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Experimental investigation of cold-formed steel shear walls sheathed with steel-gypsum composite panels

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Abstract

This paper presents an experimental investigation on the behavior and performance of the cold-formed steel (CFS) framed shear walls sheathed with a composite panel which contains a 27 mil steel sheet and a 5/8 in. gypsum board bonded together. The shear wall framing configurations were selected to be suitable for mid-rise buildings. Both monotonic and cyclic tests were conducted and the research was focused on the seismic performance. It was found that the composite panel provided considerably higher shear strength than the traditional wood based sheathing and the 33 mil steel sheathing. The composite panel demonstrated similar failure mechanism and post-peak behavior as the steel sheet sheathing. It is concluded that the tested composite panel is a suitable structural sheathing material for mid-rise buildings, particularly the Type I and II constructions, in seismic areas.

1. Introduction

The cold-formed steel (CFS) is an economic structural solution for low- and mid-rise construction due to its advantages of light weight, high strength, non-combustibility, and quick installation. The American Iron and Steel Institute S213 (AISI S213, 2007) "The North American Standard for Cold-Formed Steel Framing - Lateral Design, 2007 Edition" provides nominal shear strength values for cold-formed steel (CFS) framed shear walls with a limited range of sheathing materials including 15/32 in. Structural 1 4-ply plywood sheathing, 7/16 in. oriented strand board (OSB), 0.018 in. and 0.027 in. steel sheet. Those published values were based on monotonic and cyclic test results by Serrette et al. (1996, 1997, 2002). Two recent research projects by Yu et al. (2010, 2011) studied the CFS shear walls sheathed with 0.030 in. and 0.033 in. steel sheets. Compared to the wood and wood-based panels, the steel sheet sheathing yields significantly lower shear strength and lower initial stiffness. It greatly limits the use of steel sheathing in the mid-rise commercial and multi-family residential buildings in seismic areas. On the other side, the non-combustibility of steel sheathing makes it eligible to be used in the Type I and Type II constructions. The International Building Code (IBC 2006)

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requires non-combustible materials for those two construction categories. To achieve both high strength and non-combustibility, a composite sheathing product using steel sheets and gypsum boards, Sure-Board® panels, was recently developed by the industry. The Sure-Board® panel combines the strength of steel with a myriad of gypsum-based substrate panels. The concept is to utilize the gypsum board as reinforcement to the steel sheet to restrict the out-of-plane deformation which is the main drive for the screw pull-out failure. As a result, the strength of the sheathing screw connections can be significantly increased therefore the shear strength of the entire wall system will be improved eventually. Meanwhile the steel-gypsum composite panel is non-combustible and provides a smooth surface for interior finishes.

The research presented herein is a test program conducted at the University of North Texas (UNT). The research objective is to investigate the performance of CFS framed shear walls sheathed with the steel-gypsum composite panels for mid-rise commercial buildings. The framing and sheathing details of the specimens were designed to accommodate the typical requirements for the mid-rise construction. The research was focused on the seismic performance therefore majority of the tests were conducted in a cyclic loading fashion. However monotonic tests were also conducted to establish the pre-defined cyclic displacement history required by the cyclic CUREE protocol.

This test program is part of a comprehensive and fundamental research project aimed at developing analytical models for CFS framed shear walls sheathed by different sheathing materials. The experiments will help to understand the shear resistance characteristics and the failure mechanism for the steel-gypsum composite panel. The experimental results create a solid basis for the development of analytical models. This paper is focused on documenting the test setup and discussing the experimental results.

2. Test Setup

The test program included a total of 4 monotonic and 8 cyclic shear wall tests. Both the monotonic and the cyclic tests were performed on a 16-ft. span, 12-ft. high adaptable structural steel testing frame in the structural testing laboratory at UNT. Figure 1 shows the schematic of the testing frame with a 4-ft. × 8-ft. steel shear wall. The wall was bolted to the base beam and loaded horizontally at the top. The base beam was made of a 5-in. × 5-in. × ½-in. structural steel tube. The out-of-plane displacement of the shear wall was restricted by steel rollers on both sides of the load beam. The load beam was made of a steel 'T' shape as shown in Figure 2. The 'T' shape was attached to the top track of the shear wall by two lines of No. 12 × 1-1/2-in. hex washer head self-drilling screws spaced at 3-in. on center.

The testing frame was equipped with one MTS 35-kip hydraulic actuator with ±5-in. stroke. A MTS® 407 controller and one 20-GPM MTS hydraulic power unit were employed to support the loading system. A 20-kip universal compression/tension load cell was placed to connect the hydraulic actuator to the 'T' shape for force measuring. Five position transducers were employed to measure the horizontal displacement at the top of wall, the vertical displacements of the two boundary studs, and the horizontal displacements of the bottom of the two boundary studs, as shown in Figure 1. The data acquisition system consisted of a National Instruments SXCI unit and a desktop. The applied force and the five displacements were measured and recorded instantaneously during the test.

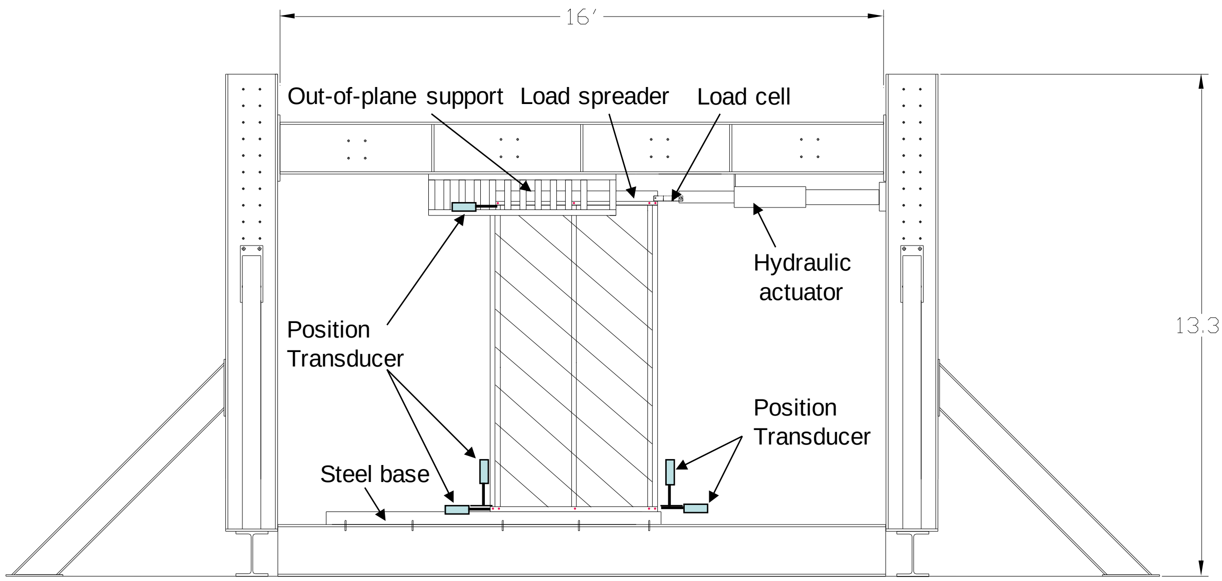

Figure 1: Schematic drawing of the test setup

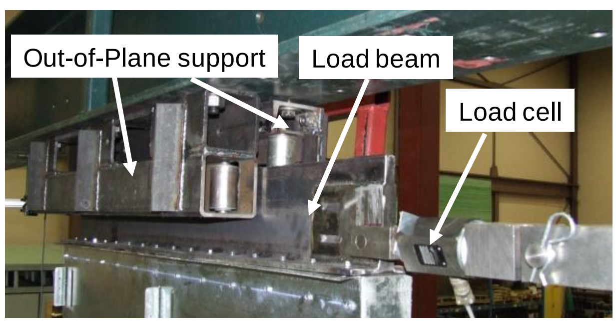

Figure 2: Close up of the top of the wall

3. Testing Method

Both the monotonic and the cyclic tests were conducted in a displacement control mode. The procedure of the monotonic tests was in accordance with ASTM E564-06 “Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings”. A preload of approximately 10% of the estimated ultimate load was applied first to the specimen and held for 5 minutes to seat all connections. After the preload was removed, the incremental loading procedure started until failure using a load increment of 1/3 of the estimated peak load.

The CUREE protocol, in accordance with AC130 “Acceptance Criteria for Prefabricated Wood Shear Panels (2004)” was chosen for the cyclic tests. The CUREE basic loading history shown in Figure 3 includes 40 cycles with specific displacement amplitudes. In this program, CUREE with up to 49 cycles was adopted in order to investigate the post-peak behavior of the shear walls. Table 1 lists the 49 CUREE displacement amplitudes. The reference displacement, Δ, equal to 60% of the shear wall drift at 80% post-peak capacity in the monotonic test. A constant cycling frequency of 0.2 Hz in the CUREE loading history was used for all the cyclic tests in this research.

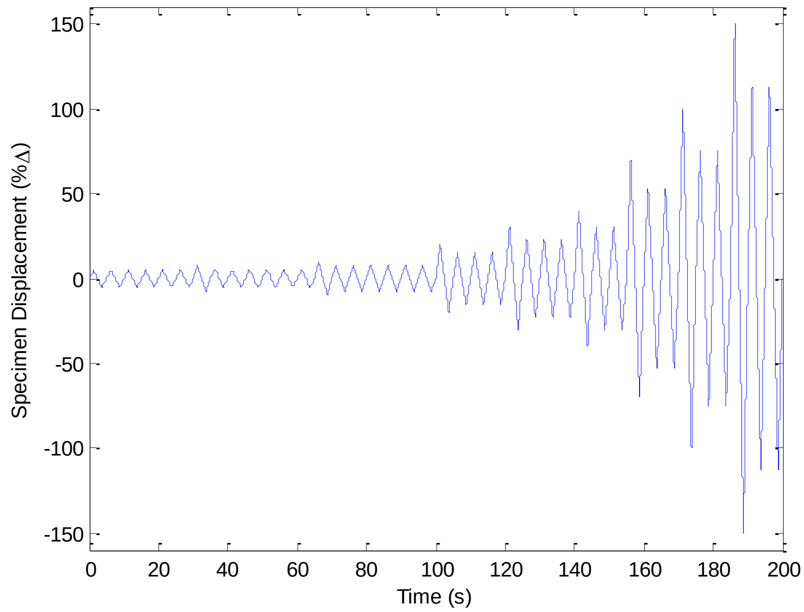

Figure 3: CUREE basic loading history (0.2 Hz, 40 cycles)

Table 1: CUREE basic loading history – 49 cycles

Cycle No.	1	2	3	4	5	6	7
%Δ	5.0	5.0	5.0	5.0	5.0	5.0	7.5
Cycle No.	8	9	10	11	12	13	14
%Δ	5.6	5.6	5.6	5.6	5.6	5.6	10
Cycle No.	15	16	17	18	19	20	21
%Δ	7.5	7.5	7.5	7.5	7.5	7.5	20
Cycle No.	22	23	24	25	26	27	28
%Δ	15	15	15	30	23	23	23
Cycle No.	29	30	31	32	33	34	35
%Δ	40	30	30	70	53	53	100
Cycle No.	36	37	38	39	40	41	42
%Δ	75	75	150	113	113	200	150
Cycle No.	43	44	45	46	47	48	49
%Δ	150	250	188	188	300	225	225

4. Test Specimens

A total 12 shear walls were tested. All walls have the same configurations listed as follows.

- Overall wall dimension: 8 ft. tall and 4 ft. wide.
- Singled sided sheathing: Sure-Board® 200 panel which was made of 5/8 in. Type X gypsum board laminated to 33 ksi 27 mil steel sheet.

- Screw type and spacing: 1-3/4 in. No. 8 self-drilling screws, 2 in. spacing on the panel edge and 12 in. spacing in the field.
- Double studs at boundary and single stud in the field.
- SSMA (2001) standard studs and tracks were used. ASTM A653/A1003 steel with G60 coating.
- Hold-down: two Simpson Strong Tie SHD10 hold-downs were used for each wall. The hold-downs were attached to the boundary studs from inside. The hold-downs were in contact with the bottom track.
- Anchorage bolt: ASTM A325 5/8 in. bolts. Four anchor bolts were used for each wall. Two for hold-downs, two for anchoring the bottom track to test bed.

The various configurations considered in this test program include the thickness of the framing members and the stud spacing. Table 2 summarizes the framing details for the specimens.

Table 2: Framing configurations for shear walls

Test No.	Stud Spacing	End Double Studs	Interior Studs	Track
1 (monotonic)	16”	50 ksi 362S162-54	33 ksi 362S162-43	33 ksi 362T125-43
2, 3 (cyclic)	16”	50 ksi 362S162-54	33 ksi 362S162-43	33 ksi 362T125-43
4 (monotonic)	24”	33 ksi 362S162-43	33 ksi 362S162-43	33 ksi 362T125-43
5, 6 (cyclic)	24”	50 ksi 362S162-54	33 ksi 362S162-43	33 ksi 362T125-43
7 (monotonic)	16”	50 ksi 362S162-68	50 ksi 362S162-54	50 ksi 362T125-54
8, 9 (cyclic)	16”	50 ksi 362S162-68	50 ksi 362S162-54	50 ksi 362T125-54
10 (monotonic)	24”	50 ksi 362S162-54	50 ksi 362S162-54	50 ksi 362T125-54
11, 12 (cyclic)	24”	50 ksi 362S162-68	50 ksi 362S162-54	50 ksi 362T125-54

5. Test Results and Discussion

5.1 Shear Wall Tests

The experiments show that the behavior of shear wall using the steel sheet – gypsum composite panel has two phases. At the initial stage, Phase 1, the wall behaves elastically, and the panel moves and rotates as a rigid body. The shear resistance of the wall is provided by the shear rigidity of the panel through screws on the entire sheathing. The gypsum board functions as reinforcement to the steel sheet to restrict the out-of-plane shear buckling. On the other hand the shear stiffness of the gypsum board also contributes to the shear resistance of the entire wall. Once the steel sheet shear buckling reaches the level that the gypsum board cannot restrict the out-of-plane deformation of the steel sheet, the wall behavior enters the Phase 2. In this phase, the shear buckling shape of the steel sheet can be seen; the direction of the corrugation is diagonal from corner to corner. The shear resistance of the wall is provided by the tension field action of the steel sheet. The shear wall reaches its peak load when the tension field action causes failure of the screw connections at the corners of the wall. The screw connection failure can have four different phenomena: screw shear, steel sheet bearing, screw pull-out, and distortion of the stud flange at bottom. The Phase 2 behavior is similar to the behavior of the CFS shear walls sheathed by steel sheets observed in Yu and et al. (2010. 2011).

Overall, seven types of damages were observed in the shear walls tests as the following.

A. Steel sheet buckling. The steel sheet demonstrated out-of-plane deformation in the diagonal strip region where the material was subject to concentrated tensile stresses. Figure 4 shows a typical steel sheet buckling. Shear strength of the wall is provided by the tensile strength of the steel sheet in the diagonal strip region. The buckling of sheet would cause distortion of the studs, screw pull-out, and cracking of the gypsum board.

B. Gypsum board cracking. The cracking in the gypsum board was primarily caused by the out-of-plane shear deformation of the steel sheet. Figure 5 shows the gypsum board cracking failure.

C. End stud distortion at bottom. The tension field action on the steel sheet can cause the distortion of the stud flange at the bottom of the end studs. In Figure 5, the end stud distortion can be seen.

D. End stud buckling. The end studs are subjected to the overturning forces. Stud buckling may occur when the stud size is not properly selected. In this test program, the Test #4 failed in end stud buckling. The Test #4 used 43 mil framing members. Figure 6 shows the end stud buckling.

E. Sheathing screw pull-out. The screw pull-out failure is the result of a combined action by the out-of-plane deformation and the tension field action of the steel sheet. The screw pull-out failure is preliminary located on the end studs close to the corners of the wall. Figure 7 shows a typical screw pull-out failure. Due to the relatively long length of screws used in this research, the screws were not totally pulled off from the frame but the sheathing was no longer in contact with the frame. The shear resistance of the wall was significantly reduced when the screw pull-out happened.

F. Interior stud distortion. The distortion of the interior stud was only observed on Test #2 which used 16” stud spacing and was subjected to cyclic loading. Figure 8 shows the distorted interior studs. The damage is caused by significant out-of-plane deformation of the sheathing.

G. Hold-down failure. In Test #9, the screw heads were sheared off from the hold-downs. To prevent such failure, the hold-downs on Test #11 and #12 were reinforced by additional welds. Figure 9 shows the hold-down failure in Test #9. Due to the undesirable failure mode, the Test #9 is excluded from the analyses on the shear strength of walls.

Figure 4: Shear buckling of steel sheet

Figure 5: Gypsum board cracking and end stud distortion

Figure 6: End stud buckling

Figure 7: Screw pull out failure

Figure 8: Distortion of interior studs

Figure 9: Hold-down failure

Table 3 summarizes the peak loads, the lateral deflection at peak loads, and the damage types for the shear wall tests. The results indicate that the peak load of the cyclic tests is systematically higher than that of the monotonic tests. Tests #1, #2, #3, and Tests #7, #8 are two groups that have the same wall configurations but different testing methods in each group. The peak loads of cyclic tests are 12% and 4% higher than that of the monotonic tests respectively. The impact of the loading method has less impact to the thicker framed walls.

Tests #2, #3, #5, #6 and Tests #8, #11, #12 are two groups that can be used to study the impact of the stud spacing because the stud spacing is the only difference in the wall configurations in each group and all those tests are cyclic. It can be found that the smaller stud spacing leads to higher peak loads. The strength increase due to the smaller spacing is 22% and 5% respectively for those two groups of tests. The stud spacing has less impact to the walls with thicker frames.

Tests #1, #7, Tests #2, #3, #8, and Tests #5, #6, #11, #12 are three groups that can be used to evaluate the impact by the framing thickness. The walls with thicker framing members systematically yield higher shear strength than those with thinner members. On average, walls with 68 mil end studs demonstrate 13% higher strength than that of walls using 54 mil end studs.

Table 3: Summary of shear wall test results

Test Label (protocol)	Peak Load (lbs)		Lateral Deflection at Peak Load (in.)		Avg. Peak Load (lbs)	Avg. Δ (in.)	Damage Types
	+P	-P	+Δ	-Δ			
1 (monotonic)	12385	-	1.837	-	12385	1.837	A, B, C
2 (49 cycles)	13480	13660	2.283	2.348	13570	2.316	A, B, C, E, F
3 (49 cycles)	14930	13442	2.860	2.388	14186	2.624	A, B, E
4 (monotonic)	9522	-	1.413	-	9522	1.413	B, D
5 (49 cycles)	11040	9817	1.505	1.377	10428	1.441	A, B, C, E, F
6 (43 cycles)	12421	12186	1.726	0.913	12304	1.320	A, B, E, F
7 (monotonic)	13991	-	2.174	-	13991	2.174	A, B, C
8 (49 cycles)	14908	14239	2.664	2.325	14574	2.495	A, B, E
9 (43 cycles)	14599	12984	2.722	1.974	13792	2.348	G
10 (monotonic)	10463	-	1.480	-	10463	1.480	A, B, C
11 (43 cycles)	14859	13080	1.898	1.747	13970	1.823	A, B, E
12 (43 cycles)	14334	13110	1.932	1.903	13722	1.918	A, B, E
Note: A - steel sheet buckling; B - gypsum board cracking; C - end stud distortion at bottom; D - end stud buckling; E - sheathing screw pull-out; F - interior stud distortion; G - hold-down failure.							

Figure 10 shows a comparison of the load vs. displacement curves for Tests #1 and #2, it can be seen that the monotonic and the cyclic behavior of the shear wall have close initial stiffness and similar nonlinearity. The shear walls with composite panels show similar cyclic hysteretic behavior as the steel sheet shear walls (Yu 2010, Yu et al. 2011). Pinching starts at early stage of the cyclic loading, and stiffness and strength degradation begin once the load passes the peak. Because of the fastener failures, the wall loses its shear resistance significantly in the post-peak region. The same findings can also be observed in other tests of this test program. Due to the fact that the CFS shear walls using the steel-gypsum panels demonstrate similar failure modes and cyclic hysteretic behaviors as the CFS shear walls using steel sheet sheathing, it is recommended the same code approved seismic performance factors for steel sheet sheathed CFS shear walls can be applied to the CFS shear walls sheathed with the steel-gypsum composite panels.

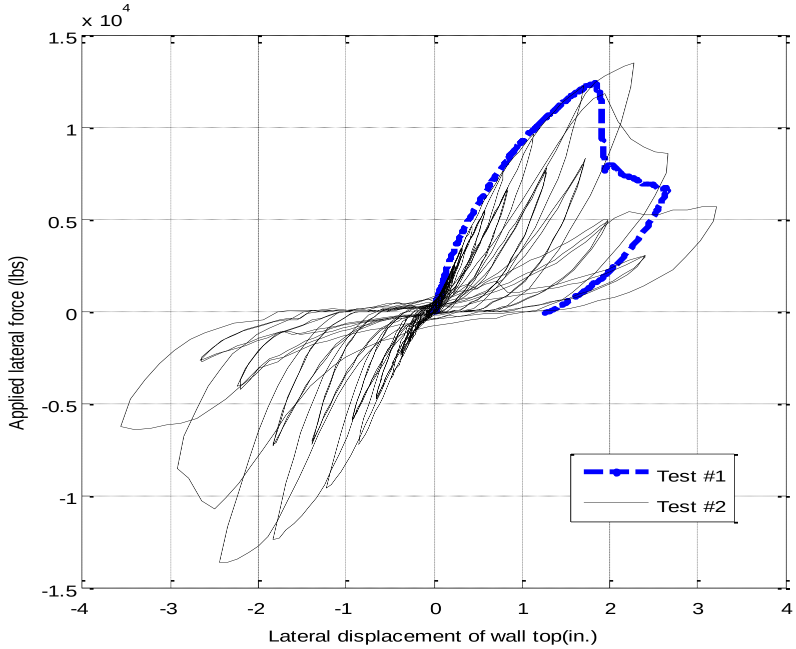

Figure 10: Load vs. displacement curves for Tests #1 and #2

Table 4: Nominal shear strength for seismic loads for various sheathing materials (plf)*

Framing thickness	0.027" steel + 5/8" gypsum	15/32" Structural 1 sheathing (4-ply)	7/16" OSB	0.033" steel sheet
54 mil	2842	2190	2350	1872
68 mil	3462	-	3080**	-
Note: * unless specified, the shear walls have the same configuration: aspect ratio (h/w) 2:1, fastener spacing at panel edges 2", sheathing screw size No.8, stud spacing 24". ** screw size No. 10				

Table 4 lists the comparison of the nominal strength for the composite panels with the traditional sheathing materials listed in AISI S213 (2007). The nominal strength for the steel-gypsum composite panel is determined by the average peak loads of the identical cyclic tests. The nominal strength for the 15/32" Structural 1 4-ply sheathing and the 7/16" OSB is published in the Table C2.1-3 in AISI S213 (2007). The nominal strength for 0.033" steel sheet is based on the experimental results in Yu et al. (2011). Table 4 indicates that the tested 27 mil steel – 5/8" gypsum board composite panel has considerably higher shear strength than all the other three sheathing materials. The composite panel is a suitable structural sheathing material for mid-rise CFS building to replace the traditional wood based panels. The composite panel has particular advantages in the Type I and Type II constructions due to its feature of non-combustibility.

5.2 Material Properties

Coupon tests were conducted according to the ASTM A370-06 "Standard Test Methods and Definitions for Mechanical Testing of Steel Products". The test results are summarized in Table 5. The coating on the steel was removed by hydrochloric acid prior to the coupon tests. The coupon tests were conducted on the INSTRON 4480 universal testing machine. An INSTRON 2630-106 extensometer was employed to measure the tensile strain.

Table 5: Coupon test results

Component	Uncoated Thickness (in.)	Yield Stress F_y (ksi)	Tensile Strength F_u (ksi)	F_u/F_y Ratio	Elongation for 2 in. Gage Length (%)
43 mil stud (Grade 33)	0.0429	34.1	43.3	1.27	25.8%
54 mil stud (Grade 50)	0.0545	54.1	61.9	1.15	21.7%
68 mil stud (Grade 50)	0.0671	59.1	65.9	1.11	19.7%
43 mil track (Grade 33)	0.0445	38.3	43.1	1.13	22.4%
54 mil track (Grade 50)	0.0544	57.0	67.4	1.18	16.8%
27 mil sheet (Grade 33)	0.0276	41.2	44.7	1.09	28.8%

The coupon test results indicate that the measured uncoated thicknesses of all the components are greater than the minimum delivered thickness but less than the design thickness specified in AISI Standard – Product Data (AISI S201, 2007). All the materials have both tensile strength and yield stress greater than the specified values. All the materials meet the minimum ductility requirements specified by the North American Specification for Design of Cold-Formed Steel Structural Members 2007 Edition (AISI S100, 2007), which requires the tensile strength to yield strength ratio greater than 1.08, and the elongation on a 2-in. gage length higher than 10%.

6. Conclusions and Future Research

A test program was conducted to investigate the behavior of the 68 mil and 54 mil CFS framed shear walls using 27 mil steel – 5/8" gypsum composite panels. The test results show that the composite panel gives considerably higher shear strength than the traditional sheathing material listed in AISI S213 (2007). A systematic analysis following the FEMA P695 (2009) methodology may be needed to accurately obtain the seismic performance factors for CFS building using the composite panel shear walls. However due to the fact that the tested panels demonstrate similar failure mechanisms and post-peak behaviors as the CFS shear walls sheathed by steel sheets. It is recommended that the seismic performance factors for CFS buildings using steel sheet shear walls can be applicable to the CFS buildings using the composite panel shear walls.

The test program is part of an ongoing research project to develop analytical models for CFS shear walls using various sheathing materials. The experimental data will be used to verify the future models for accurately predicting the strength, stiffness, and drift capacity of the CFS shear walls.

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