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Innovative Cold-Formed Steel Framed Shear Wall Sheathed with Corrugated Steel Sheets: Experiments and Dynamic Analysis

Cheng Yu

Guowang Yu

Jie Wang

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Innovative Cold-Formed Steel Framed Shear Wall Sheathed with Corrugated Steel Sheets: Experiments and Dynamic Analysis

Cheng Yu[1], Guowang Yu[2], Jie Wang[3]

Abstract

Cold-formed steel framed shear wall sheathed with corrugated steel sheets is a promising shear wall system for low- and mid-rise constructions in high wind and seismic zones due to its advantages of non-combustibility, high shear strength, and high stiffness. However recent research projects showed that the corrugated steel sheathing demonstrated low ductility. This paper presents an experimental study aimed at improving the ductility of cold-formed steel shear walls sheathed with corrugated steel sheathing. A method of using opening in the sheathing is employed to improve the shear wall's ductility meanwhile controlling the damage locations and failure mechanism. A total of 11 sheathing configurations were investigated and 19 monotonic and cyclic full-scale shear wall tests were conducted in this project. The research discovered that with proper opening in the sheathing, the corrugated sheet shear wall can yield significantly improved ductility while maintaining high-level shear strength. Additionally, nonlinear dynamic analyses were also carried on to verify the building's seismic performance when the innovative shear wall was installed. The dynamic analyses show that the new shear wall system can greatly reduce the seismic effects and decrease the building's collapse probability.

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[1] Associate Professor, Department of Engineering Technology, University of North Texas, Denton, TX, <cheng.yu@unt.edu>

[2] Graduate Research Assistant, Department of Engineering Technology, University of North Texas, Denton, TX, <websteryu75@gmail.com>

[3] Graduate Research Assistant, Department of Civil Engineering, Tsinghua University, Beijing, China, <mjdwangjie@163.com>

1. Introduction

Cold-formed steel (CFS) becomes an attractive construction material for low- and mid-rise buildings because of its attributes of light weight, high strength, ease mass production and prefabrication, uniform quality, non-combustibility, etc. The lateral force resisting system in CFS buildings usually employs CFS framed shear walls sheathed by steel sheets, oriented strand board (OSB), plywood panels, or braced by diagonal steel straps. The sheathing is usually fastened to the frame around boundary elements and on the interior studs by self-drilling screws. The International Building Code (IBC 2006) and the North American Standard for Cold-Formed Steel Framing - Lateral Design (AISI S213-12) provide provisions for CFS shear walls using three type of sheathing materials: 15/32 in. Structural 1 plywood, 7/16 in. OSB, and 0.018 in., 0.027 in., 0.030 in., 0.033 in. steel sheet. Those published values were based on research of Serrette et al (1996, 1997, and 2002), and Yu (2011). Compared with shear walls sheathed by wood-based panels, the steel sheet shear walls yield considerably lower shear strength. On the other hand, IBC (2006) requires non-combustible materials to be used for shear walls for Type I and II constructions. Therefore "all steel" shear walls with high strength and stiffness are in great need for low- and mid-rise CFS buildings. One solution of high strength CFS shear wall is to use corrugated steel sheets as sheathing for shear walls.

The CFS corrugated steel sheets, commonly used as floor or roof decking, have considerably high in-plane strength and stiffness due to the cross section shape. Therefore, if designed properly, CFS shear wall sheathed with corrugated sheet could be used as an alternative lateral-force resisting system. Some studies have been done to investigate the behavior of CFS corrugated sheet shear walls. L.A. Fülöp and D. Dubina (2004) developed a testing program to investigate the structural characteristics of 8 ft. high $\times$ 12 ft. wide CFS shear walls with different sheathing materials including LTB20/0.5 corrugated steel sheets, gypsum boards, and OSB. A total of 7 monotonic tests and 8 cyclic tests were conducted. The protocol for cyclic tests adopted ECCS Recommendation (1985) with a relatively low loading frequency of either 0.00028 Hz (6 min/cycle) or 0.0056 Hz (3 min/cycle). The CFS frames used U154/1.5 tracks (6 in. web depth, 0.060 in. thickness), and C150/1.5 C-section studs (6 in. web depth, 0.060 in. thickness) placed at 24 in. on center. Double studs (back-to-back) were used at the ends of the walls and around the opening. Fülöp and Dubina (2004) concluded that the CFS walls were rigid and could effectively resist lateral loads. The failure of the seam fastener caused the failure for the corrugated sheet specimens. The test results showed the 3/8 in. OSB specimens had significantly higher shear strength than the corrugated sheet specimens. However the geometries and material properties of the corrugated sheets were not reported in Fülöp and Dubina (2004).

Stojadinovic and Tipping (2007) conducted 44 cyclic racking tests on CFS shear walls sheathed with corrugated sheet steel. 40 specimens were 8 ft 2 in. × 4 ft and 4 specimens were 8 ft 2 in. × 2 ft. The shear walls were sheathed with 0.027 in., 0.033 in. and 0.043 in. corrugated Shallow-Vercor type decking with 9/16 in rib height. The framing members were SSMA 33 mil, 43 mil, 54 mil, and 68 mil structural studs and tracks. The boundary frames of all of the shear walls were strengthened by double L6×4×3/8" angles which excluded failures in the boundary elements and also required no hold-down to be installed. In the test, screws gouge elongated holes in the metal studs and/or sheeting due to racking shear. And warping of the end corrugation became evident and coinciding diagonal tension and compression fields developed across the panel. The shear walls failed in a large of "popping" out (pulling out) of the screws along the boundary members due to the distortion of the corrugated sheet steel. Based on the test results, nominal shear values and seismic performance factors of tested shear walls were proposed.

Yu et al (2009) conducted a preliminary research on CFS corrugated shear walls. A total of 8 tests on 8 ft. × 4 ft. CFS walls with corrugated steel sheathing placed on one side of the wall were conducted. The corrugated steel sheets were Vulcraft deck type 0.6C with 27 mil thickness and 9/16 in. rib height. For each shear wall specimen, the sheathing was formed by three corrugated steel sheets. The sheets were overlapped for one rib and connected by a line of screws at each joint. The screw spacing was 2.5 in on the panel edges and joints and 5 in. the field. The preliminary research was focused on developing appropriate framing details to achieve the failure in the sheathing which could be considered the ultimate shear strength that the corrugated CFS shear wall can deliver. A variety of configurations was considered in the preliminary work including the thickness of the framing members (43 mil and 68 mil), the sheathing and framing screw size (No. 8 and No. 12) and spacing, as well as the boundary studs details. All the specimens had the same wall aspect ratio of 2:1 with 8 ft high and 4 ft wide. The research discovered that the 0.027 in. corrugated steel sheet has considerably high stiffness and high in-plan shear strength. Thicker framing members (68 mil) and larger screws (No. 12) were recommended to fully utilize the strength of the 0.027 in. corrugated sheathing. The preliminary research also found that the tested corrugated CFS shear wall demonstrated poor ductility. The research presented in this paper is a test program recently conducted at the University of North Texas to investigate the behavior and strength of CFS shear walls using corrugated steel sheathing with openings. The research goal is to develop a noncombustible, high strength, high stiffness, and high ductility shear wall system for low- and mid-rise construction in seismic zones.

2. Test Program

2.1 Test Setup

A total of 11 sheathing configurations (9 perforated, 2 nonperforated) were investigated in the test program and 19 monotonic and cyclic full-scale shear wall tests were conducted. The monotonic and cyclic tests were conducted on a 16 ft span, 12 ft high self-equilibrating steel testing frame in the Structural Testing Laboratory of the University of North Texas. Figure 1 shows the front view of the test frame with an 8 ft × 4 ft shear wall. The testing frame was equipped with one 35 kip hydraulic actuator with 10 in. stroke. The shear wall was fixed to the base beam by two hold-downs and 4 anchor bolts. A 20 kip compression/tension load cell was used to measure the applied force. Five position transducers were employed to measure the horizontal displacement at the top of the wall and the vertical and horizontal displacements of the bottoms of the two boundary studs. The lateral load initiated by the actuator was applied directly to the T-shape steel load beam which was attached to the top track with 2 – No.12. Consequently, a uniform linear racking force could be transmitted to the top track of the shear wall. The out-of-plane movement of the wall was prevented by the lateral supports placed on both sides of the T shape beam. The applied force and the five displacements were measured and recorded instantaneously during the test.

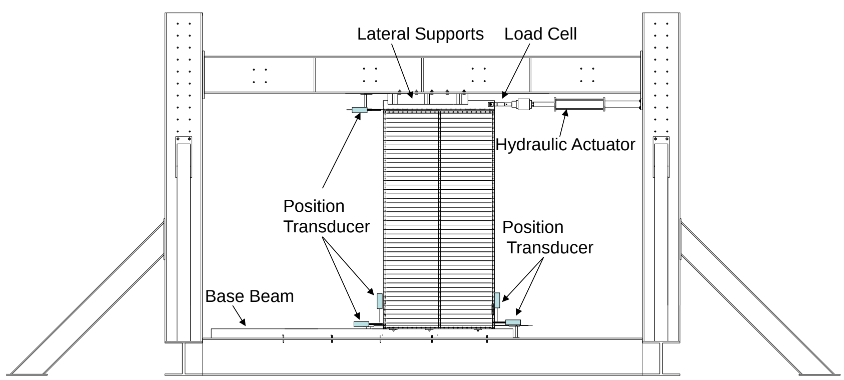


Figure 1: Front view of the test setup

2.2 Test Procedure

The research focused on the seismic performance of the shear walls, therefore at least two cyclic tests were performed for each wall configuration. In order to obtain the wall's displacement capacity for establishing the cyclic test protocol, monotonic tests were also conducted. 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 (2012) “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 followed until structural failure was achieved using a load increment of 1/3 of the estimated ultimate load.

The CUREE protocol, in accordance with AC130 (2004), was chosen for the reversed cyclic tests. The standard CUREE loading history included 40 cycles with specific displacement amplitudes. This test program used 43 cycles as listed in Table 1 in order to investigate the post peak behavior of the walls. The specified displacement amplitudes are chosen based on a percentage of the ultimate displacement capacity determined from the monotonic tests. In this test program, the displacement capacity of walls without sheathing opening was chosen for all cyclic tests. The ultimate displacement capacity was defined as a portion (i.e. γ=0.60) of maximum inelastic response, Δ, which corresponds to the displacement at 80% peak load. A constant cycling frequency of 0.2-Hz (5 seconds) for the CUREE loading history was adopted for all the cyclic tests in this research.

Table 1: CUREE loading history

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

2.3 Test Specimens

All the tested shear walls in this project were 8 ft high and 4 ft wide (2:1 aspect ratio). Steel Studs Manufacturers Association (SSMA) structural stud (350S163-68) and track members (350T150-68) were used for the framing of all walls. The chord studs used double C-shaped sections fastened together back-to-back with No.12 × 1 in. hex head self-drilling screws pairs at 6 in. on center. The middle stud used one C-shaped section. In each wall, two Simpson Strong-Tie® S/HD15S hold-down (one on each side) were attached to both boundary studs by using No.14× 1 in. hex washer head self-drilling screws. For chord studs having a punch-out at the hold-down location, additional welding was used to reinforce the hold-down to studs attachment. The corrugated steel sheets were 0.6C, 27 mil thick corrugated steel sheet with 9/16 in. rib height (shown in Figure 2) manufactured by Vulcraft Manufacturing Company. The sheathing was installed on one side of the wall using No.12 × 1 in. hex head self-drilling screws. For each wall specimen, the sheathing was composed of three corrugated steel sheets which were connected by single line of screws. The screw spacing was 2.5 in. at the horizontal seams of the sheets and along the edges of the wall. The screw spacing was 5 in. along the interior stud. Two 5/8 in. diameter grade 5 anchor bolts were used as the shear bolts in each wall. One minimum 5/8 diameter grade 8 anchor bolt was used for the each hold-down.

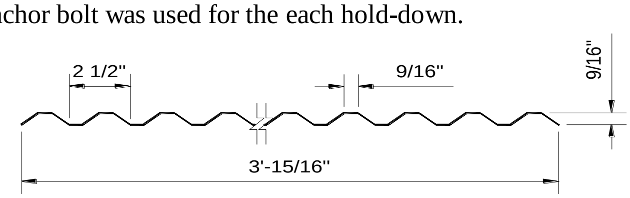


Figure 2: Corrugated sheet steel profile

Coupon tests were conducted according to the ASTM A370 (2006) "Standard Test Methods and Definitions for Mechanical Testing of Steel Products" to obtain the actual properties of the test materials in this project. The coupon test results are summarized in Table 2. A total of 19 shear walls sheathed by corrugated steel sheathing were tested (Table 3) and 9 opening configurations were studied in this test program (Table 4). The circular holes were made by using a plasma cutter. The slits were made by a grinder with 0.045 in. thick sand blade. The average slit width was measured as 0.059 in.

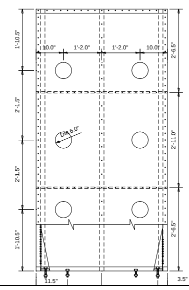
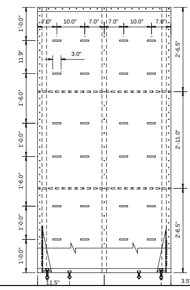
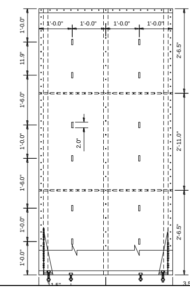
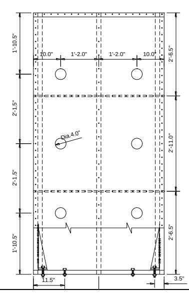
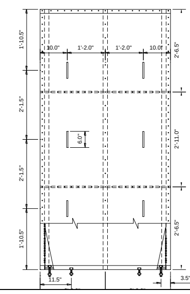
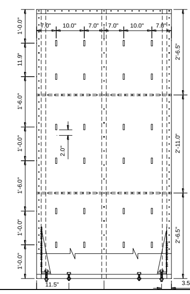
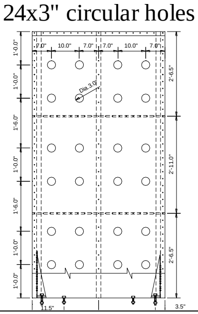
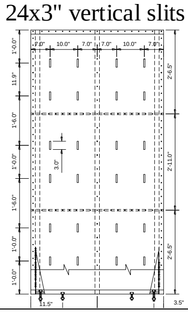
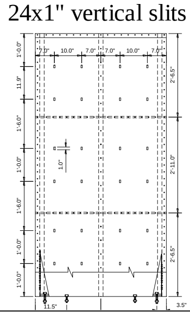
Table 2: Material properties

Component	Uncoated Thickness (in.)	Yield Stress F_y (ksi)	Tensile Strength F_u (ksi)	F_u/F_y	Elongation for 2 in. Gage Length (%)
0.027 in. corrugated sheet	0.0290	95.00	96.50	1.02	22.2%
68 mil stud	0.0711	55.85	69.81	1.25	18.2%
68 mil track	0.0721	54.33	71.63	1.32	20.0%

Table 3: Test matrix for shear wall test

Test label	Opening configuration	Test protocol
No.1	No-seaming screws	Cyclic
No.2	No-opening	Monotonic
No.3	No-opening	Monotonic
No.4	No-opening	Cyclic
No.5	No-opening	Cyclic
No.6	6x6" circular holes	Monotonic
No.7	6x6" circular holes	Cyclic
No.8	6x6" circular holes	Cyclic
No.9	6x4" circular holes	Cyclic
No.10	6x6" vertical slits	Cyclic
No.11	24x3" circular holes	Cyclic
No.12	24x3" vertical slits	Cyclic
No.13	24x3" vertical slits	Cyclic
No.14	24x3" horizontal slits	Cyclic
No.15	12x2" vertical slits	Cyclic
No.16	24x1" vertical slits	Cyclic
No.17	24x2" vertical slits	Monotonic
No.18	24x2" vertical slits	Cyclic
No.19	24x2" vertical slits	Cyclic

Table 4: Opening configurations

6x6" circular holes 	24x3" horizontal slits 	12x2" vertical slits 
6x4" circular holes 	6x6" vertical slits 	24x2" vertical slits 
24x3" circular holes 	24x3" vertical slits 	24x1" vertical slits 

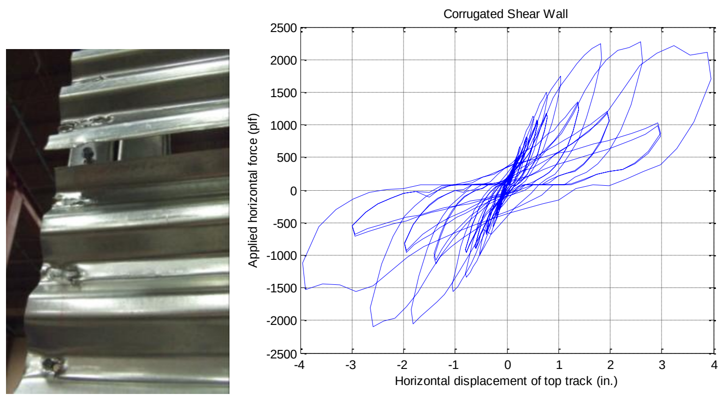
3. Test Results

The average peak load, initial stiffness, deflection of top of the wall at the peak load, and the ductility factor are provided in Table 5. The shear wall's ductility can be evaluated by using the concept of equivalent energy elastic plastic model (EEEP) which was first proposed by Park (1989) and later revised by Kawai et al. (1997). The ductility factors were calculated as the ratio of maximum displacement to the maximum elastic displacement. $\mu = \frac{\Delta_{max}}{\Delta_e}$. The maximum displacement, Δ_{max}, was defined by the intersection point of the EEEP curve and the observed test curve. The maximum elastic displacement, Δ_e, was defined by the intersection point of the EEEP curve elastic and plastic portion.

Table 5: Summary of shear wall test results

Test label	Average peak load (lbf)	Average deflection (in)	Average stiffness (lb/in.)	Ductility factor
No.1_no seaming screws	2189	2.592	8601	3.793
No.2_no opening	4154	2.326	5399	1.511
No.3_no opening	5008	3.032	10879	2.051
No.4_no opening	4289	2.635	10430	1.644
No.5_no opening	5033	2.563	10971	1.757
No.6_6x6" holes	3223	3.097	5399	1.678
No.7_6x6" holes	3149	2.543	6333	1.679
No.8_6x6" holes	2923	2.671	6892	2.415
No.9_6x4" holes	3733	2.516	8489	2.039
No.10_6x6" slits	2753	1.870	8045	2.297
No.11_24x3" holes	2939	3.324	5678	2.204
No.12_24x3" slits	2938	3.266	8568	3.699
No.13_24x3" slits	2964	2.444	8310	3.365
No.14_24x3" horizontal slits	4156	1.966	11132	1.534
No.15_12x2" slits	3569	1.861	11392	2.128
No.16_24x1" slits	4616	2.385	11129	1.595
No.17_24x2" slits	3093	3.741	8480	3.090
No.18_24x2" slits	3095	2.808	11126	3.646
No.19_24x2" slits	3103	3.414	9987	3.027

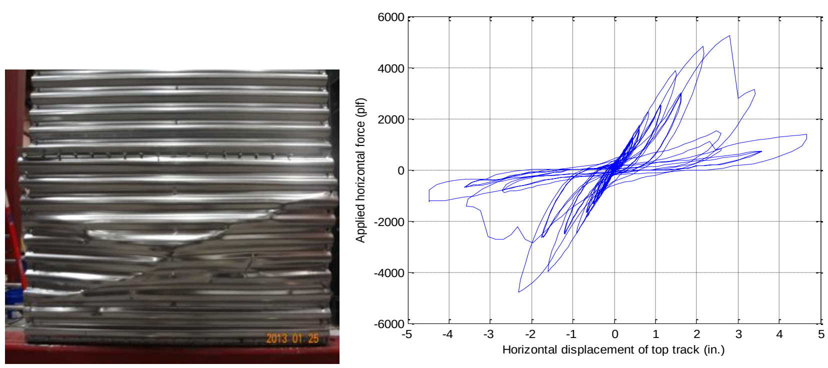
Test No.1 had no stitch screws at the sheet joints, the steel decks worked individually. In the test, a large relative horizontal movement was found between every two adjacent sheets. The shear failed by the sheathing screw's bearing and pull out. The shear wall demonstrated low shear strength but reasonably high ductility. Figure 3 shows the screw failure and the test curve.



(a) screw failure (b) hysteresis curve

Figure 3: Test results of No. 1 shear wall

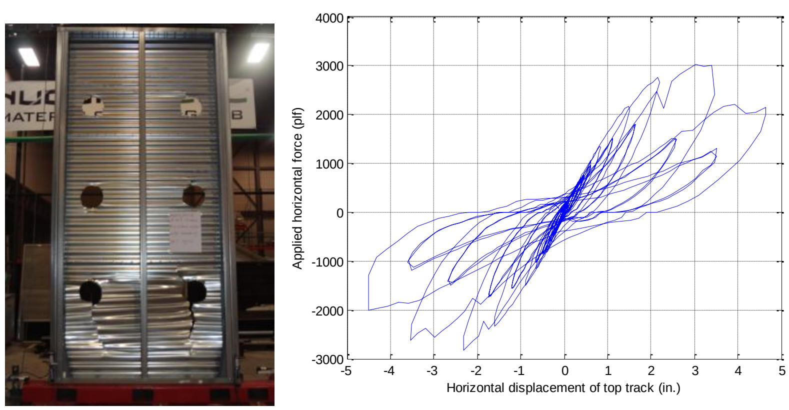
Tests No. 2 to 5 were walls using unperforated corrugated sheets with screws on the seams. The walls showed high strength and high stiffness but low ductility, the strength dropped instantly once the sheathing buckled. Shear buckling in the sheathing was observed in tests No. 3 and 5. Unexpected failure in hold-down occurred in tests No. 2 and 4. Figure 4 shows the results of test No. 5



(a) shear buckling (b) hysteresis curve

Figure 4: Test results of No. 5 shear wall

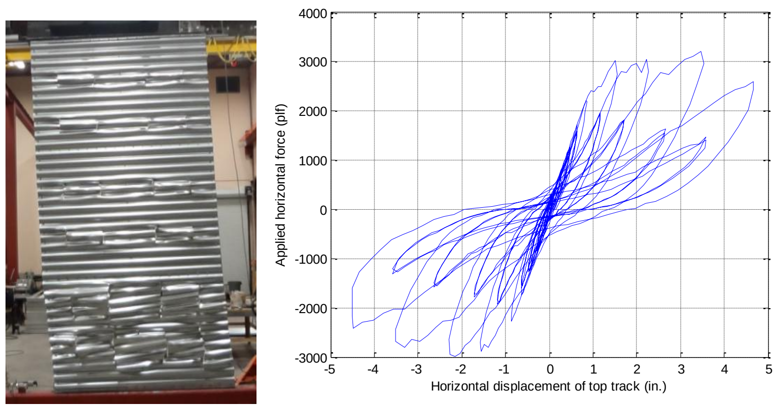
The concept of creating opening in the corrugated steel sheathing is to force the material yielding and rupture to occur in the sheathing at the opening locations, and allow the out of plane deformation and material yielding to become the energy dissipation mechanism of the shear wall. It is expected that the wall will lose its strength gradually as the ruptures grow gradually in the sheathing. Another advantage of introducing the opening in the sheathing is that the damage locations can be controlled to be away from the boundary elements and fasteners on the edges so that the building collapse can be intentionally protected. Various circular hole configurations were investigated in the program. It stated with 6 × 6-in. holes and the wall demonstrated improved ductility. In the test, the sheathing showed large out of plane deformation in the opening areas. The shear wall reached the peak load when the rupture of the bottom holes occurred, the rupture continued to grow and started to occur in upper holes areas when the shear wall lost its shear strength in the post-peak stage. The walls with circular holes demonstrated significantly reduced stiffness and slightly improved ductility. The shear wall's performance was improved as the circular hole size became smaller, but the stiffness and the strength were still largely reduced and no significantly improved ductility was observed. Figure 5 shows the results of test No 8. It was concluded that the circular holes was able to yield large out-of-plane deformation and ruptures at the hole edges to improve the wall's ductility, but the holes significantly weakened the structural integrity of the corrugated sheets, the wall's strength and stiffness were largely reduced. The circular holes are not recommended for the purpose of ductility improvement.



(a) sheathing rupture (b) hysteresis curve

Figure 5: Test results of No. 8 shear wall

The research moved on to investigate the behavior of shear walls sheathed with corrugated sheathing using slits. The idea was to reduce the opening area to maintain stiffness of the wall at the same time improve the wall's ductility by the gradual ruptures at the slits. Specimen No.10 had six 6 in. long vertical slits, the rupture started from the two end points of slits and extended vertically up and down. Comparing to 6×6-in circular opening, the No. 10 shear wall's stiffness was increased, but the ductility was not improved. The 24×3-in. vertical slits configuration was used for shear walls No.12 and 13. The same failure mode as that of 6-in vertical slits wall was found. The short slit's length did not significantly weaken the sheathing's integrity, the slits were extended progressively and the shear wall stiffness degraded gradually. A higher average ductility factor of 3.532 was achieved on tests No. 12 and 13. More slit configurations were analyzed and it was found that less slit length would cause higher shear wall strength and stiffness but lower ductility. The slit configuration of 24×2-in. demonstrated a high ductility, a high initial stiffness, and a considerably high strength. The average results of two cyclic tests, No. 18 and 19, are 3.34 for ductility factor, 10557 lb/in. for initial stiffness, and 3103 plf for peak load (similar to 7/16" OSB and higher than 15/32" plywood). This sheathing configuration showed a balanced structural performance and therefore is considered as a suitable configuration for mid-rise buildings in seismic areas. Figure 6 shows the results of test No. 18.

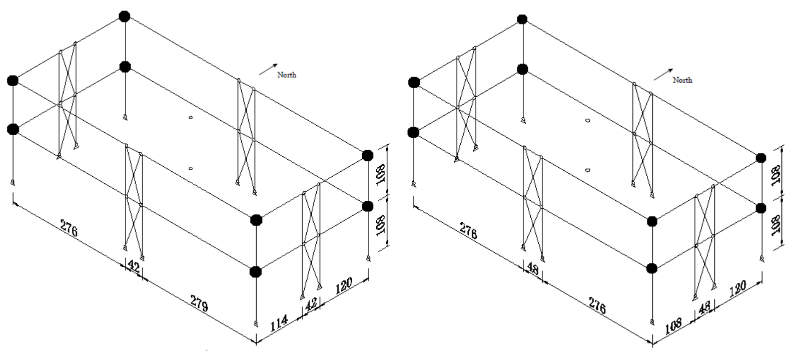


(a) sheathing rupture (b) hysteresis curve

Figure 6: Test results of No. 18 shear wall

4. Dynamic Analysis

The nonlinear dynamics analysis tool OpenSees was used to analyze a 2-story CFS light framed building using the two shear wall configurations: (1) wall sheathed by corrugated steel sheathing (2) wall sheathed by corrugated steel sheathing with slits. The building archetype in the NEES-CFS project (Madsen, Nakata, Schafer, 2011) was used in this research as the baseline archetype. The hypothetical symmetrical 2-story office building is assumed to be located in Orange County, California which has a total plan area of 1150 sq ft. For simplicity, torsion is neglected. Site Class D was chosen as it is typical for sites in the vicinity of this project. For the office occupancy chosen, IE = 1.0 was used. Two OpenSees models were created, as shown in Figure 7. In the models, the mass of each story is divided equally and lumped to the four corners. Two corrugated sheet shear walls were designed in each floor in each direction for resisting the lateral forces.



(a) wall without opening in sheathing (b) wall with opening in sheathing
Figure 7: OpenSees models for building archetypes

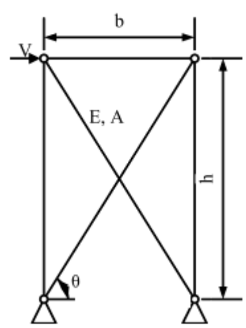


Figure 8: OpenSees model for shear wall

In the OpenSees models shear walls are modeled as a pin-connected panel with two diagonals as illustrated in Figure 8. The boundary members form a mechanism and the lateral stiffness and strength derive directly from the diagonals. The nonlinear shear wall $V—\Delta$ relationship can be expressed as a nonlinear one-dimensional $\sigma—\varepsilon$ relationship for the material in the diagonal members in Figure 8. The nonlinear behavior of the shear wall can be simulated by modeling the diagonal members with appropriate one–dimensional force-deformation hysteretic response characteristics. In this research, the Pinching 4 material model (Lowes and Altoontash 2003) in OpenSees is used for the diagonal members. Figure 9 shows a comparison of the OpenSees model with the test result for test No. 18. It can be seen that the model has a good agreement with the test result. Most importantly, the model is able to simulate the post-peak behavior of the shear wall.

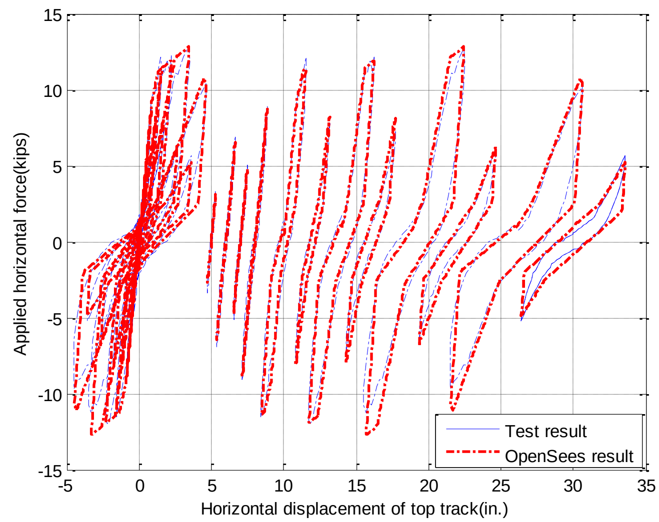
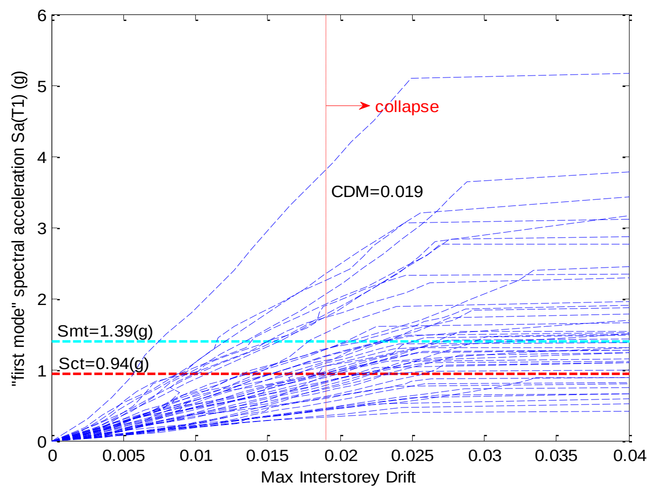
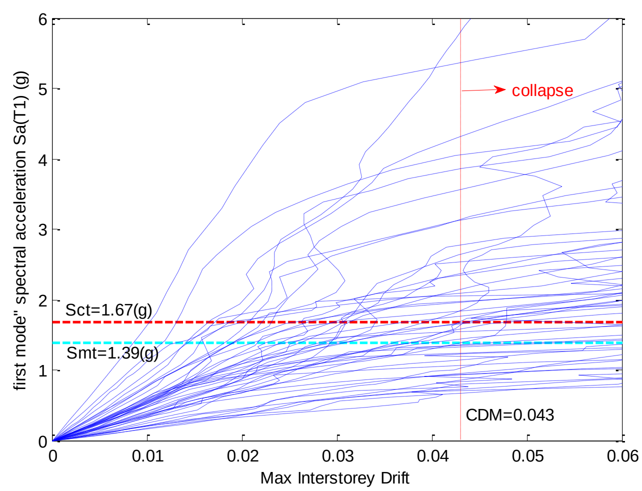


Figure 9: Comparison of OpenSees model with test

Building collapse is one of the major earthquake devastating consequences. Damages of buildings generally reflect the degree of earthquake disaster. As a result, the aseismic capacity of building structures, especially their capacity to prevent collapse, is of great importance to the seismic design of buildings. This research employed the capacity to prevent collapse of building as the indicator to compare the seismic performance of the two models. The Incremental Dynamic Analysis (IDA) described in FEMA P695 (2009) was used in the analysis. A total of 44 far-field earthquake records were used in the IDA. Figure 10 shows the IDA results for the two building archetypes. The spectral acceleration at collapse is obtained for each of the 44 curves. Based on the IDA results, the collapse fragility curves can be constructed as illustrated in Figure 11. The median collapse intensity, S_{CT}, is defined as the spectral acceleration causing 50% collapse probability. The ratio between the median collapse intensity (S_{CT}) and the Maximum Considered Earthquake (MCE) intensity (S_{MT}) is the collapse margin ratio (CMR). CMR is the primary parameter used to evaluate the collapse safety of the building design. The collapse fragility results indicate that the collapse probability of the 2-story office building at the MCE level will be reduced from 80% to 35% if the 24×2” slits are formed in the corrugated steel sheathing for shear walls. The dynamic analysis clearly demonstrates the advantage of the innovative shear wall for the CFS light framed buildings in seismic zones.

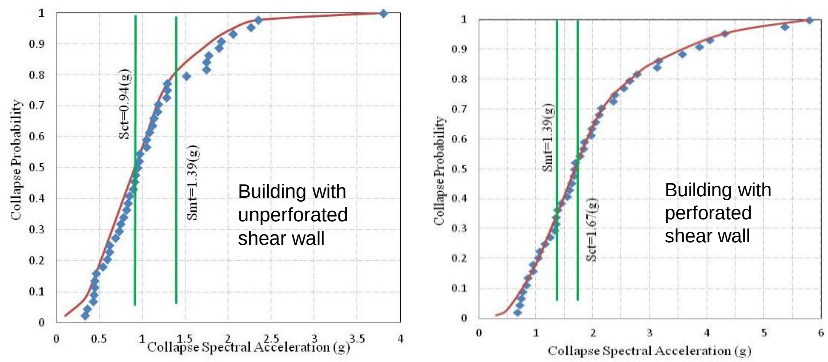


(a) wall without opening in sheathing



(b) wall with opening in sheathing

Figure 10: IDA curves



(a) wall without opening in sheathing (b) wall with opening in sheathing
Figure 11: Comparison of collapse fragility curves

4. Conclusion

CFS light framed shear wall sheathed by corrugated sheets with various opening configurations were experimentally examined and numerically. The research found that the walls using nonperforated corrugated sheets yielded significantly high strength and stiffness but poor ductility under cyclic loading. The shear walls demonstrated improved ductility when openings were introduced in the corrugated sheathing. On the other hand, the shear strength and stiffness may be reduced by the openings, particularly for circular hole configurations. The research discovered that with optimized slit opening in the corrugated sheathing, the shear wall could give desirable ductility and initial stiffness while maintaining relatively high shear strength. Nonlinear dynamic analysis was carried out to study two building archetypes: one with innovate perforated shear wall, the other with nonperforated shear wall. The analysis followed the FEAM P695 methodology with a focus on comparing the seismic performance against collapse. The analysis shows that building with the proposed perforated corrugated shear wall has largely reduced collapse probability, the innovative perforated corrugated steel sheets is a promising noncombustible sheathing solution for mid-rise CFS light framed buildings in high-earthquake or high-wind areas.

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