom track (Fig. 10b) and sheet buckling (Fig. 10c) emerged. Meanwhile, the sheet buckling also caused slight distortion on the flanges of the boundary studs (Fig. 10d). Compared with SW-W-C2, screw pulling-over at the middle of the sheet was not noticed for SW-W-C1. Fig. 9 shows the failure phenomenon. The main failure mode was sheet buckling and sheet tearing around sheathing screws. The hysteresis response curves are shown in Fig. 10.

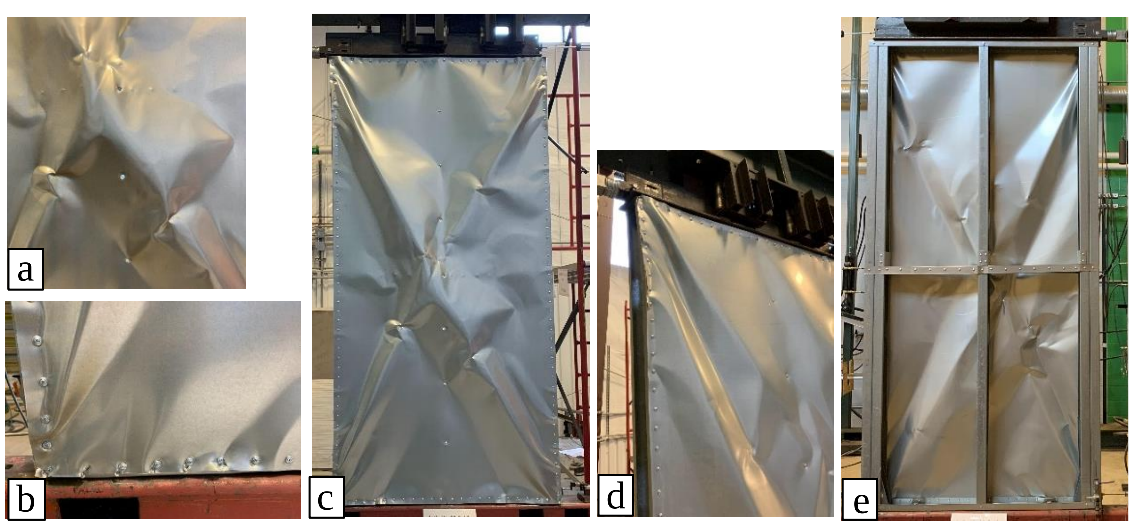
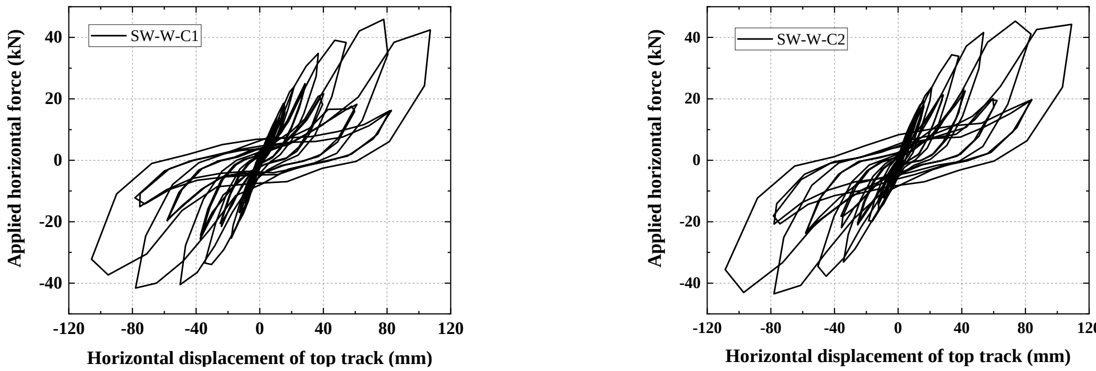


Figure 9: Failure phenomenon of Group 4 shear walls, (a) Screws pulling-over; (b) Sheet tearing; (c) Sheet buckling; (d) Buckling on boundary studs; (e) Sheet buckling of SW-W-C2



(a) (b)

Figure 10: Hysteresis response curves, (a)SW-W-C1; (b)SW-W-C2

Fig. 11 compares the envelope curves of shear walls in Group 2&4. Same with shear walls under monotonic loading, the CFS-SSSWs with blocking demonstrated the higher shear strength under cyclic loading. The peak load and yield load of Group 2 shear walls are 37.20 kN and 34.42 kN averagely while the two values of Group 4 shear walls are 43.92 kN and 40.04 kN averagely. In the cyclic tests, the yield load and the peak load of CFS-SSSWs with blocking was improved by 16.3%, 18.1% respectively compared with CFS-SSSWs without blocking. For 1.727 mm-frame CFS-SSSWs under cyclic loading, installing blocking and straps to shear walls can improve the shear capacity.

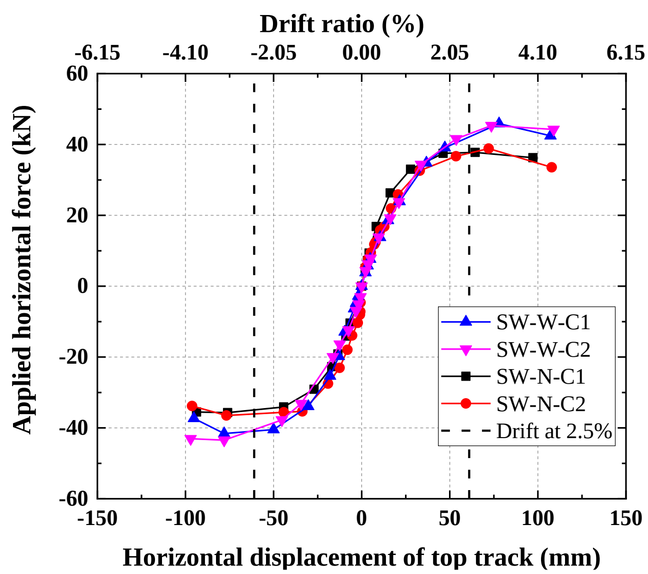


Figure 11: Comparison of envelope curves

The recommended nominal shear strength values of CFS-SSSWs for seismic load are given in Table 5, along with the nominal shear strength in AISI S400 (2015). Compared to the 1.372mm-frame CFS-SSSW obtained from AISI S400 (2015), the shear strength was improved by 10.2% for Group 2 shear walls, and 12.7% for Group 4 shear walls. Therefore, by increasing the thickness of frame members, the shear strength of the shear wall can be improved for seismic load.

Table 5: Nominal shear strength for seismic load (kN/m)

Assembly description	Aspect ratio (h:w)	Screws spacing: 50.8-mm / 304.8-mm	
		No blocking	Blocking
1.727mm studs, tracks 0.838mm sheet	2:1	30.06	34.27
1.372mm studs, tracks 0.838mm sheet (AISI S400)	2:1	27.27	30.41

4. Comparison of experimental results with AISI standards provisions

In AISI S240 (2015) and AISI S400 (2015), the effective strip method is an alternative method to determine the nominal shear strength of CFS-SSSWs, which was proposed by Yanagi and Yu (2014) based on analysis of a total of 142 shear wall test data. Structure designers were permitted to use the effective strip method within the designation thickness of frame member: 33mils (0.838mm) to 54mils (1.37mm). However, the thickness of higher strength CFS-SSSWs in this study is beyond the application scope of the effective strip method. Comparisons of nominal shear strength between the test values and the predicted values obtained from the effective strip method are provided in Table 6. V_{test} is the test values and V_{equation} is the predicted values obtained from the effective strip method. The average test-to-equation ratio of Group 1-4 are 1.12, 1.18, 1.37, and 1.35 respectively.

Table 6: The statistical analysis of the test-to-equation ratio

Group No.	Test label	V_{test}	V_{equation}	V_{test} / V_{equation}	Average test-to-equation ratio
1	SW-N-M1	34.32	31.14	1.10	1.12
	SW-N-M2	35.09	31.14	1.13	
2	SW-N-C1	36.48	31.14	1.17	1.18
	SW-N-C2	36.85	31.14	1.18	
3	SW-W-M1	43.73	31.14	1.40	1.37
	SW-W-M2	41.36	31.14	1.33	
4	SW-W-C1	41.64	31.14	1.34	1.35
	SW-W-C2	41.97	31.14	1.35	

For a more intuitive comparison, the test-to-equation ratio of 142 tests data and this research data was shown in Fig. 12. As can be seen, the experimental data points of Group 1 and Group 2 shear walls are right above the horizontal line, which indicates that the effective strip method is applicable to calculate the shear strength of higher strength CFS-SSSWs without blocking. As for Group 3-4 shear walls, the experimental data points are far above the horizontal line and the 142 tests data points, which indicates that the effective strip method is too conservative to calculate the shear strength of higher strength CFS-SSSWs with blocking. Therefore, it is suggested that the beneficial effect of the blocking should be considered in the effective strip method.

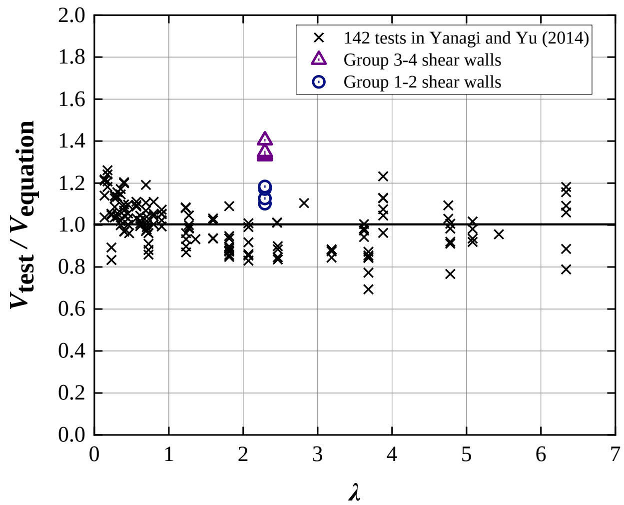


Figure 12: Comparison of the nominal shear strength between equation and test data

5. Conclusions

This paper presented the experimental study on the higher strength CFS shear walls with sheet steel sheathing. Thicker frame members (1.727 mm) and more sheathing screws (50.8mm) were adopted for CFS-SSSWs in this study. Eight specimens were designed and tested under monotonic and cyclic loading. The main failure mode for the higher strength CFS-SSSWs without blocking was buckling on the boundary studs and sheet tearing around screws, while for the shear walls with blocking was sheet tearing around screws and sheet buckling. Based on the experimental results, the recommended nominal shear strength of higher strength CFS shear walls with sheet steel sheathing (aspect ratio: 2:1, the thickness of frame member: 1.727mm, the thickness of sheet: 0.838mm and sheathing screw size: No. 10) for wind load and seismic load were provided. The results showed installing blocking to higher strength CFS-SSSWs could improve the shear strength. The applicability of design provisions in AISI standards was analyzed. The comparison of experimental results with the effective strip method showed the effective strip method provided good prediction of the shear strength of steel sheet shear walls using 1.73mm CFS framing members. It is suggested that the beneficial effect of the blocking should be considered in the effective strip method.

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