Engineering Structures 32 (2010) 1522–1529

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

Shear resistance of cold-formed steel framed shear walls with 0.686 mm, 0.762 mm, and 0.838 mm steel sheet sheathing

Cheng Yu *

Department of Engineering Technology, University of North Texas, Denton, TX 76207, United States

ARTICLE INFO

Article history:
Received 15 June 2009
Received in revised form
25 January 2010
Accepted 26 January 2010
Available online 19 February 2010

Keywords:
Cold-formed steel
Shear wall
Shear strength
Monotonic test
Cyclic test

ABSTRACT

The cold-formed steel (CFS) framed shear walls using steel sheet sheathing is a code approved lateral force resisting system in residential and low-rise commercial buildings in the United States. The current design specifications in the US provide nominal shear strength for a limited range of CFS shear wall configurations in terms of sheathing thickness and wall aspect ratio. This paper presents a research project aimed to add shear strength values for 0.686 mm, 0.762 mm, and 0.838 mm steel sheet sheathed CFS shear walls with aspect ratios of 2:1 or 4:1. The project consisted of two series of tests. The first series was monotonic tests for determining the nominal shear strength for wind loads. The second series was the cyclic tests using CUREE protocol to obtain the shear strength for seismic loads. The studied shear walls used 0.838 mm or 1.092 mm thick CFS framing members. The sheathing was only attached to one side of the frame. No. 8 modified truss head self-drilling screws were used for all the test specimens. This paper details the testing methods, specimen configurations, and the test results.

1. Introduction

Cold-formed steel (CFS) framed shear walls with steel sheet sheathing is a practical lateral force resisting system in buildings. The current American Iron and Steel Institute (AISI) Standard for Cold-Formed Steel Framing—Lateral Design 2007 Edition [1] provides nominal shear strength for a limited range of sheet steel sheathed CFS shear wall configurations in terms of the sheathing thickness and the wall aspect ratio (height divided by width).

AISI Lateral Design Standard [1] only provides nominal shear strength for CFS shear walls using 0.457 mm and 0.686 mm steel sheet sheathing with a limited range of wall aspect ratio. The ratio is up to 2:1 for walls with 0.457 mm steel sheathing, and up to 4:1 for walls using 0.686 mm steel sheathing. As the CFS applications become more popular in the mid-rise commercial buildings and multi-family homes, a wider range options of the wall aspect ratio and the steel sheathing thickness are desired by engineers. A test program was therefore conducted at the University of North Texas (UNT) to determine the shear strengths of CFS framed shear walls using 0.762 mm or 0.838 mm steel sheet sheathing with an aspect ratio of 4:1 or 2:1, and 0.686 mm steel sheet sheathing with an aspect ratio of 2:1.

The published shear strengths in AISI Lateral Design Standard [1] are based on the research conducted by Serrette et al. [2] who focused on relatively thin steel sheathing. The test protocol used by Serrette et al. [2] for monotonic tests was similar to ASTM E564 "Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings" [3] except that the incremental loading procedure in Serrette et al.'s work was based on the lateral displacement while ASTM E564 [3] used the estimated peak load to determine the load increments. For the cyclic tests, the sequential phase displacement protocol was used in Serrette et al.'s tests. Serrette et al. [2] discovered that for shear walls with fasteners spaced further apart, sheathing fastener pullout and significant deformation of sheathing were observed. For the shear walls with less fasteners spacing, the failure mode was buckling in the sheathing as well as in the studs.

Similar to Serrette et al. [2]'s research, the test program presented here also consisted of two series of tests: (1) monotonic tests for determining the nominal shear strength for wind loads and (2) cyclic tests for determining the shear strength for seismic loads.

2. Test program

2.1. Test setup

The shear wall tests were performed on a 4.88 m span, 3.66 m high adaptable structural steel testing frame. Fig. 1 shows the front view of the testing frame with a 2.44 m × 1.22 m CFS shear wall. Fig. 2 shows the back view of the test specimen. The wall specimen was bolted to the testing frame and loaded horizontally at the top via a load beam made of a hot-rolled steel T shape. The load beam

* Tel.: +1 940 891 6891; fax: +1 940 565 2666.
E-mail address: cyu@unt.edu.

0141-0296/$ – see front matter © 2010 Elsevier Ltd. All rights reserved.
doi:10.1016/j.engstruct.2010.01.029

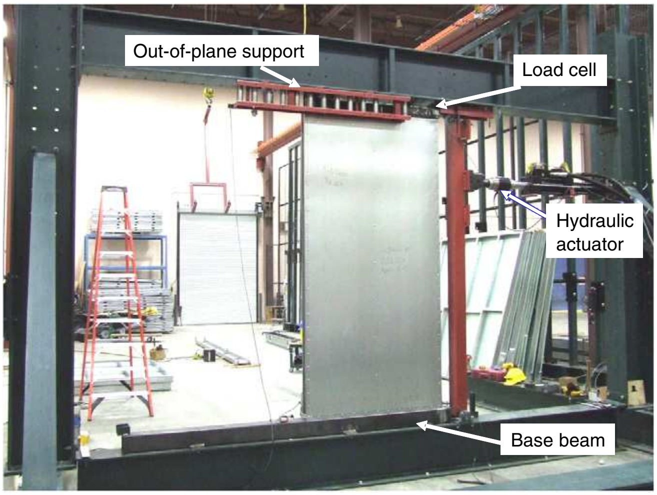


Fig. 1. Front view of the test setup.

Fig. 2. Back view of the test setup.

was attached to the top track of the wall specimen by No. 12 hex washer head (HWH) self-drilling screws placed every 76 mm on center. Fig. 3 shows a close-up view of the wall top. Steel rollers were used on both sides of the load beam to restrict the out-of-plane displacement of the wall during the test. The load beam was wider than the width of the shear walls so that the rollers would not touch the steel sheet sheathing during the tests.

The testing frame was equipped with one 116 KN hydraulic actuator with a ± 127 mm stroke. The lateral force was applied to the load beam via a lever made of structural steel tubing. A 44 KN universal compression/tension load cell was used to connect the top of the lever to the load beam. Five position transducers were employed to measure the horizontal displacements of the wall at top and bottom and the vertical displacements of the two boundary studs.

2.2. Test methods

Both monotonic and cyclic tests were conducted in a displacement control mode. The procedure of the monotonic tests conformed to the ASTM E564 [3]. A preload of approximately 10% of

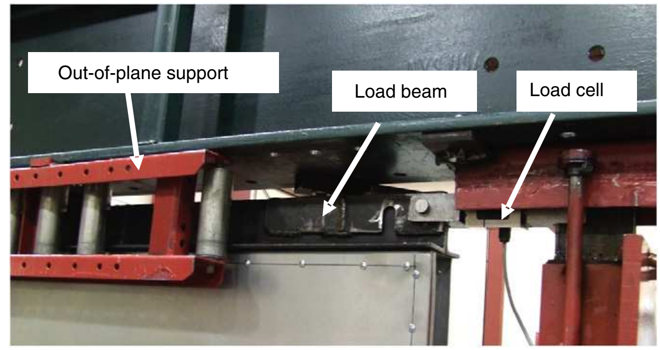


Fig. 3. Close-up of the top of the wall specimen.

the estimated ultimate load was applied first to the specimen and held for 5 min to seat all connections. After the preload was removed, the incremental loading procedure started until the failure occurred, the load increment was set to 1/3 of the estimated peak load.

The cyclic tests used the CUREE (Consortium of Universities for Research in Earthquake Engineering) protocol in accordance with ICC-ES AC130 "Acceptance Criteria for prefabricated Wood Shear Panels" [4]. A constant frequency of 0.2 Hz for was adopted for the CUREE cyclic loading history in this research.

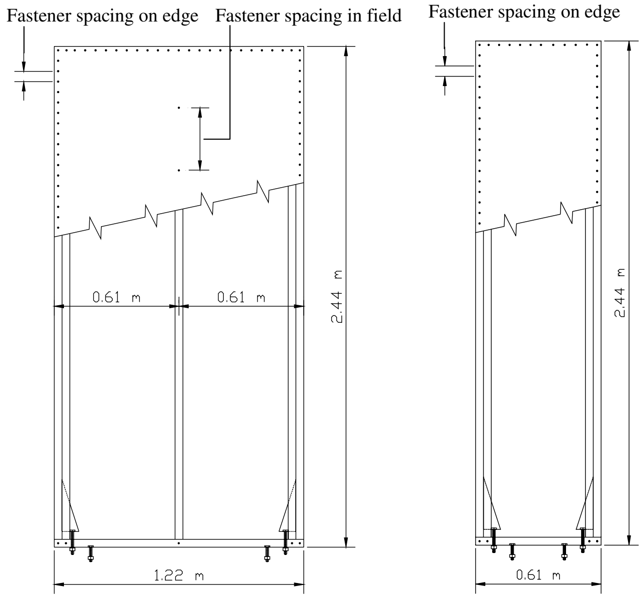
For each specimen configuration, two identical tests were conducted. For the monotonic tests a third specimen would be tested if the shear strength or stiffness of the second specimen tests is not within 15% of the result of the first specimen tested, which was in agreement with ASTM E564 [3]. For the cyclic tests, a third identical specimen would be tested if the difference between the ultimate test loads of the first two specimens is more than 10% apart, this arrangement conformed to ICC-ES AC130 [4].

2.3. Test specimens

Two wall aspect ratios were investigated in this test program: 2.44 m wide × 1.22 m high (aspect ratio 2:1) and 2.44 m wide × 0.61 m (aspect ratio 4:1). The test parameters also included three steel sheathing thicknesses: 0.686 mm, 0.762 mm, and 0.838 mm and three fastener spacing configurations on the panel edges: 152 mm, 102 mm, and 51 mm. The fastener spacing in the field of the sheathing was 305 mm for all shear walls. No. 8 13 mm long modified truss head self-drilling screws were used for both framing and sheathing. The sheathing was attached to only one side of the shear wall.

Standard SSMA (Steel Stud Manufacturers Association) structural studs and tracks were used for the framing. SSMA 350S162-43 studs and 350S150-43 tracks were used for shear walls with 0.838 mm and 0.762 mm sheathing. SSMA 350S162-33 studs and 350S150-33 tracks were used for shear walls with 0.686 mm sheathing. The details of the SSMA standard framing members refer to the SSMA Product Catalog [5].

Fig. 4 shows the typical framing details and screw spacing arrangement for shear walls tested in cyclic loading. The studs were placed at 610 mm on center. Double C-shaped studs (back-to-back) were used at the boundaries, and a single C-shaped stud was used at the center of the shear wall. The webs of the double studs were attached together by two lines of No. 8 13 mm long modified truss head self-drilling screws with 152 mm spacing between two adjacent screws. For the sheathing screw arrangement at the boundary studs on both ends, a single line of screws was installed on the outer flange of studs. Test results by Serrette [2] indicated that the shear strength of the walls with steel sheet sheathing installed with a horizontal joint blocked at mid-height was higher than the walls with no horizontal joints. Therefore, the results



(a) 2.44 m × 1.22 m shear wall.

(b) 2.44 m × 0.61 m shear wall.

Fig. 4. Framing details for CFS shear walls for cyclic test.

from this test program can conservatively be used for shear walls that have sheet steel sheathing with horizontal blocked joints or vertical blocked joints.

Simpson Strong-Tie S/HD10S hold-down was used to prevent the boundary studs from being uplifted. For the monotonic tests, only one hold-down was used to attach to the tension boundary studs from inside by No. 14 25 mm long HWH self-drilling screws. For the cyclic tests, two hold-downs were used, one on each end of the wall. For all tests, the hold-down was raised 38 mm above the flange of the bottom track. The manufacturer recommends 22 mm diameter bolts for the S/HD10S hold-downs. However 13 mm diameter SAE Grade 8 anchor bolts with standard cut washers and nuts were used in this research due to the relatively low ultimate strength expected for the studied shear walls. As shown in Fig. 4, four bolts were used for each wall to anchor the hold-downs and the bottom track to the base beam.

All the cold-formed steel members in the shear walls were made of ASTM A1003 [6] Grade 33 steel. The material thicknesses of the sheathing and framing members were monitored throughout this test program. Coupon tests were performed to obtain the actual material properties.

3. Test results

3.1. Monotonic shear wall tests

A total of 30 monotonic tests were conducted in this test program. Table 1 summarizes the monotonic test results. The nominal shear strengths are calculated as the average of the peak loads of two identical tests. Fig. 5 illustrates the definitions of the notations in the test labels.

In all the tests, the in-plane shear force caused the buckling of the steel sheathing and large out-of-plane deformation of the sheathing. Fig. 6 shows the failure mode of a 2.44 m × 1.22 m wall using 0.762 mm steel sheathing with a screw spacing of 51 mm/305 mm (spacing at edges/spacing in the field). The sheathing warped with the rib of the buckling wave in the diagonal direction parallel to the tensile stress in the sheathing material. For the 2.44 m × 1.22 m walls, the back-to-back double studs at the wall boundaries were able to provide enough resistance

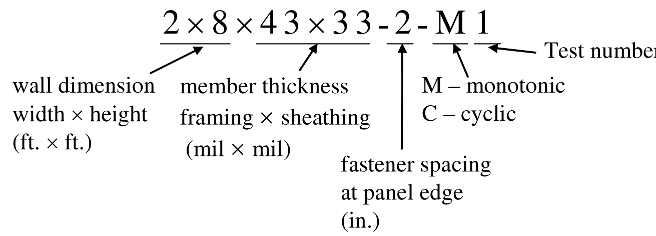


Fig. 5. Definitions of the test label.

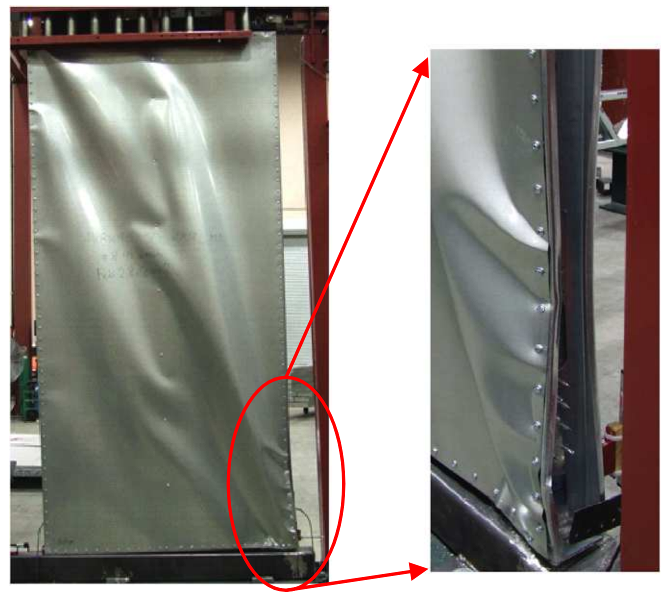


Fig. 6. Sheathing buckling and stud flange distortion on a 2.44 m × 1.22 m wall in monotonic test.

against overturning forces. For the walls with large screw spacing (e.g. 102 mm, 152 mm) at the panel edges, the failure resulted from a combination of buckling of the sheathing and pullout of screws from the boundary studs. Fig. 7 shows a typical failure mode for a 0.762 mm sheet steel walls with 152 mm/305 mm screw spacing arrangement. Fig. 8 illustrates the load vs. lateral displacement curve for the test shown in Fig. 7. For the 2.44 m × 1.22 m walls with 51 mm/305 mm screw schedule, the failure was buckling of the sheathing without screw pullout. However for walls with 51 mm/305 mm screw spacing, the distortion on the flanges of boundary studs under uplift force was observed at the end of the studs above the hold-down, as shown in Fig. 6. The unexpected flange distortion was likely caused by the bending moment exerted from the hold-down which was eccentrically loaded [7]. The distortion on the flange of tension studs was not the failure mechanism in the monotonic tests. However the damaged boundary studs may cause the collapse of the structure when the lateral load changes its direction (for instance in seismic loading condition) to develop compression forces to those boundary studs. Therefore the distortional buckling of the boundary studs caused by loading eccentricity shall be checked in shear wall design. A recommended design procedure was provided in [7].

In the monotonic tests on the 2.44 m × 0.61 m walls, it was found that the peak loads usually occurred at a drift greater than 2.5% of the wall height (61 mm) which were consistently greater than those for the 2.44 × 1.22 m shear walls. Similar to the failure modes for the 2.44 m × 1.22 m walls, a combination of sheathing buckling and screw pullout was observed for 2.44 m × 0.61 m

Table 1
Monotonic test results for CFS shear walls.

Test label	Peak load (kN/m)	Nominal shear strength (kN/m)	Drift ratio (%)	Avg. drift ratio (%)
4 × 8 × 43 × 33-6-M1	14.93	15.67	2.17	1.98
4 × 8 × 43 × 33-6-M2	16.40		1.79	
4 × 8 × 43 × 33-4-M1	17.12	17.35	1.80	2.11
4 × 8 × 43 × 33-4-M2	17.57		2.42	
4 × 8 × 43 × 33-2-M1	19.22	19.66	2.64	2.18
4 × 8 × 43 × 33-2-M2	20.08		1.72	
4 × 8 × 43 × 30-6-M1	11.69	11.59	2.62	2.57
4 × 8 × 43 × 30-6-M2	11.47		2.53	
4 × 8 × 43 × 30-4-M1	13.72	14.00	2.57	2.72
4 × 8 × 43 × 30-4-M2	14.26		2.87	
4 × 8 × 43 × 30-2-M1	15.73	15.38	3.60	3.33
4 × 8 × 43 × 30-2-M2	15.03		3.06	
4 × 8 × 33 × 27-6-M1	9.40	9.14	1.95	1.99
4 × 8 × 33 × 27-6-M2	8.86		2.03	
4 × 8 × 33 × 27-4-M1	10.00	9.98	1.98	2.19
4 × 8 × 33 × 27-4-M2	9.95		2.41	
4 × 8 × 33 × 27-2-M1	12.49	12.20	2.10	2.07
4 × 8 × 33 × 27-2-M2	11.91		2.04	
2 × 8 × 43 × 33-6-M1	15.54	14.84	3.26	2.92
2 × 8 × 43 × 33-6-M2	14.13		2.57	
2 × 8 × 43 × 33-4-M1	16.74	16.87	2.74	2.89
2 × 8 × 43 × 33-4-M2	16.99		3.03	
2 × 8 × 43 × 33-2-M1	20.23	19.86	3.49	3.33
2 × 8 × 43 × 33-2-M2	19.48		3.18	
2 × 8 × 43 × 30-6-M1	12.73	12.87	3.44	3.49
2 × 8 × 43 × 30-6-M2	13.00		3.54	
2 × 8 × 43 × 30-4-M1	13.67	13.86	3.46	3.42
2 × 8 × 43 × 30-4-M2	14.05		3.39	
2 × 8 × 43 × 30-2-M1	16.00	16.01	3.44	3.50
2 × 8 × 43 × 30-2-M2	16.02		3.57	

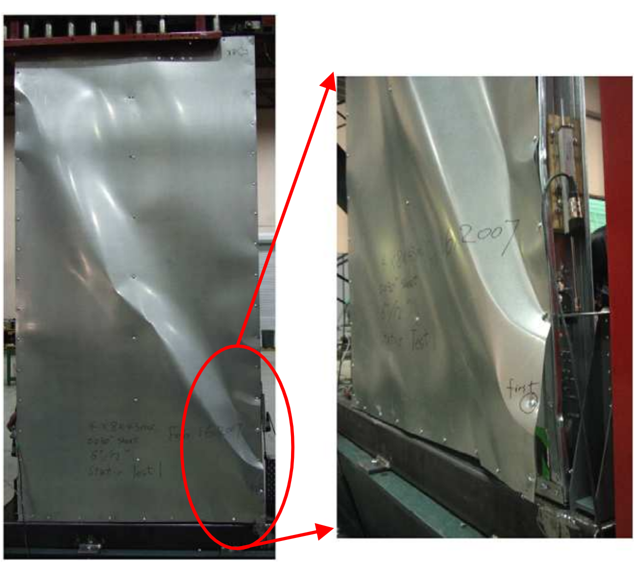


Fig. 7. Sheathing buckling and screw pullout on a 2.44 m × 1.22 m wall in monotonic test.

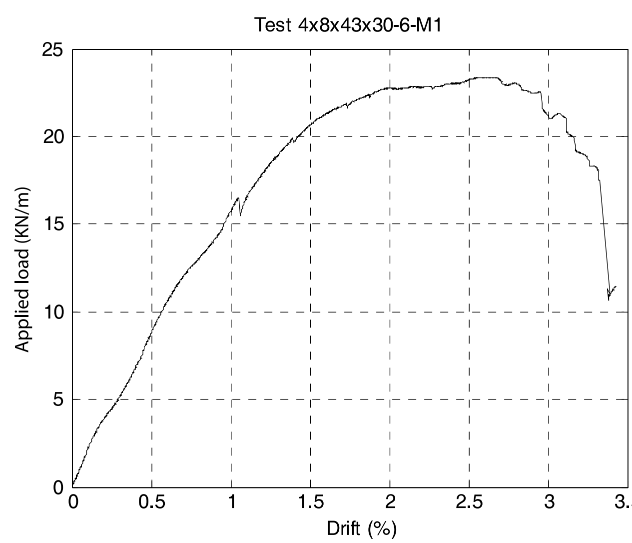


Fig. 8. Load vs. lateral displacement of wall top for Test 4 × 8 × 43 × 30-6-M1.

walls with 152 mm/305 mm or 102 mm/305 mm screw spacing arrangement. Also a combination of sheet buckling and flange distortion of the double boundary studs was observed for walls with 51 mm/305 mm screw spacing arrangement. In addition to those modes, the buckling in the web and flange of the double studs at the compression side was also observed on walls with 51 mm/305 mm screw schedule. Fig. 9 shows this boundary stud buckling failure.

The monotonic test results of this project may be conservatively compared to the actual field applications, as the hold-downs generally are installed on the boundary studs at each end of the shear wall and the hold-down on the compression side may help to reduce the actual boundary stud buckling length.

3.2. Cyclic shear wall tests

A total of 30 cyclic tests were conducted. Table 2 summarizes the test results. The peak load for each test is taken as the average of the peak loads from the positive and negative quadrants of the hysteresis curve.

For the cyclic tests on 2.44 m × 1.22 m walls, a combination of steel sheet sheathing buckling and screw pullout was observed. For walls with 152 mm/305 mm or 102 mm/305 mm screw spacing arrangement, the screw pullout occurred on the boundary studs as well as on the interior studs. For walls with 51 mm/305 mm screw spacing, the screws were primarily pulled out from the interior studs. Similar to the monotonic tests, flange distortion on the boundary studs were also observed on walls with 51 mm/305 mm screw spacing. Figs. 10 and 11 respectively show the hysteresis

Table 2
Cyclic test results for CFS shear walls.

Test label	Peak load (kN/m)	Nominal shear strength (kN/m)	Drift ratio (%)	Avg. drift ratio (%)
4 × 8 × 43 × 33-6-C1	16.24	15.95	1.72	1.70
4 × 8 × 43 × 33-6-C2	15.65		1.68	
4 × 8 × 43 × 33-4-C1	17.32	17.66	1.87	1.80
4 × 8 × 43 × 33-4-C2	17.98		1.74	
4 × 8 × 43 × 33-2-C1	20.42	19.70	1.81	1.84
4 × 8 × 43 × 33-2-C2	18.99		1.87	
4 × 8 × 43 × 30-6-C1	13.15	13.30	2.00	2.17
4 × 8 × 43 × 30-6-C2	13.44		2.35	
4 × 8 × 43 × 30-4-C1	15.19	14.80	2.06	2.09
4 × 8 × 43 × 30-4-C2	14.40		2.12	
4 × 8 × 43 × 30-2-C1	15.66	15.62	1.80	1.82
4 × 8 × 43 × 30-2-C2	15.56		1.85	
4 × 8 × 33 × 27-6-C1	9.53	9.44	1.60	1.59
4 × 8 × 33 × 27-6-C2	9.34		1.58	
4 × 8 × 33 × 27-4-C1	10.60	10.36	1.26	1.27
4 × 8 × 33 × 27-4-C2	10.13		1.27	
4 × 8 × 33 × 27-2-C1	11.70	12.33	1.77	1.86
4 × 8 × 33 × 27-2-C2	12.95		1.95	
2 × 8 × 43 × 33-6-C1	16.52	16.56	3.10	3.17
2 × 8 × 43 × 33-6-C2	16.59		3.24	
2 × 8 × 43 × 33-4-C1	18.27	18.45	3.15	3.27
2 × 8 × 43 × 33-4-C2	18.62		3.39	
2 × 8 × 43 × 33-2-C1	20.86	19.86	3.22	3.17
2 × 8 × 43 × 33-2-C2	18.86		3.11	
2 × 8 × 43 × 30-6-C1	13.37	13.49	3.13	3.26
2 × 8 × 43 × 30-6-C2	13.59		3.40	
2 × 8 × 43 × 30-4-C1	15.40	15.37	3.35	3.29
2 × 8 × 43 × 30-4-C2	15.34		3.22	
2 × 8 × 43 × 30-2-C1	17.48	17.56	3.22	3.15
2 × 8 × 43 × 30-2-C2	17.63		3.08	

Fig. 9. Stud buckling on a 2.44 m × 0.61 m wall with 51 mm/305 mm screw spacing.

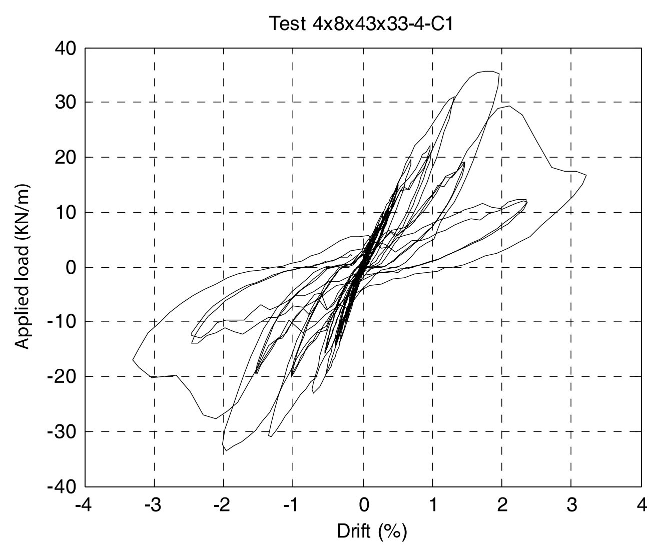


Fig. 10. Hysteresis curve for test 4 × 8 × 43 × 33-4-C1.

curve and the typical failure mode for 2.44 m × 1.22 m walls with 102 mm/305 mm screw spacing. Fig. 12 shows the observed failure mode for a 2.44 m × 1.22 m wall with 51 mm/305 mm screw spacing. The CFS walls yielded similar peak loads on both the positive and negative loading directions, and the walls were able to remain the stiffness prior to the peak load cycle. After passing the peak load cycle, both strength degradation and stiffness degradation were observed. On average, the CFS walls yielded 17% reduction in shear strength and 33% reduction in stiffness when comparing the first post-peak cycle with the peak cycle.

The 2.44 m × 0.61 m walls yielded large displacement capacities in the monotonic tests and those values were greater than the cap on the CUREE cyclic test reference displacement specified by ICC-ES AC130 [4] (2.5% of wall height). Therefore for the cyclic tests on 2.44 m × 0.61 m walls, the maximum applied displacement was not sufficient to achieve post-peak behavior in some cases. Figs. 13 and 14 show the typical hysteresis curve and failure mode for 2.44 m × 0.61 m shear walls.

3.3. Material properties

Coupon tests were conducted following ASTM A370 "Standard Test Methods and Definitions for Mechanical Testing of Steel Products" [8]. The coupon test results are summarized in Table 3.

The test results indicate that the measured uncoated thicknesses of the 0.762 mm and 0.686 mm sheathing are less than

Table 3
Coupon test results.

Member	Uncoated thickness (mm)	Yield stress, F_y (MPa)	Tensile strength, F_u (MPa)	F_y/F_u	Elongation for 51 mm gage length (%)
0.838 mm sheathing	0.909	299	371	1.24	27
0.762 mm sheathing	0.726	337	383	1.08	24
0.686 mm sheathing	0.610	347	399	1.15	21
1.092 mm stud	1.092	328	380	1.15	29
0.838 mm stud	0.838	329	384	1.17	24
1.092 mm track	1.067	297	383	1.29	25
0.838 mm track	0.838	396	463	1.17	28

Note: Steel is specified as Grade 33 for all members. The specified minimum yield stress is 228 MPa and specified minimum tensile strength is 310 MPa.

Fig. 11. Failure mode for test 4 × 8 × 43 × 33-4-C1.

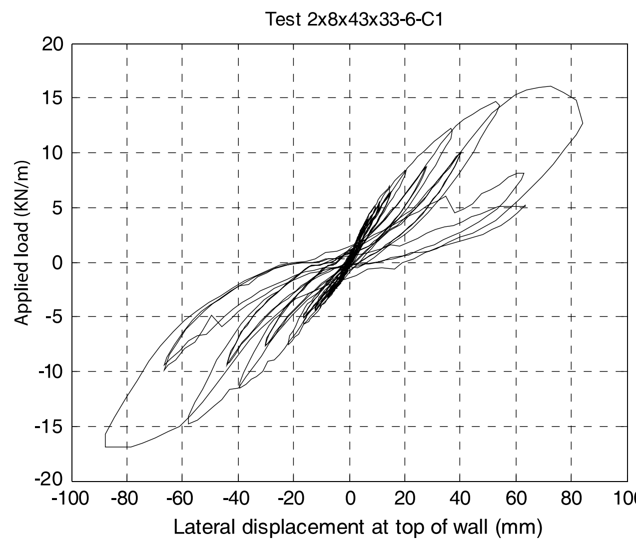


Fig. 13. Observed hysteresis curve for cyclic test 2 × 8 × 43 × 33-6-C1.

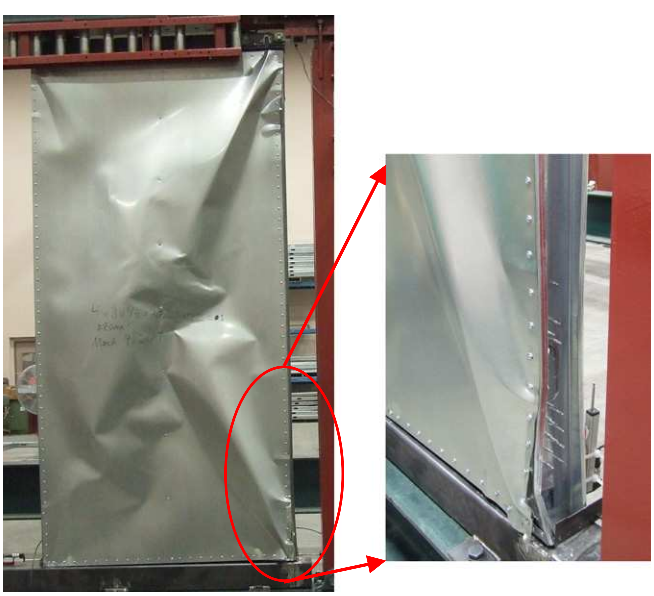


Fig. 12. Failure mode for test 4 × 8 × 43 × 30-2-C1.

Fig. 14. Failure mode for test 2 × 8 × 43 × 33-6-C1.

the required minimum base metal (i.e., uncoated) thicknesses per the AISI General Provisions (2004). All the coupons meet the minimum ductility requirements by North American Specification for Design of Cold-Formed Steel Structural Members 2007 Edition [9] which requires the tensile strength to yield strength ratio greater than 1.08, and the elongation on a 51 mm gage length higher than 10%. The material used in this project was in accordance with ASTM A1003 [6] Grade 33 Type H classification as per AISI Lateral Design Standard [1] Section 5.4.

3.4. Nominal shear strength

The nominal shear strength is determined as the average peak load of all the identical tests. The nominal shear strength for wind loads is based on monotonic test results and the nominal

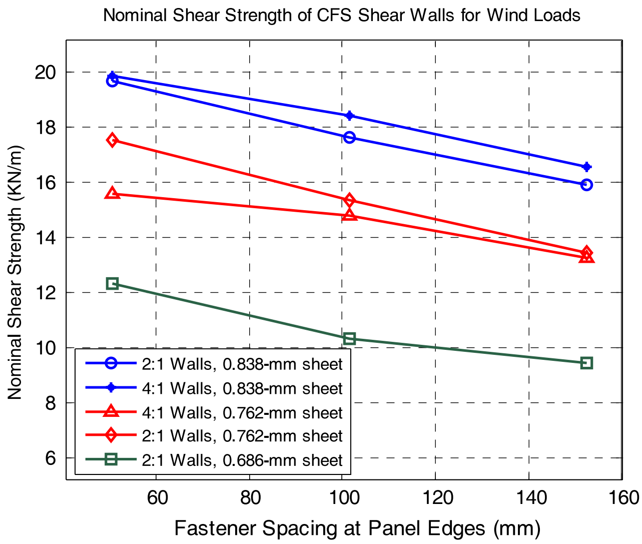


Fig. 15. Nominal strengths for wind loads vs. fastener spacing at panel edges.

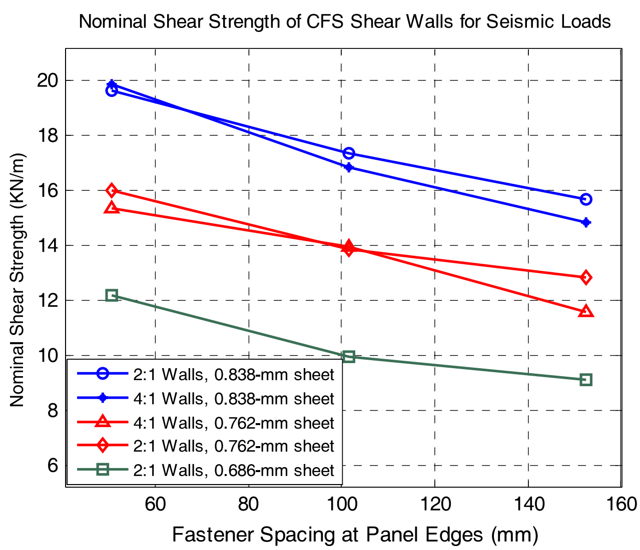


Fig. 16. Nominal strengths for seismic loads vs. fastener spacing at panel edges.

shear strength for seismic loads is obtained from the cyclic tests. Figs. 15 and 16 present the plots of the nominal strength vs. the fastener spacing at panel edges for wind loads and seismic loads respectively.

Figs. 15 and 16 indicate that a linear relationship could be assumed between the nominal shear strength and the fastener spacing at panel edges. In this test program, fastener spacing of 152 mm, 102 mm, and 51 mm were investigated, therefore the nominal strength for walls with other fastener spacing can be estimated accordingly.

Per the NASPEC (2007) [9] Chapter F, the nominal strengths obtained from experimental tests shall be adjusted (downwards only) according to the variation of the thickness and strength of controlling members between the actual values and the specified values by the manufacturers. For the tested CFS shear walls, the primary failure mechanism was the buckling of the sheathing therefore the adjustment shall be based on variation in the sheathing materials. The coupon tests indicate that the measured base metal (i.e., uncoated) thickness for 0.762 mm sheathing was less than the design thickness, 0.792 mm, and the measured base metal thickness for 0.686 mm sheathing is less than the design thickness, 0.719 mm, therefore no adjustment due to variation in the thickness is needed for 0.762 mm and 0.686 mm steel sheet walls. However for the 0.838 mm sheathing, the measured base metal thickness was greater than the design thickness (0.879 mm according to AISI Standard for Cold-Formed Steel Framing – Product Data [10]), therefore the nominal strength needs to be adjusted by a ratio of 0.966. The difference between the measured material strength and the specified value was also found by the coupon tests. To address the variation in the material strength, footnotes on the tables state the minimum material strengths required to use the tabulated values. Based on the results of this research project, thickness-adjusted nominal shear strengths for sheet steel CFS shear walls are summarized in Tables 4 and 5.

3.5. Reduction factor for slender CFS shear walls

The previous research [2] and this project have shown that CFS framed shear walls with large aspect ratios had relatively low stiffness but yielded significantly large drift capacity. The AISI Lateral Design Standard [1] permits some CFS shear walls resisting wind or seismic loads to exceed the 2:1 aspect ratio limit, but requires that the nominal shear strength be reduced by a factor of 2w/h for those assemblies with a height to width aspect ratio greater than 2:1. It also requires that the allowable strength (ASD) be determined by dividing the nominal shear strength by a safety factor of 2.5 for shear walls resisting seismic loads and 2.0 for shear walls resisting wind loads. The 2006 International Building Code IBC 2006 [11] limits story drift for seismic force resisting systems for structures 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts to 0.025 h. This story drift limit is 15 mm for LRFD and 11 mm for ASD for a 2.44 m high shear wall. There is no in-plane story drift limit for wind loads yet defined by the codes, but ASCE Standard – Minimum Design Loads for Buildings and Other Structures (ASCE 7-05) [12] Commentary Section CC.1.2 states "Drifts of concern in serviceability checking arise primarily from the effects of wind. Drift limits in common usage for building design are on the order or 1/600 to 1/400 of the story height. An absolute limit on interstory drift may also need to be imposed in light of evidence that damage to non-structural partitions, cladding and glazing may occur if the interstory drift exceeds about 3/8 inches (9.5 mm) unless special detailing practices are made to tolerate movement."

Table 6 summarizes the actual drift reduction factors for 2.44 m × 0.61 m shear walls for wind loads. The factors in Table 6 were determined by dividing the allowable shear strength (ASD) at h/180 drift by the allowable shear strength (ASD) based on the ultimate load limit from monotonic test results. Table 7 lists the actual reduction factors for seismic loads which were determined by dividing the allowable shear strength (ASD) based at 11 mm drift by the allowable shear strength (ASC) based on the ultimate load limit from the cyclic test results. The code reduction factor by AISI Lateral Design Standard [1] is 2w/h for shear walls with an aspect ratio greater than 2:1, but not exceeding 4:1, where permitted. For the shear wall configurations listed in Tables 6 and 7, the code reduction factor is 0.5. It can be found that the code reduction factor is a simple and conservative reduction factor that represents fairly well the strength reduction based on the drift limit for shear walls that have an aspect ratio of 4:1 in this research. It is recommended to keep using the 2 w/h reduction factor for slender CFS shear walls.

4. Conclusions

Monotonic and cyclic shear wall tests on CFS framed walls with single sided steel sheet sheathing were conducted. The nominal shear strengths for wind loads and seismic loads were established from the test results. The buckling of the steel sheathing and pullout of sheathing screws were the primary failure modes for sheet steel CFS shear walls. Flange distortion of the boundary

Table 4
Recommended nominal shear strength for wind loads for shear walls (kN/m).

Assembly Description	Aspect ratio (h:w)	Fastener spacing at panel edges (mm)			
		152	102	76	51
0.838 mm steel sheet, one side	2:1	15.1	16.7	17.9	19.0
	4:1	14.3	16.3	17.7	19.2
0.762 mm steel sheet, one side	2:1	11.6	14.0	14.7	15.4
	4:1	12.9	13.9	14.9	16.0
0.686 mm steel sheet, one side	2:1	9.1	10.0	11.1	12.2

Note: (1) Screws in the field of panel shall be installed 305 mm on center. (2) Sheet steel sheathing, wall studs, tracks, and blocking shall be of ASTM A1003 Grade 33 Type H steel with minimum yield strength, F_y, of 296 MPa and a minimum tensile strength, F_u, of 372 MPa. (3) Shear wall height to width aspect ratios (h/w) greater than 2:1, but not exceeding 4:1, shall be permitted provided the tabulated nominal strength values are multiplied by 2w/h. (4) Wall studs, tracks, and blocking shall be of 1.092 mm or thicker. (5) Wall studs, tracks, and blocking shall be of 0.838 mm or thicker.

Table 5
Recommended nominal shear strength for seismic loads for shear walls (kN/m).

Assembly description	Aspect ratio (h:w)	Fastener spacing at panel edges (mm)			
		152	102	76	51
0.838 mm steel sheet, one side	2:1	15.4	17.1	18.0	19.0
	4:1	16.0	17.8	18.5	19.2
0.762 mm steel sheet, one side	2:1	13.3	14.8	15.2	15.6
	4:1	13.5	15.4	16.5	17.6
0.686 mm steel sheet, one side	2:1	9.4	10.4	11.4	12.3

Note: (1) Screws in the field of panel shall be installed 305 mm on center. (2) Sheet steel sheathing, wall studs, tracks, and blocking shall be of ASTM A1003 Grade 33 Type H steel with minimum yield strength, F_y, of 296 MPa and a minimum tensile strength, F_u, of 372 MPa. (3) Shear wall height to width aspect ratios (h/w) greater than 2:1, but not exceeding 4:1, shall be permitted provided the tabulated nominal strength values are multiplied by 2w/h. (4) Wall studs, tracks, and blocking shall be of 1.092 mm or thicker. (5) Wall studs, tracks, and blocking shall be of 0.838 mm or thicker.

Table 6
Actual reduction factors for monotonic tests.

Assembly description	Aspect ratio (h:w)	Fastener spacing at panel edges (mm)		
		152	102	51
0.838 mm steel sheet, one side	4:1	0.732	0.777	0.578
0.762 mm steel sheet, one side	4:1	0.719	0.707	0.552

Table 7
Actual reduction factors for cyclic tests.

Assembly description	Aspect ratio (h:w)	Fastener spacing at panel edges (mm)		
		152	102	51
0.838 mm steel sheet, one side	4:1	0.800	0.692	0.596
0.762 mm steel sheet, one side	4:1	0.705	0.622	0.613

studs subjected to tension was also observed on the walls with 251 mm/305 mm screw spacing. Current AISI Lateral Design Standard [1] employs a reduction factor 2w/h to account for the flexibility of narrow shear walls that have an aspect ratio exceeding 2:1. The test results indicate that the code reduction factor is a simple reduction factor that represents fairly well the strength reduction based on the drift limit for walls that have an aspect ratio of 4:1.

Acknowledgements

The sponsorship of American Iron and Steel Institute and the donation of materials by Steel Stud Manufacturers Association and Simpson Strong-Tie Company, Inc. are gratefully acknowledged. The assistance and guidance provided by the AISI COFS Lateral Design Task Group members is highly appreciated. Special thanks are due to UNT students Hitesh Vora, Tony Dainard, Jimmy Tucker, Pradeep Veetvkuri, who assembled the specimens and assisted in the testing.

References

[1] AISI Lateral Design Standard. AISI standard for cold-formed steel framing—Lateral design 2007 edition. Washington (DC): American Iron and Steel Institute; 2007.
[2] Serrette R.L., Nguyen H., Hall G.. (1996). Shear wall values for light weight steel framing. Report No. LGSRG-3-96, Santa Clara University. Santa Clara (CA).
[3] ASTM E564-06. E564-06 Standard practice for monotonic load test for shear resistance of framed walls for buildings. West Conshohocken (PA): American Society for Testing and Materials; 2006.
[4] ICC-ES AC130. Acceptance criteria for prefabricated wood shear panels. Whittier (CA): ICC Evaluation Service, Inc; 2004.
[5] SSMA. SSMA Product technical information ICBO ER-4943P'. Glen Ellyn (IL): Steel Stud Manufacturers Association; 2001.
[6] ASTM A1003-06. Standard specification for steel sheet, carbon, metallic- and nonmetallic-coated for cold-formed framing members. West Conshohocken (PA): American Society for Testing and Materials; 2006.
[7] Yu C.. A unique buckling mode for cold-formed steel framed shear wall with sheet steel sheathing. In: Proceeding of annual stability conference, structural stability research council; 2008.
[8] ASTM A370-06. A370-06 Standard test methods and definitions for mechanical testing of steel products. West Conshohocken (PA): American Society for Testing and Materials; 2006.
[9] NASPEC. North American specification for design of cold-formed steel structural members 2007 edition. Washington (DC): American Iron and Steel Institute; 2007.
[10] AISI Product Data. AISI Standard for cold-formed steel framing — Product data 2007 edition. Washington (DC): American Iron and Steel Institute; 2007.
[11] IBC. 2006 International building code. Washington (DC): International Code Council, Inc; 2006.
[12] ASCE 7. Minimum design loads for buildings and other structures: SEI/ASCE 7-05. Washington (DC): American Society of Civil Engineers; 2005.