COLD-FORMED STEEL FRAMED SHEAR WALL SHEATHED WITH CORRUGATED SHEET STEEL

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Incombustibility is one important advantage of the sheet steel sheathed shear wall over wood panel sheathed shear wall. Compared to shear wall sheathed with plywood and OSB panel, shear wall sheathed with flat sheet steel behaved lower shear strength. Although shear wall sheathed with corrugated sheet steel exhibited high nominal strength and high stiffness, the shear wall usually behaved lower ductility resulting from brittle failure at the connection between the sheathing to frames.

This research is aimed at developing modifications on the corrugated sheathing to improve the ductility of the shear wall as well as derive practical response modification factor by establishing correct relationship between ductility factor μ and response modification factor R.

Totally 21 monotonic and cyclic full-scale shear wall tests were conducted during the winter break in 2012 by the author in NUCONSTEEL Materials Testing Laboratory in the University of North Texas. The research investigated nineteen 8 ft. × 4 ft. shear walls with 68 mil frames and 27 mil corrugation sheet steel in 11 configurations and two more shear walls sheathed with 6/17-in.OSB and 15/32-in. plywood respectively for comparison. The shear walls, which were in some special cutting arrangement patterns, performed better under lateral load conditions according to the behavior of ductility and shear strength and could be used as lateral system in construction.

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CHAPTER 1

INTRODUCTION

Cold-formed steel is widely used in buildings, automobiles, equipment, home and office furniture, utility poles, storage racks, grain bins, highway products, drainage facilities, and bridges. Its popularity can be attributed to ease of mass production and pre-fabrication, uniform quality, lightweight designs, economy in transportation and handling, and quick and simple erection or installation (New Steel 2007).

In building construction, cold-formed steel products can be into three categories: members, panels, and prefabricated assemblies. Typical cold-formed steel members such as studs, tracks, purlins, girts and angles are mainly used for carrying loads while panels and decks constitute useful surfaces such as floors, roofs and walls, in addition to resisting in-plane and out-of-plane surface loads. Prefabricated cold-formed steel assemblies include roof trusses, panelized walls or floors, and other prefabricated structural assemblies. Cold-formed steel possesses a significant market shear because of its advantages over other construction materials and the industry-wide support provided by various organizations that promote cold-formed steel research and products, including codes and standards development that is spearheaded by the American Iron and Steel Institute (New Steel 2007).

In residential and commercial construction, steel studs and tracks are generally covered with cladding to form a wall assembly with significant shear strength. It is common design practice to use this wall shear strength to resist lateral loads, such as those caused by wind and earthquake. Most commonly used walls constructed with materials included: (1) plywood and oriented strand board (OSB) on the exterior wall surfaces, (2) steel X-bracing on one side, (3) flat and corrugation steel sheathing on one side. In terms of the requirement for the using function, gypsum wall board (GWB) could be used as the interior wall, which assembly, to some extent, would improve the performance of the building under lateral loads. A typical cold-formed sheathing configuration is showed in Figure 1.

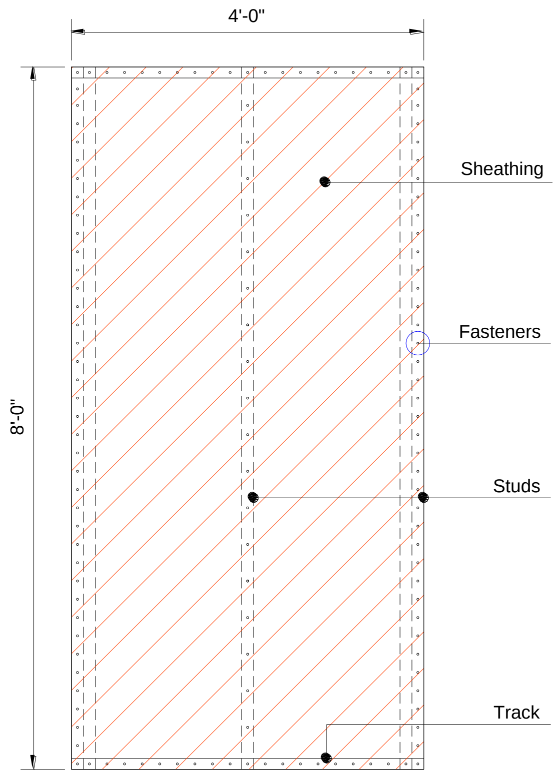


Figure 1. Shear wall assembly

The strength of a wall system highly depends on the interaction of many factors including size and strength of the sheathing; the strength, size of the frame members; the type, size and spacing of the fasteners used to fix the sheathing; and the shear wall aspect ratio (ratio of long to short dimension). Because of these variables, the design strength of shear walls is usually based on test of full-scale specimens.

Since the early 1990s, wide research works have been done investigating the behavior of cold-formed shear wall sheathed with OSB and plywood. Whereas, shear walls sheathed with steel corrugation, which could be the strongest lateral force resistant unit, still need intensive study to obtain a thorough understanding of the performance.

Based on fore-mentioned background, this research studied the cold-formed steel framed shear walls sheathed with corrugation steel sheet under monotonic and cyclic loading modes. The testing and the analyzing were presented in detail of this research. The research focus was on the improvement of the ductility of shear wall for seismic application.

This research work was organized as: chapter 1 gives the introduction of the cold-formed steel industry and the products used in construction and corresponding study background. The second chapter of the thesis reviewed the literatures which were conducted in relevant studies that have been done by some other researchers. Chapter 3 stated the research background and objective in terms of current research vacancy. Test program is stated in chapter 4 which includes test setup, test procedures, materials and test specimens. Test result and analysis was discussed in chapter 5. The sixth chapter of the thesis described the ductility and response modification factor of the shear wall assembly. Finally, chapter 7 provided the conclusions for this study and recommendations for further research on cold-formed frame shear walls sheathed with steel corrugation.

CHAPTER 2

LITERATURE REVIEW

Cold-formed steel (CFS) has become a favorite construction material for multi-family homes, mid-rise hotel and office buildings because of its advantages of light weight, high durability, fast site installation and relatively lower costs comparing with conventional materials, hot-rolled steel and concrete masonry. The lateral force resisting system in cold-formed steel construction typically adopts CFS frame members, sheet steel or woody sheathing and self-drilling screws which fastens them together.

Serrette (1996) investigated the behavior of CFS framed shear walls sheathed with plywood, OSB and GWB with a total of 48 tests in three phases. In phase 1, the goal of the program was to investigate the differences in static behavior of 15/32 in. plywood and 7/16 in. OSB shear walls. Four aspects behavior were examined: (a). Static strength of 8 ft. $\times$ 8 ft. OSB vs. plywood walls—sheathing on one side of the wall; (b). performance of the weaker of OSB and plywood with panels on one side of the wall; (c). performance of the weaker of OSB and plywood with studs framed at 24 in. and 16 in.—panels on one side of the wall; (d). performance of the 8 ft. $\times$ 8 ft. and 8 ft. $\times$ 4 ft. walls of the same panel (which was picked from the weaker of OSB and plywood)—panels on one side of the wall. The materials used in the test program included: 33ksi yield strength, 33 mil (20 gauge) 3.5 in. depth stud with 1.625 in. flange and 0.375 in. lip; 33ksi yield strength, 33 mil (20 gauge) 3.5 in. depth track with 1.25 in. flange; No.8 $\times$ 0.5 in. self-drilling framing screws and No.8 $\times$ 1 in. Flat Head plywood and OSB screws.

Phase 2 was conducted static tests by using the weaker of OSB and plywood panels which focused on: (i). behavior of OSB walls with small fastener spacing-panels on one side of the wall; (ii). behavior of OSB panels one side and GWB panels on the other side; (iii). behavior of walls with GWB panels on both sides. In the third phase, comprehensive investigations were carried out on the panels with OSB and plywood, covering all fastener schedules. The author concluded that: the nominal capacity of the 8 ft. × 8 ft. plywood wall was approximately 17% higher than that of the 7/16 in. OSB wall; Plywood walls presented much larger deformation capacity at the maximum load compared to the OSB wall. Compared to the wall with panels parallel to studs, OSB wall with panels in perpendicular installation has a higher load and deformation capacity; Among the 8 ft. × 4 ft. OSB walls, a denser fastener schedule pattern exhibited a higher maximum load capacity; Attaching the GWB on the other side of OSB wall, could improve the capacity of the shear with 6 in./12 in. fastener schedules, whereas no significant increase in capacity was observed due to the addition of GWB panel; In the cyclic test under a given screw schedule, the plywood walls had generally outperformance corresponding OSB walls approximately 10% in their capacity.

Serrette (1997) initiated a wide range investigation which, in addition the OSB and plywood wall, included flap strap X-braced walls and steel flat sheathed walls. The comprehensive test program was categorized into five phases on the expected projects objective:

Phase 1 (cyclic): Examine the performance of 8 ft. x 4 ft. 15/32 in. plywood and 7/16 in. OSB wall assemblies framed with 0.033 in. middle studs and 0.043 in. back to back end studs in fasteners schedules 3 in./12 in. and 2 in./ 12 in.

Phase 2 (cyclic): For panels attached with No. 8 screws, investigated at what thickness does the behavior of the shear wall system change to establish the limit on framing members thickness (up to 0.054 in.).

Phase 3 (monotonic & cyclic): Examine the performance of 0.033 in. flat strap X-braced walls framed with 0.033 in. and 0.043 in. studs.

Phase 4 (monotonic & cyclic): shear walls sheathed with 0.018 in. and 0.027 in. steel sheets were investigated in this step. And also those were the first whole steel- composed shear walls which ever have been tested.

Phase 5 (monotonic & cyclic): Observed the behavior of high aspect ratio (4:1) walls sheathed with 15/32 in. plywood and 7/16 in. OSB.

In phase 1, the shear wall assemblies failed in the screw heads pulling through the plywood and OSB panels and this failure mode resulted in the detachment of the wood board along the chord studs and bottom track of wall assembly. It was strong enough of the back-to-back 0.043 chord studs to prevent the member from local buckling. In phase 2, 7/64 in dia. predrilled holes were employed for the No. 8 screws, since the 0.054 in. studs were too tough to drill though. The failure in the 0.054 in. framed assemblies resulted from shearing of the screws and the screw heads pulling through the sheathing. While a combination of the screws pulling out of the bottom track and chord studs and screw heads pulling though the sheathing was observed. Both the static and cyclic tests were carried out on 4-1/2 in. wide strap and 7-1/2 in. wide strap in phase 3. Under static load, the assemblies with 4-1/2 in. wide strap failed in the local buckling of the compression chord stud. Due to out-of-plane bending, the failure in assemblies with 7-1/2 in. wide strap resulted from local buckling of the top chord track in the test. For assemblies with the 4-1/2 in. wide strap possessed the identical failure mode whatever the static test and cyclic were carried out. In cyclic test, a combined failure mode of local buckling in the chord stud and local buckling in the top track due to out-of-plane force was recorded. In the static test of the fourth phase, metal sheathing on high aspect ratio wall assemblies (8 ft. × 2 ft.) were investigated. The metal sheathing deformed so significantly that resulted in unzipping of sheathing due to the rupture at the sheathing edges. But when the fastener schedule was reduced from 6 in. /12 in. to 4 in. /12 in., the length of unzipped shorten and occurred at the corner. In the meanwhile, local buckling in the chord studs was viewed. In addition to the failure mode observed during the static test, fasteners pulling out of the framing were recorded as well. In phase 5, in either static and cyclic, reduced fastener schedule transited the failure from the unzipped to chord stud buckling.

Based on above observation, the author concluded that: in the 8 ft. × 4 ft. plywood and OSB assembly tests, with the using of back-to-back 0.043 in. chord studs, the plywood walls were found to be stronger and more ductile than the OSB walls. No. 8 screws performed well in 0.043 in. studs but fractured in shear when 0.054 in. studs were employed. So larger diameter screws would be better matched accessories to 0.054 in. or even bigger studs. Eccentricity installation on one side tends to put both the chord studs and track in strong axis bending. The eccentricity imperfection, plus the usually higher actual yield strength of the strap, drags the track out of plane which leading to in premature failure of the wall assembly. As for the steel sheathed wall assemblies, failure resulted from a combination of bearing in the sheet steel along the edges and pullout of screws from the studs. No tension field action was visible in the tests. High aspect ratio wall are capable of resisting high loads at relatively large displacement, however, after large events, the wall has low to zero initial stiffness.

Serrette (2002) conducted a series of tests which aimed at evaluating the performance of new wall configurations not permitted in the building codes then. The test program included four areas: reversed cyclic performance of shear wall framed with 54 mil and 68 mil studs and sheathed with 7/16 in. OSB one side; Reversed cyclic performance of shear wall framed with 54 mil and 68 mil studs and sheathed with 7/16 in. OSB double sides; Reversed cyclic performance shear walls sheathed with 2 pieces of 27 mil sheet steel with simple lap shear connections at the abutting edges of the panels (the abutting edge perpendicular to framing); Monotonic performance of 1/2 in. GWB sheathed shear walls (one side) with different blocking configurations and fastener spacing patterns. The OSB wall tests indicated that No.8 screws and 54 mil framing, No. 10 screws and 68 mil framing matched very well and the failure in connection has good ductile character. For the double-sided OSB walls framed with 54 mil studs, demands on the chord studs exceeded the capacity of the studs resulting in failure at the location of punch-out. While walls framed with 68 mil studs, screws fastening the hold-down to the chord studs failed. The two premature failure modes in the double-sided OSB shear wall assemblies prevented of development sheathing full strength. In the sheet steel walls, diagonal shear buckling and diagonal tension were observed. The tension field crossed the adjoining edge and unzipped the joint before the sheet steel could reach its capacity. Failure mode in the GWB monotonic tests were consistent to that were observed in the previous tests.

The North American Standard for Cold-Formed Steel Framing - Lateral Design (AISI S213-07) provides shear strength values for CFS framed shear walls with three type of sheathing materials: 15/32 in. Structural 1 plywood sheathing, 7/16 in. oriented strand board (OSB), and 0.018 in. and 0.027 in. flat steel sheet. Those published values were based on Serrette (1996, 1997, and 2002).

L.A. Fülöp and D. Dubina (2004) directed six series full-scale test on 11.81 ft. × 7.87 ft. (3600 mm × 2400 mm) shear walls constructed with different sheathing. For all of test specimens tested in the investigation, the walls were framed with U154/1.5 tracks and C154/1.5 studs. In test Series I, corrugated sheets were horizontally sheathed on the frames with 4.8 mm diameter self-drilling screws. Fasteners were scheduled in every corrugation rib along the vertical end studs, every other corrugation rib in the vertical field studs and 7.87 in. (200 mm) along the top and bottom tracks, as well as the corrugation horizontal overlap joints. In addition to the identical corrugation configuration and screws schedules to specimens in test series I, GWB were placed on the other side to walls in test Series II. In test Series IV, same corrugation and screw patterns were adopted except a 3.94 ft. (1200 mm) door opening was cut in the wall middle. Traditional diagonal bracing straps with section area 4.33 in. $\times$ 0.059 in. (110 $\times$ 1.5 mm^2) were employed in test Series III. In test Series I, along the end studs, corrugation warping was observed. Large deformation concentrated in the seam vicinity resulting in failure occurred in one of the seams. Load converged on vertical connecting screws on end studs after the failure in seam, made the corrugated sheathing unzipped vertically in end studs. Because of the attachment of GWB on the interior side, walls in test Series II demonstrated 17.8% higher peak load in comparison with that attained in test Series I. In test Series III, vertical component of the load in strap were transferred to end studs and resulted in the failure concentrated entirely in the corner connection area. Although plastic elongation of the straps was viewed, this unexpected failure of the corner connection make the behavior of the wall unable to reflect the anticipated capacity and ductility character from this shear wall configuration. In test Series IV, the behavior of walls was similar to those in Series I and II. The peak load value reduced about 20% compared to that reported in Series I. In Table 2 of L.A. Fülöp and D. Dubina (2004), ductility factors corresponding to each test were presented with a range from 4.39 to 7.78.

Stojadinovic and Tipping (2007) conducted 44 cyclic racking tests on CFS shear walls sheathed with corrugated sheet steel. 40 of the specimens measured 8 ft-2 in. $\times$ 4 ft. and the other 4 specimens were with the dimension of 8 ft-2 in. $\times$ 2 ft. Materials, the author used for the tests, included Grade 50, 27 mil, 33 mil and 43 mil corrugation sheet steel, 50ksi yield strength SSMA(The Steel Stud Manufacturers Association) studs 362S162-33, 362S162-43, 362S162-54 and 362S162-68. The fasteners applied in the tests were No.10-16 × ¾ in., No.12-14 × 1 ¼ in., No.14-20 × 1 ½ in. hex head self-drilling screws at 3” on center along the perimeter frames and seams but 6” on center in the middle stud. Two specimens were constructed with 5/8 in. GWB applied over the corrugated metal sheathing to evaluate the effect on the strength and stiffness of the specimens. The GWB was attached to the sheathing with No.6 screws spaced at 6 in. along the panel edges and field, a different attachment pattern from those in Serrette’ tests. And two walls were tested with corrugation sheathing on both sides. In the test, as the panels cyclically deformed, screws gouge elongated holes in the metal studs and/or sheeting due to racking shear. And because of the increment of the inter-story drift, warping of the end corrugation became evident and coinciding diagonal tension and compression fields developed across the panel. As the holes around the fasteners enlarged, tensile capacity of the screws was reduced and eventually that resulted in the “popping” out (pulling out) of the screws along the boundary members due to the distortion of the corrugated sheet steel. Meanwhile, the author also given a recommendation of some relevant factors that employed in the design of seismic force resisting systems, which have the design parameters: Response Modification Factor (R) = 5.5, System Overstrength Factor (Ω_o) = 2.5, Deflection Amplification Factor (C_d) = 3.25.

Since 2006, Nippon Steel (JHU-Nippon CFS Meeting, October 11th, 2011) conducted a series investigation on cold-formed shear walls with different metal sheathing configuration which aimed to be applied as the lateral force resisting unit in 4 to 5-story buildings. Various steel sheets were examined to see their behavioral characteristics, including flat sheet steel, sheet steel with slits, sheet steel with holes, but none of them showed preferable behavior due to large local deformation of sheets and members, decreased strength and stiffness, etc. However, their study on horizontally placed corrugated panels proved the sheathing configuration has superior performance. Although the failure mode resulted from the bearing of the metal sheet could provide ductility to some degree, the shear wall behaved in a slip manner due to the bearing in sheet which possessed less energy absorption capacity and large stiffness degradation.

Yu (2009) performed 8 tests on 8 ft. × 4 ft. cold-formed shear walls with horizontally placed metal sheathing on one side. Three configurations for the boundary studs used in Yu's test as shown in Figure 2. The configuration A used two studs connected back-to-back by No.8 screws one pair 6 in. o.c. along member. The outer stud was strengthened by a matching track member fastened to the stud flanges, face-to-face, by No. 8 screws 6 in. on center. The configuration B used three studs, two studs were attached back-to-back by No. 8 screws 6 in. on center, and the third stud attached to the double studs face-to-face by ½ in. length stitch weld every 12 in. on center. The boundary stud configuration C used double studs, back-to-back, fixed by No.12 screws one pair 6 in. on center. Simpson Strong-Tie S/HD 10S and S/HD15 hold-down were used in shear wall to resist the uplift force. The corrugated sheet steel was manufactured by Vulcraft Manufacturing Company. The deck type is 0.6C, 0.027 in. corrugated steel sheet with 9/16 in. rib height. The sheathing was installed one side of the wall. For each shear wall specimen, the sheathing was composed of three corrugated steel sheets which overlapped one rib and were connected by a line of screws. The screw spaced 2.5 in. on the panel edges due to the decking profile, as well as that in the overlap joint and along the top and bottom tracks. Screws were scheduled every other corrugation along the middle stud, i.e. 5 in.

In the tests, corrugated sheathing showed significantly higher shear strength and larger displacement at peak load (Table 3.) than that of other sheathing materials. Test 3 gave the lowest shear strength of 1389 plf, which was still greater than the published values in Table 2 (AISI S213, 2007) 1000plf, 1235plf, 1330plf for the 0.027 in. flat sheet steel, the 7/16 in. OSB, and the 15/32 in. Structural 1 sheathing respectively. However the corrugated sheet test failed in buckling of the studs and joint connections which resulted in immediate drop in load after peak and then caused a lower ductility factor. The ductility of representative flat and corrugated sheet sheathed CFS shear walls and that of OSB sheathed CFS shear walls are listed in Table 4.

Table 1. Test matrix for shear wall test in Yu (2009)

Test Label (protocol)	Nominal Framing thickness	Sheathing and Framing Fastener[2]	Fastener Spacing	End Studs Config[3]	Hold-down
1 (monotonic)	43 mils	#8 ¾”	5”/12 ½”	A	S/HD10S Raised
2 (monotonic)	43 mils	#8 ¾”	5”/12 ½”	B	S/HD10S Raised
3 (cyclic)	43 mils	#8 ¾”	5”/12 ½”	B	S/HD10S Raised
4 (monotonic)	68 mils	#12 1-¼”	2 ½”/5”	C	S/HD10S Raised
5 (cyclic)	68 mils	#12 1-¼”	2 ½”/5”	C	S/HD10S Raised, Reinforced
6 (cyclic)	68 mils	#12 1-¼”	2 ½”/5”	C	S/HD10S Raised, Reinforced
7 (cyclic)	68 mils	#12 1-¼”	2 ½”/5”	C	S/HD10S Flushed, Reinforced
8 (monotonic)	68 mils	#12 1-¼”	2 ½”/5”	C	S/HD15 Flushed
Note: 1- all tests used 0.027 in corrugated sheet with rib height 9/16 in. for sheathing; 2- #8 screws were modified truss head self-drilling screws, #12 screws were hex washer head self-drilling screws; 3- stud configuration refers to Figure 4.					

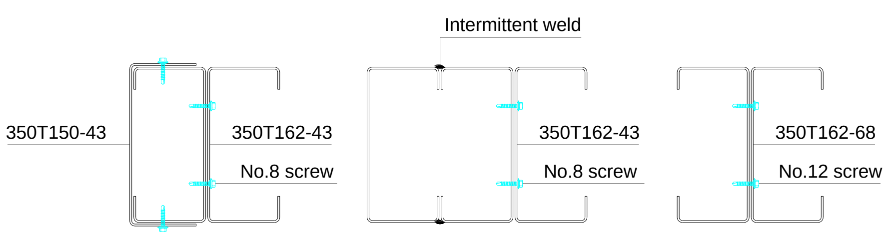


Figure 2. Boundary stud configurations in Yu (2009)

Table 2. Nominal shear strength (Rn) for shear walls (AISI S213,2007)

United States and Mexico

Nominal Shear Strength (R_n) for Seismic and Other In-Plane Loads for Shear Walls [1,4,7,8]

(Pounds Per Foot)

Assembly Description	Max. Aspect Ratio (h/w)	Fastener Spacing at Panel Edges[2] (Inches)				Designation Thickness[5,6] of Stud, Track and Blocking (mils)	Required Sheathing Screw Size
		6	4	3	2		
15/32" Structural 1 sheathing (4-ply), one side	2:1[3]	780	990	-	-	33 or 43	8
	2:1	890	1330	1775	2190	43 or 54	8
						68	10
7/16" OSB, one side	2:1[3]	700	915	-	-	33	8
	2:1[3]	825	1235	1545	2060	43 or 54	8
	2:1	940	1410	1760	2350	54	8
	2:1	1232	1848	2310	3080	68	10
0.018" steel sheet, one side	2:1	390	-	-	-	33 (min.)	8
0.027" steel sheet, one side	4:1	-	1000	1085	1170	43 (min.)	8
	2:1[3]	647	710	778	845	33 (min.)	8

Table 3. Summary of shear wall test results in Yu (2009)

Test Label	Peak load (plf)		Lateral deflection at peak load (in.)		Avg. Peak Load (plf)	Avg. Δ (in.)	Failure Mode
	+P	-P	+Δ	-Δ			
1	1942	-	2.85	-	1942	2.85	Stud buckled
2	1625	-	2.60	-	1625	2.60	Sheathing screw pullout
3	1628	1150	1.75	1.39	1389	1.57	Sheathing screw pullout
4	2451	-	0.81	-	2451	0.81	Hold-down screws sheared
5	3717	3656	1.28	1.30	3688	1.29	Lateral support failed
6	3957	3986	2.73	2.54	3972	2.64	No failure
7	4113	4315	2.84	3.12	4214	2.98	Hold down failed
8	4804	-	3.20	-	4804	3.20	Sheathing joint failed

Table 4. Ductility of representative shear wall specimens in Yu (2009)

Specimen	Test	Screw Spacing	400 (in/in)	F400 (plf)	Fy (plf)	e (in/in)	max (in/in)	μ
68 mil 4’x8’ 0.027” Corrugated	Mono-tonic	5”/12”	0.0025	805	4148	0.0129	0.0347	2.70
68 mil 4’x’8 7/16” OSB	Cyclic	2”/12”	0.0025	1229	2968	0.0061	0.0189	3.11
43 mil 4’x8’ 0.027” Corrugated	Mono-tonic	5”/12”	0.0025	213	1791	0.0211	0.0305	1.44
43 mil 4’x8’ 0.027” Flat sheet	Mono-tonic	4”/12”	0.0025	339	1015	0.0075	0.0225	3.00

CHAPTER 3

RESEARCH OBJECTIVE

The research works conducted by Tipping and L.A. Fülöp showed that the corrugated metal sheathing shear walls have good ductility and high shear strength and could be used as lateral system applied in seismic zones. However, either the material used in L.A. Fülöp or the shear wall construction method of Tipping was not the typical practice in the North American construction market.

Previous research done by Yu et al. (2009) indicated that the corrugated steel sheathing can provide significantly higher strength and stiffness compared to the conventional OSB, plywood sheathing and flat steel sheet sheathing. However brittle failures were observed in Yu's research and further research is needed to improve the ductility of the corrugated steel sheet shear walls in order to use it in seismic zones. This research is aimed at developing modifications on the corrugated sheathing to improve the ductility of the shear wall as well as derive practical response modification factor by establishing correct relationship between ductility factor μ and response modification factor R. The tasks conducted in this research included 2 major steps:

(1) Conducted full scale tests on shear walls with different opening configurations, and produced nominal shear strength, drift limit, hysteresis parameter for CFS shear walls by using modified corrugated steel sheathing.

(2) Calculated the ductility of shear walls with different opening configurations, optimized the best opening configurations and derived the practical response modification factor.

To avoid failure mode in the boundary elements and connections which usually results in a suddenly drop after the peak load, a method was employed to improve the ductility of the corrugated CFS shear wall by forcing failure mode to be material yielding and local buckling in the sheathing.

Introducing openings in the field of sheathing will be considered as an alternative method as well to improve the ductility. The similar approach has been studied by a number of researchers on hot-rolled steel plate shear walls. Hitaka and Matsui (2003) conducted a total of 43 hot-rolled steel plate shear walls under monotonic and cyclic lateral loading to investigate the shear wall behavior.

Varied opening sizes, locations, and orientation on the corrugated sheathing will result in different strength and stiffness of shear wall. Also, the location of buckling and initial yielding in the sheathing material is affected by those same configurations of openings. Because of the light gauge of the CFS materials, it is feasible to make openings on the site and no special tools are required. Additionally, the openings probably are utilized as the paths for duct, plumbing, and electrical wires.

This research focused on how to find appropriate opening locations so that the shear strength of shear wall will not drop significantly while the ductility was improved. Material yielding and out-of-plan buckling around the opening were designed intentionally to be the dominant failure mechanism and the method of energy dissipation in the new corrugated CFS shear wall. Both circular openings and slits were considered in this research.

For each shear wall specimen, the sheathing was composed of three corrugated steel sheets which overlapped one rib. Instead of opening in the sheathing field, shear wall which without stitch screws along the overlap joints was another case that was investigated.

The work in step 1 included full scale tests on the CFS shear wall with different opening patterns. The full-scale tests started from the shear wall without stitch screws at the connection joints. Because the task of this research was to optimize the desired opening configuration, some unplanned opening patterns were developed as the tests progressed.

The work in step 2 developed a practical equation associated the ductility factor μ and response modification factor R. The desired opening configurations were decided and the responding response modification factor was derived accordingly.

CHAPTER 4

TEST PROGRAM

The test program of this research was conducted during the time period from November 2012 to February 2013 in the NUCONSTEEL Materials Testing Laboratory at the University of North Texas, Denton Texas. A total of 21 monotonic and cyclic shear wall tests were included in the scope of this research, of which the author was responsible for the testing and data explanation. For each shear wall configuration, minimum one monotonic and one cyclic test were carried out to ensure a minimum level of reliability / validity of the test data.

With the intension to develop the optimal opening pattern on the sheathing, the shear wall configurations were designed as the test events were performed accordingly. For particular wall configurations, because of their attracted behavior character, additional specimen was built and to acquire sufficient validation.

4.1 Test Setup

The monotonic tests and the cyclic tests were conducted on a 16 ft. span, 12 ft. high self-equilibrating steel testing frame installed in the NUCONSTEEL Materials Testing Laboratory of the University of North Texas. The testing frame was equipped with one MTS® 35-kip hydraulic actuator with ±5-in. stroke. A MTS® 407 controller and a 20-GPM MTS® hydraulic power unit were employed to drive the loading system. A 20 kips TRANSDUCER TECHNIQUES® SWO universal compression/tension load cell was pin-connected the actuator loading shaft to the T shape load beam. Five NOVOTECHNIC® 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 data acquisition system consisted of a National Instruments® unit (including a PCI6225 DAQ card, a SCXI1100 chassis with SCXI1520 load cell sensor module and SCXI1540 LVDT input module) and an HP Compaq desktop. The applied force and the five displacements were measured and recorded instantaneously during the test.

The test wall was bolted to the base beam and loaded horizontally at the top. The base beam was 5 in. × 5 in. × ½-in. structural steel tube and was bolted to a W16 × 67 structural steel beam which was anchored to the concrete floor. One web of the structural tube base beam was cut out in several locations to provide access of the anchor bolts connecting hold-downs to the base beam. Figure 3 illustrates the test setup of a typical 8 ft. × 4 ft. CFS shear wall assembly. Figure 4 shows the front view of the test frame with an 8 ft. × 4 ft. steel shear wall and Figure 5 shows the back view of the test installation.

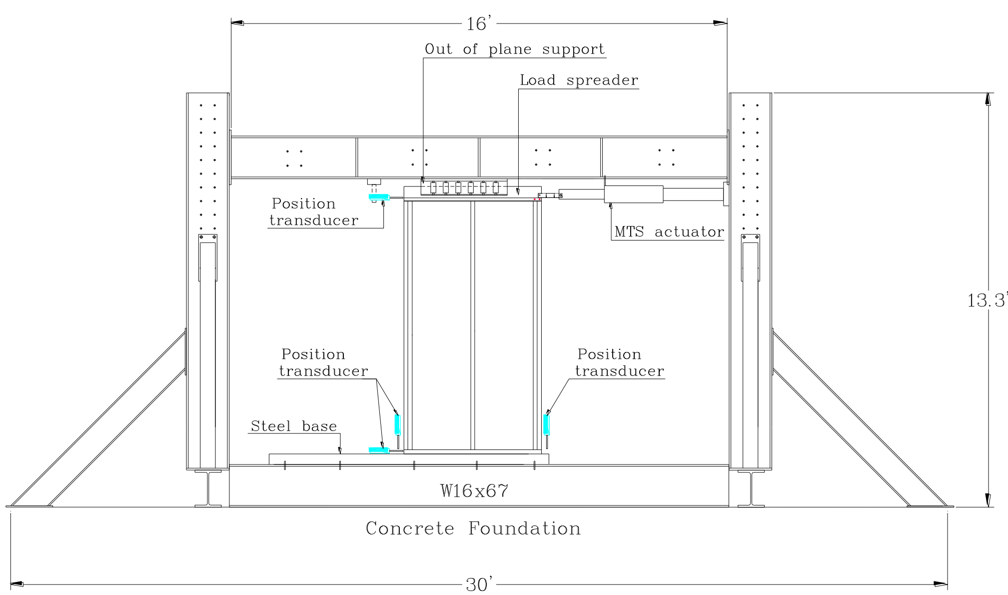


Figure 3. Testing frame with 8 ft. × 4 ft. wall assembly

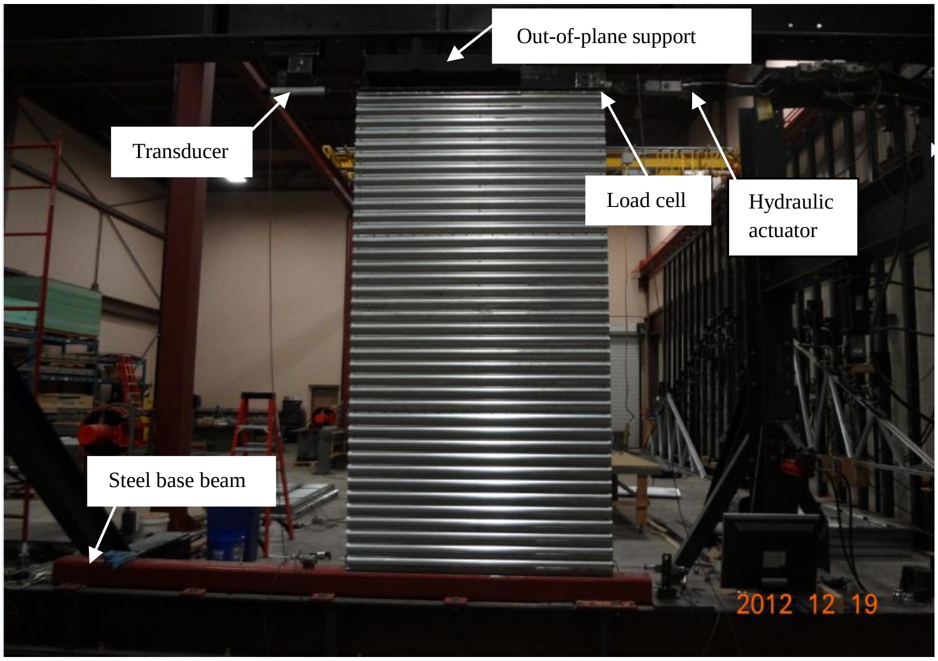


Figure 4. Front view of the test setup

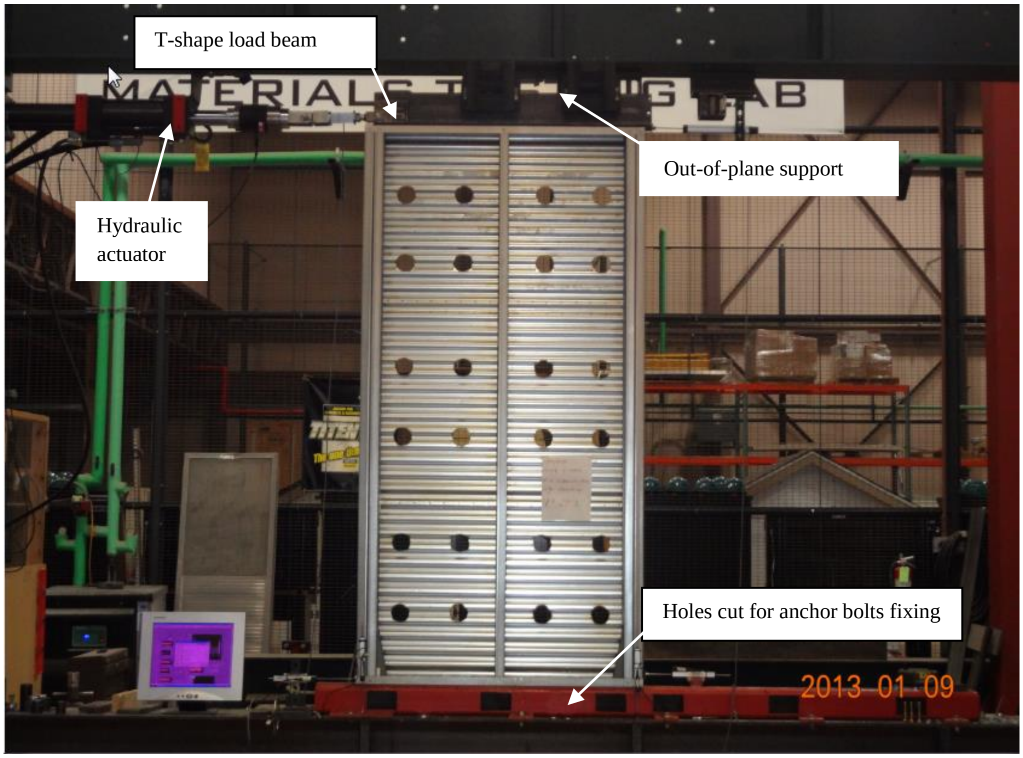


Figure 5. Back view of the test setup

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.12-14 × 1 ¼ in. hex head self-drilling screws scheduled every 3 in. on center. Consequently, a uniform linear racking force could be transmitted to the top track of the shear wall. The stem of the T-shape beam was placed in the 1.0 in. gap between the rollers located on the front and back side of the shear wall specimen so that the out-of-plane movement of the wall was prevented by the rollers. The rotation of the rollers could not only reduce the friction generated from the T-shape beam in plane movement to avoid stuck during the test but also worked as a guide for the loading T-shape as well. Two Simpson Strong-Tie ® S/HD15S hold-down with 33 pre-drilled holes corresponding to No.14-14 × 1 in. hex head self-drilling were used as the anchorage system.

4.2 Test Procedure

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 (2006) "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 ICC-ES AC130 (2004), was chosen for the reversed cyclic tests. The CUREE basic loading history shown in Figure 6 includes 43 cycles with specific displacement amplitudes, which are listed in Table 5. The specified displacement amplitudes are based on a percentage of the ultimate displacement capacity determined from the monotonic tests. The ultimate displacement capacity is defined as a portion (i.e. γ=0.60) of maximum inelastic response, Δ, which corresponds to the displacement at 80% peak load. However, the CUREE protocol was originally developed for wood frame structures, and it was found in this test program that using $0.60\Delta_m$ as the reference displacement was not large enough to capture the post peak behavior of the sheet steel walls in the cyclic test. Therefore, the lesser of 2.5% of the wall height (2.4-in.for 8 ft. high wall) and the displacement at the peak load in the monotonic tests was used as the reference displacement in the CUREE protocol. 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 5. CUREE basic loading history

Cyclic No.	***% Δ***	**Cyclic No.**	***% Δ***	**Cyclic No.**	***% Δ***	**Cyclic 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*		

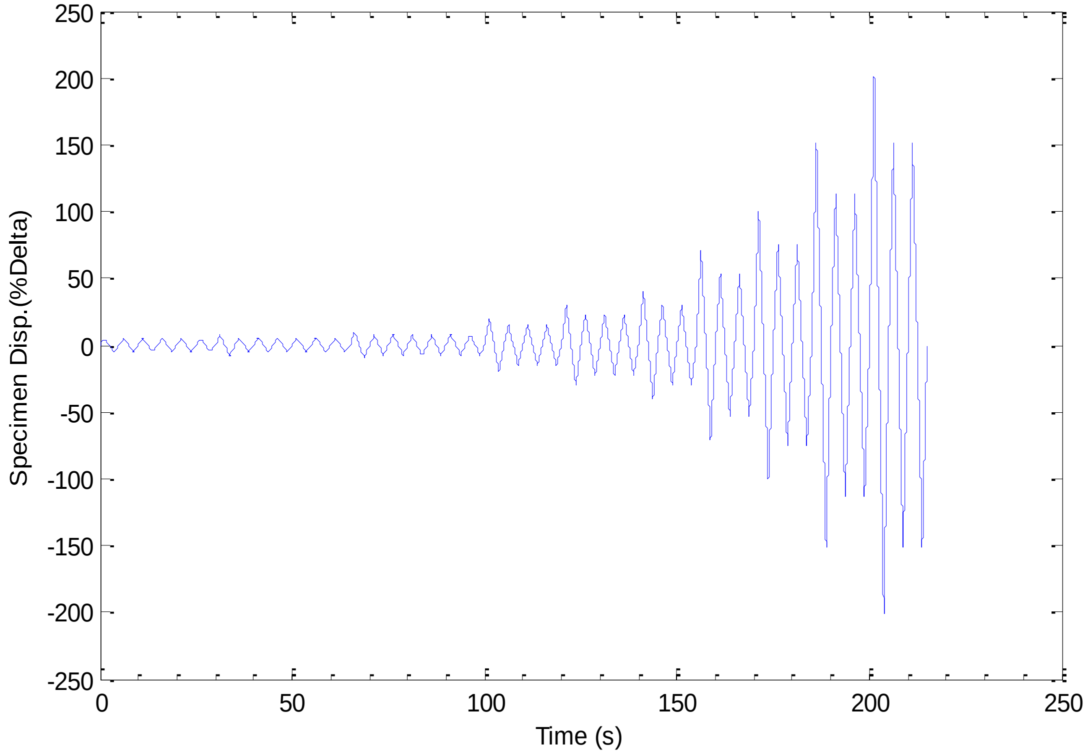


Figure 6. CUREE basic loading history (0.2 Hz)

4.3 Test Specimens

The specimens tested in this research just included one type dimension: 8 ft. (high) $\times$ 4 ft. (wide) (2:1 aspect ratio). Test No.1 to No.19 used 0.027 in. thickness low profile (Figure 8) corrugated sheet steel and Test No.20 & 21 used 15/32 in. 4-ply plywood and 7/16 in OSB respectively. The corrugated sheet steel (metal decking) was manufactured by Vulcraft manufacturing company. The deck type was 0.6C, 0.027 in. (22 gauge) corrugated steel sheet with 9/16 in. rib height (Figure 7). The sheathing was installed one side of the wall. For each wall specimen, the sheathing was made of three corrugated steel sheets which were connected by single line of screws. Due to the metal sheathing profile, the spacing of the screws was limited to a 2.5 in. module along the members. Fastener spacing at horizontal seams used 2.5 in. as well while 5 in. fastener spacing was used along the middle stud. 12 types of the sheathing opening configurations were designed and examined in this research as listed in Table 6: Type 1: corrugated sheathing without holes; Type 2: corrugated sheathing without stitch fasteners along the horizontal seam joint; Type 3: corrugated sheathing with six 6-in. diameter circular holes; Type 4: corrugated sheathing with six 4-in. diameter holes; Type 5: corrugated sheathing with six 6-in. length vertical slits; Type 6: corrugated sheathing with twenty-four 3-in. diameter holes; Type 7: corrugated sheathing with twenty-four 3-in. vertical slits; Type 8: corrugated sheathing with twenty-four 3-in. horizontal slits; Type 9: corrugated sheathing with twelve 2-in. vertical slits; Type 10: corrugated sheathing with twenty-four 1-in. vertical slits; Type 11: corrugated sheathing with twenty-four 2-in. vertical slits; Type 12: one plywood and one OSB shear wall for comparison. Table 6 listed the test matrix and Figure 8~19 shows the opening configuration of the corrugation sheathed steel framed shear wall. Enlarged figures of the wall configurations were attached in Appendix C.

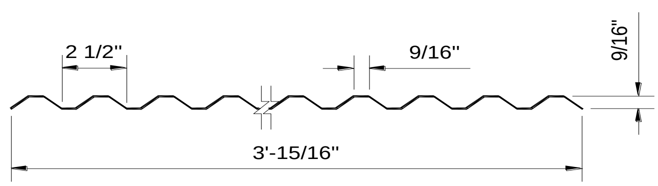


Figure 7. Corrugated sheet steel profile

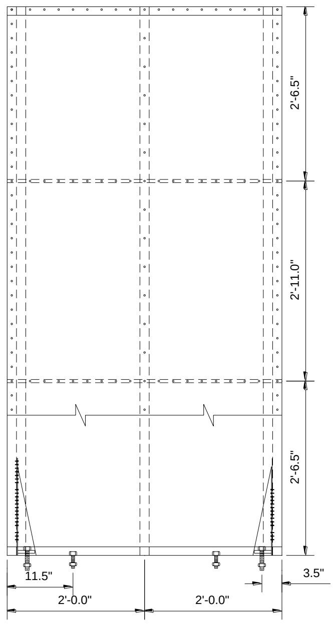


Figure 8. Wall configuration Type 1

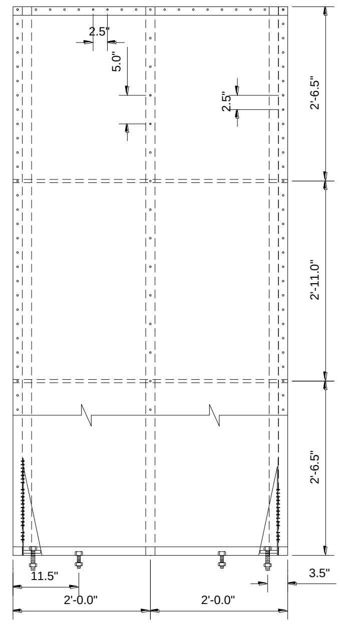


Figure 9. Wall configuration Type 2

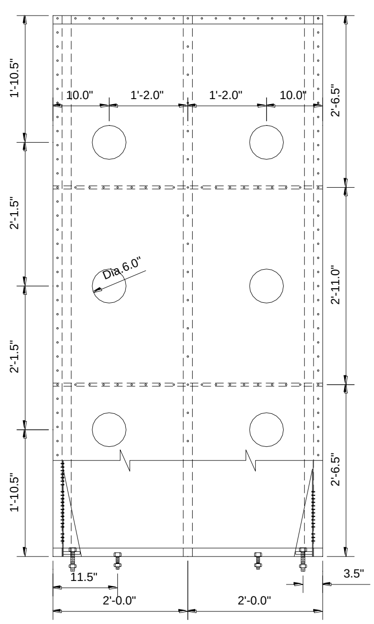


Figure 10. Wall configuration Type 3

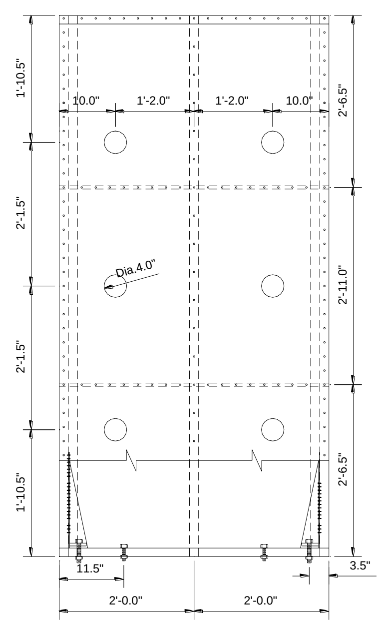


Figure 11. Wall configuration Type 4

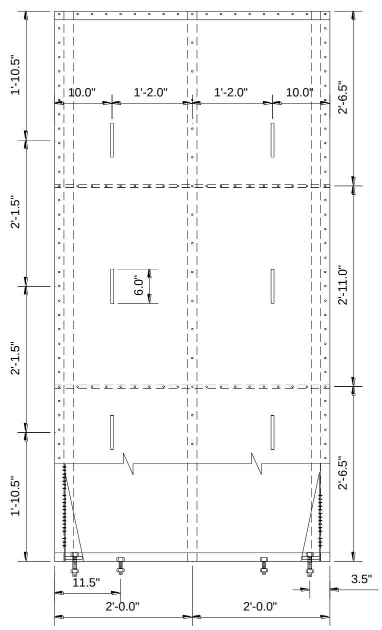


Figure 12. Wall configuration Type 5

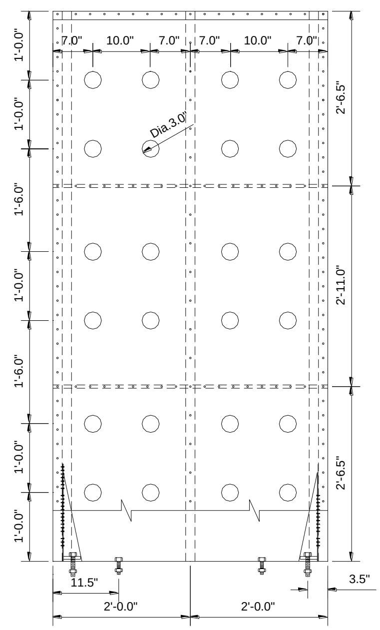


Figure 13. Wall configuration Type 6

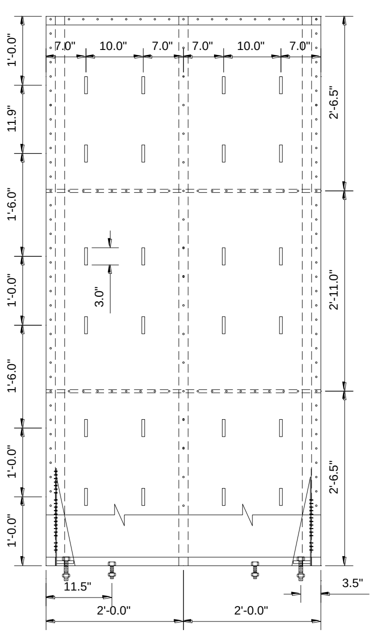


Figure 14. Wall configuration Type 7

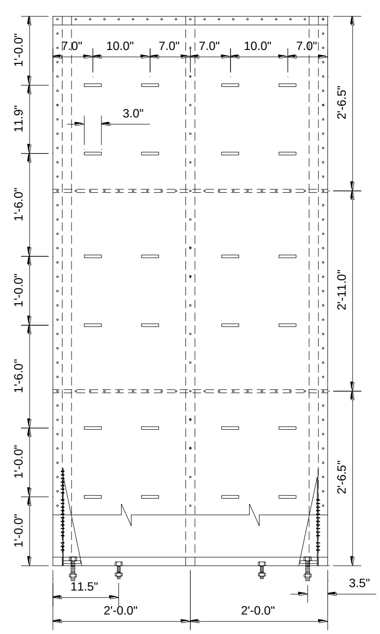


Figure 15. Wall configuration Type 8

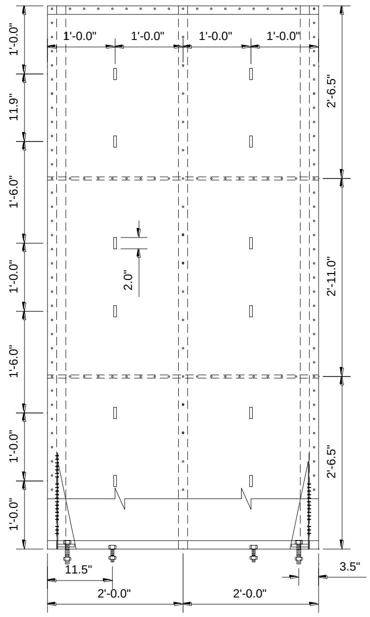


Figure 16. Wall configuration Type 9

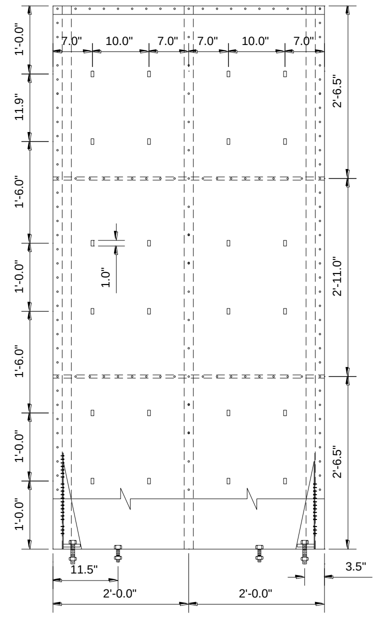


Figure 17. Wall configuration Type 10

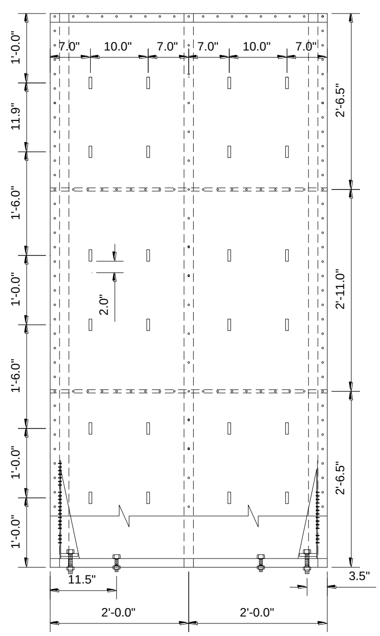


Figure 18. Wall configuration Type 11

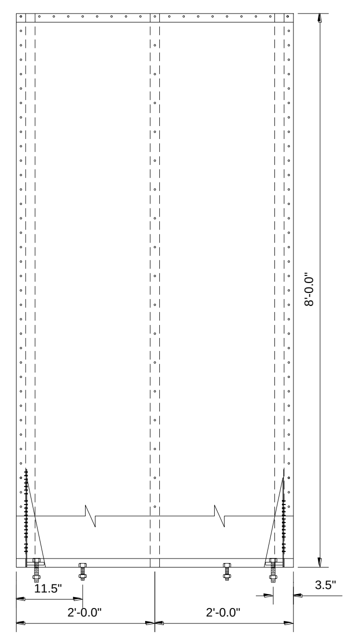


Figure 19. Wall configuration Type 12

Above figures show the dimensions of the corrugation sheathed steel framed shear wall, opening pattern and size, anchor bolts, and the hold-downs. Both boundary studs used double C-shaped studs fastened together back -to-back with No. 12 ×1 in. hex head self-drilling screws pairs at 6-in on center. While the middle stud used one C-shaped stud at the half width of the wall. One U-shaped steel member used as top and bottom tracks respectively. Studs were inserted into tracks and flanges were attached with No. 8 × 18-1/2 in. modified truss head self-drilling screws on the front side while No. 12 ×1 in. hex head self-drilling screws, which assembly constructed the shear wall frame at back side. 0.068-in. thickness SSMA (Steel Studs Manufacturers Association) standard framing members were adopted as the wall assembles. Two Simpson Strong-Tie® S/HD15S (Figure 20) hold-down were attached to both boundary studs from inside by using a total of 33 - No. 14×1 in. hex washer head self-drilling screws. Since shear failure resulting from the screws connecting hold-down to boundary studs occurred in Yu (2009), in this research, the connection was strengthened by welding the hold-down to boundary studs at the stud punch-out location and hold-down top edge (Figure 21). Also the material strengths were obtained by coupon tests on the untested but same batch of materials at the end of this test program.

Figure 20. Simpson Strong-Tie® S/HD15S

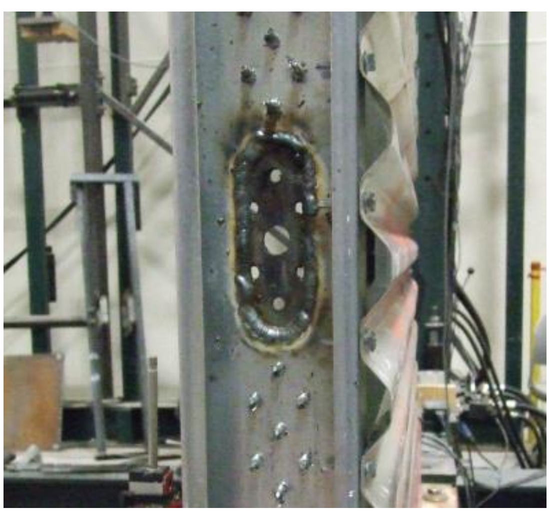


Figure 21. Simpson Strong-Tie® S/HD15S Hold-down welding connection

Table 6. Test matrix for shear wall test

Test lable	Opening configuration	Test protocal	Test Number	Nominal framing thickness	Nominal sheathing thickness	Fastener spacing	Hold-down
Type 1	no opening	Monotonic-ASTM E564	No.2 & 12	68 mil	27 mil	2.5"/5"	S/HD15S
		Cyclic-CUREE	No.7 & 19				
Type 2	no seam screws	-	-	68 mil	27 mil	2.5"/5"	S/HD15S
		Cyclic-CUREE	No.1				
Type 3	6x6" circular holes	Monotonic-ASTM E564	No.3	68 mil	27 mil	2.5"/5"	S/HD15S
		Cyclic-CUREE	No.4 & 8				
Type 4	6x4" circular holes	-	-	68 mil	27 mil	2.5"/5"	S/HD15S
		Cyclic-CUREE	No.5				
Type 5	6x6" vertical slit	-	-	68 mil	27 mil	2.5"/5"	S/HD15S
		Cyclic-CUREE	No.6				
Type 6	24x3" circular holes	-	-	68 mil	27 mil	2.5"/5"	S/HD15S
		Cyclic-CUREE	No.9				
Type 7	24x3" vertical slit	-		68 mil	27 mil	2.5"/5"	S/HD15S
		Cyclic-CUREE	No.10 & 13				
Type 8	24x3" horizontal slit	-	-	68 mil	27 mil	2.5"/5"	S/HD15S
		Cyclic-CUREE	No.11				
Type 9	12x2" vertical slit	-		68 mil	27 mil	2.5"/5"	S/HD15S
		Cyclic-CUREE	No.14				
Type 10	24x1" vertical slit	-	-	68 mil	27 mil	2.5"/5"	S/HD15S
		Cyclic-CUREE	No.16				
Type 11	24x2" vertical slit	Monotonic-ASTM E564	No.18	68 mil	27 mil	2.5"/5"	S/HD15S
		Cyclic-CUREE	No.15 & 17				
Type 12	plywood & OSB	-	-	68 mil	27 mil	2.5"/5"	S/HD15S
		Cyclic-CUREE	No.20 & 21				

A small length curve was created between flange and web when the U-shaped track was cold formed from a thin flat sheet steel. When the 8 ft. long studs were inserted into the track opened flanges, the studs end contacted the track web tightly and a gap between them was existed. This gap, plus the web thickness made the shear wall height a little longer than 8 ft., i.e. 8 ft. 0.2 in. Considering the original manufacturing width of the corrugated sheathing 3ft.-15/16 in., one shear wall should comprise of three corrugated sheets but need to be cut off a length to fit the shear wall height. Figure 22 shows the detailed cutting pattern of the metal sheathing. Figure 23 shows the utilization of plasma cutter in the cutting of sheathing edge and circular holes while Figure 24 shows that a grinder with 0.045 in. thickness sand blade was used in the cutting of slits. The cutting pattern of slits used less labor time and simple machines and it was easy to control the cutting quality, moreover there is hardly limitation to carry out this work procedure on the job site. Whereas in the cutting of circular holes, the sheathing should either be pre-cut by a special machine before the wall is constructed or cut by a very skilled worker after constructed but it is very hard to control the cutting quality. Figure 25 illustrating gives the cutting width of 3-in. long vertical slits and an average width 0.059-in. was adopted in this research.

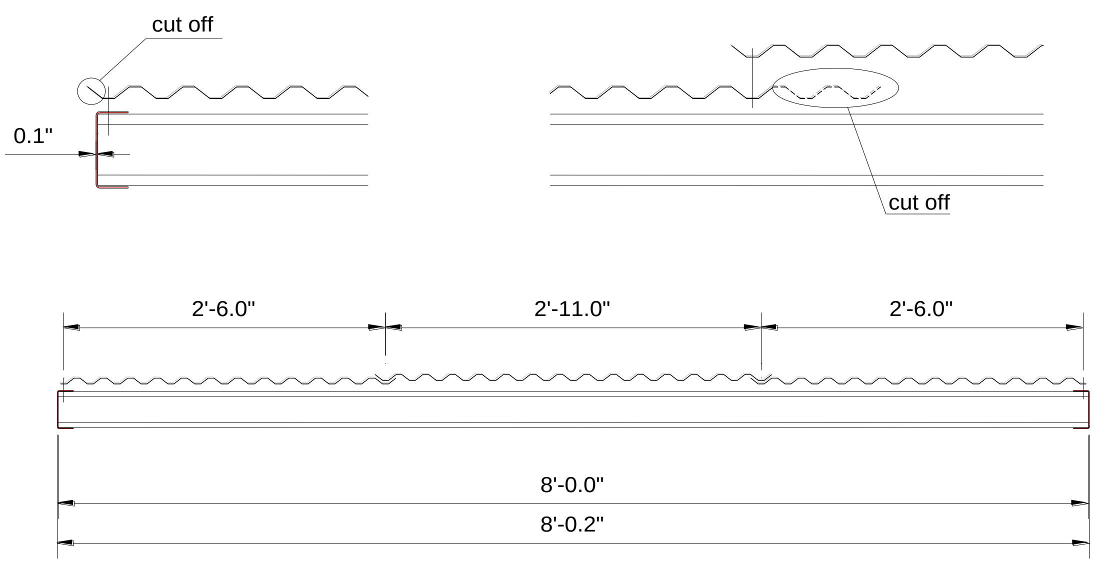


Figure 22. Corrugation cutting pattern

Figure 23. Plasma cutter operation

Figure 24. Grinder cutter operation

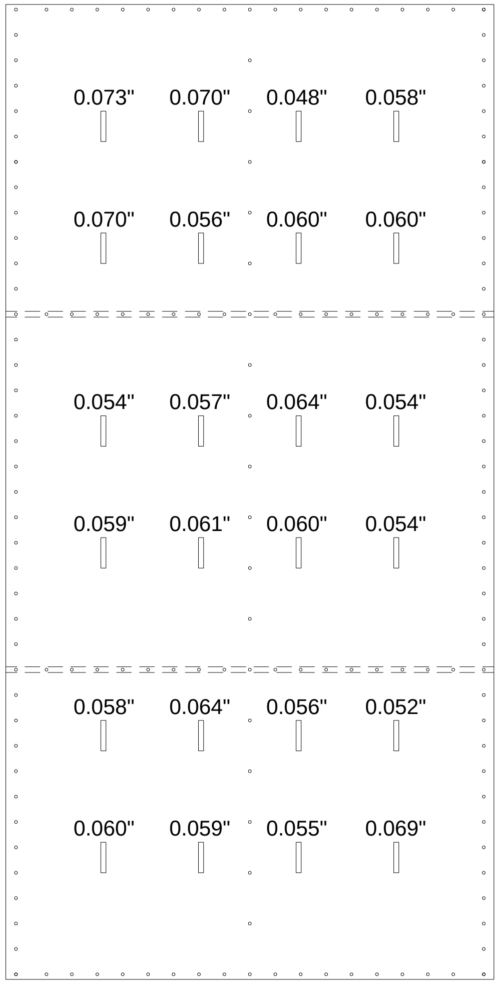


Figure 25. Cutting width of slits

The details of the components of the tested steel sheet walls are given as follows:

Studs:

- 350S162-68 SSMA structural stud, 0.068-in. 3-1/2-in. × 1-5/8-in. made of ASTM A 1003 Grade 33 steel.

Tracks:

- 350T150-68 SSMA structural track, 0.068-in. 3-1/2-in. × 1-1/2-in. made of

ASTM A 1003 Grade 33 steel.

Sheathing:

- 0.027-in. thick ASTM A1003 Fy= 90 ksi high strength steel.
- Steel sheet was installed on one side of the wall assembly.
- 15/32-in. thick plywood.
- 7/16-in. thick OSB.

Framing and Sheathing Screws:

- No. 12×1-in. hex washer head screws. Spacing along panel edge is 2.5-in. o.c.. Spacing in the field of the sheathing is 5-in. for all specimen configurations.

Hold-Downs:

- Simpson Strong-Tie® S/HD15S hold-downs with 33 - No. 14×1-in. HWH self- drilling screws, and were welded to boundary studs with 1/8-in. fillet weld around the stud punch-out and top edge of the hold-downs.

Tension Anchor Bolts:

- 5/8-in. diameter Grade 8 anchor bolts with standard cut washers and nuts. Two bolts were used for each wall assembly to anchor the boundary studs to base beam.
- 3/4-in. diameter Grade 8 anchor bolts with standard cut washers and nuts. Two bolts were used for each wall assembly to anchor the boundary studs to base beam.

Shear Anchor Bolts:

- 5/8-in. diameter Grade 5 anchor bolts with standard cut washers and nuts. Two bolts were used for each wall assembly.

4.4 Material Properties

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 were summarized in Table 7. The coating on the steel was removed by hydrochloric acid prior to the coupon tests. The coupons test specimens were tensioned on the INSTRON® 4482 universal testing machine. An INSTRON® 2630-106 extensometer was employed to measure the tensile strain. The tests were conducted in displacement control at a constant rate of 0.05 in./min. A total of three coupons were tested for each member, and the average results are provided in Table 7.

Table 7. Material properties

Component	Uncoated Thickness (in.)	Yield Stress Fy (ksi)	Tensile Strength Fu (ksi)	Fu/Fy Ratio	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%

All the coupons meet the minimum ductility requirement by North American Specification for Design of Cold-Formed Steel Structural Members (2007) Edition (NASPEC 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%.

CHAPTER 5

TEST RESULTS AND DISCUSSION

A total of 21 monotonic and cyclic tests were conducted in this research. The specimens tested in this research included one type dimension: 8 ft. (high) × 4 ft. (wide) (2:1 aspect ratio). Test No.1 to No.19 used 0.027 in. thickness low profile corrugated sheet steel and Test No.20 & 21 used 15/32 in. 4-ply plywood and 7/16 in OSB respectively. Fastener spacing along boundary studs and at horizontal seams used 2.5 in. while 5 in. fastener spacing was employed along the middle stud. The test results for this research are summarized in Table 8. The displacements in Table 8 represent the lateral displacement of the wall top at the peak load. The ductility factor, μ, is defined as the ratio of the ultimate displacement (Δ_u) and the yield displacement (Δ_y), $\mu=\Delta_u/\Delta_y$. The response modification factor R, was calculated from the ductility factor and expressed by equation $R=\sqrt{2\mu - 1}$. The Δ_u and Δ_y are determined in accordance with ASTM E2126 (2007). The observed failure mechanism is listed in Table 9. The detailed test results are provided in Appendix A, in which measured responses of all of the tested shear walls, Matlab EEEP plotting, and related photos showing shear wall behavior are included.

Type 1 configuration, shear wall without any opening on the 27 mil corrugated sheathing, included 4 individual tests, two monotonic and two cyclic. In No.2 monotonic test, when the shear wall peak load reached 17 kips resulting 39.18 kips applied force in the tension anchor bolt, the tension bolt of the hold-down was broken. The tensile capacity of Grade 8 5/8-in. diameter bolt is 46.02 kips which is greater than the actual applied force 39.18 kips. Since the anchor bolts have been used in previous cyclic tests, this failure probably resulted from the high fatigue stress in the bolts. In No.7 cyclic test, to avoid the bolts tension failure, two new Grade 8 5/8-in. diameter bolts were employed. In the test, one anchor broken in the positive actuator stroke at first post-peak loop and another bolt failed in the negative actuator stroke at first post-peak loop as well. The working load in the anchor were 39.65 kips and 39.24 kips respectively, which value are still less than the tensile capacity. The test results from test No.2 & No.7 should not be included in further analysis because the failure modes were not expected.

Table 8. Summary of shear wall test results

wall configuration	test number	peak load (lbf)		lateral deflection at peak load (in.)		average peak load (lbf)	average deflection (in)	average stiffness (lb/in.)
		+p	-p	+Δ	-Δ			
Type 1	No.2_no holes_monotonic	4152.5	-	2.326	-	4152.5	2.326	9195
	No.12_no holes_monotonic	5007.5	-	3.032	-	5007.5	3.032	10879
	No.7_no holes_cyclic	4265.0	4312.5	2.490	2.278	4288.8	2.384	10430
	No.19_no holes_cyclic	5257.5	4807.5	2.800	2.326	5032.5	2.563	10971
Type 2	No.1 no stitch screws	2276.3	2102.3	2.585	2.598	2189.3	2.592	8601
Type 3	No.3_6x6 holes_monotonic	3222.5	-	3.097	-	3222.5	3.097	5399
	No.4_6x6 holes_cyclic	3295.0	3002.5	2.820	2.266	3148.8	2.543	6333
	No.8_6x6 holes_cyclic	3027.5	2817.5	3.027	2.314	2922.5	2.671	6892
Type 4	No.5_6x4 holes_cyclic	3882.5	3582.5	2.865	2.167	3732.5	2.516	8489
Type 5	No.6_6x6 slits_cyclic	2865.0	2640.0	2.153	1.586	2752.5	1.870	8045
Type 6	No.9_24x3 holes_cyclic	3027.5	2850.0	3.461	3.187	2938.8	3.324	5678
Type 7	No.10_24x3 slits_cyclic	2950.0	2925.0	3.223	3.308	2937.5	3.266	8568
	No.13_24x3 slits_cyclic	3180.0	2747.5	2.598	2.290	2963.8	2.444	8310
Type 8	No.11_24x3 horiz. slits_cyclic	4130.0	4182.5	1.921	2.010	4156.3	1.966	11132
Type 9	No.14_12x2 slits_cyclic	3920.0	3217.5	2.138	1.584	3568.8	1.861	11392
Type 10	No.16_24x1 slits_cyclic	4757.5	4475.0	2.432	2.339	4616.3	2.385	11129
Type 11	No.18_24x2 slits_monotonic	3092.5	-	2.801	-	3092.5	2.801	8480
	No.15_24x2 slits_cyclic	3207.5	2982.5	3.465	2.150	3095.0	2.808	11126
	No.17_24x2 slits_cyclic	3110.0	3095.0	2.861	2.315	3102.5	2.588	9987
Type 12	No.20_plywood_cyclic	3505.0	2927.5	3.263	2.337	3216.3	2.800	6939
	No.21_OSB_cyclic	3595.0	3282.5	3.336	3.542	3438.8	3.439	8318

Table 9. Failure mode of the tested shear walls

wall configuration	Test number	Observed failure mode
Type 1	No.2, 7, 12 & 19	two specimens failed in the breaking of hold down bolts. In other two test, the sheathing buckled, screws in the middle studs were pulled over.
Type 2	No.1	the upper sheet steel was gouged slot hole due to the reverse cyclic movement, screws in the boundary studs were pulled over.
Type 3	No.3, 4, 8	large relative out-of-plane movement of the sheet elements at the both sides of circles , resulting in the rupture of sheathing.
Type 4	No.5	large relative out-of-plane movement of the sheet elements at the both sides of circles, resulting in the rupture of sheathing. some screws were pulled over through head.
Type 5	No.6	large relative out-of-plane movement of the sheet elements at the both side of vertical slits, resulting in the rupture of sheathing.
Type 6	No.9	large relative out-of-plane movement of the sheet elements at the both side of circles, resulting in the rupture of sheathing. screws along the lower seam joint became loose.
Type 7	No.10 & 13	large relative out-of-plane movement of the sheet elements at the both side of vertical slits, resulting in the rupture of sheathing. screws along the lower seam joint became loose.
Type 8	No.11	specimen failed in the breaking of hold down bolts. No other failure was observed.
Type 9	No.14	large relative out-of-plane movement of the sheet elements at the both side of vertical slits, resulting in the rupture of sheathing. The slits which was aligned vertically connected together.
Type 10	No.16	the lower sheet steel overall buckled and unzipped.
Type 11	No.15, 17 & 18	large relative out-of-plane movement of the sheet elements at the both side of vertical slits, resulting in the rupture of sheathing. Some screws became loose.
Type 12	No.20	shear failure resulting in rupture of sheathing horizontally. Then sheathing failed in bearing around screws on the boundary stud.
	No.21	screws were pulled through head along boundary stud and bottom track.

In No.12 and No.19 tests, Grade 8 3/4-in. diameter bolts were applied, which have theoretically minimum tensile capacity 66.27 kips. In test No.12, on the bottom corrugated sheet, shear bucking was first observed and became evident with the increasing of horizontal shear force. When the peak load was achieved, the metal sheathing failed suddenly with large out-of-plane deformation accompanied by screws pulling over. And a 30 degree half-wave appeared. Same failure mode was observed in No.19 cyclic test. From the validated test No. 12 & 19, shear walls without any opening were found to be stronger but less ductile.

Figure 26. Failure mode of wall configuration Type 1

Shear wall Type 2**,** because no stitch screws were applied along the horizontal overlap joints, the metal sheets worked individually while not a whole sheathing under external force. In the test, large relative horizontal movement was found between every two adjacent sheets. In below Figure 27, because of missing of stitch screws, horizontal shear force could not be transferred to bottom track, the force in the screws along the both edges made the boundary studs in bending about the weak section axis. Screws at the sheet corner have the maximum shear force. The metal sheet was gouged long slots on the vertical edge and middle horizontal edge due to large relative movement. When the shear wall works in a real building, vertical force due to live load and dead load will also be applied on the studs. Without the stitch screws along the overlap joint, the metal sheathing will not supply enough restraint of the studs, the effective length of boundary members under compression will increase. This will result in premature failure of the wall assembly before development of shear wall capacity. Based on the failure mode and the wall capacity and stiffness, this configuration was concluded as undesired and improvement in the opening configuration was needed.

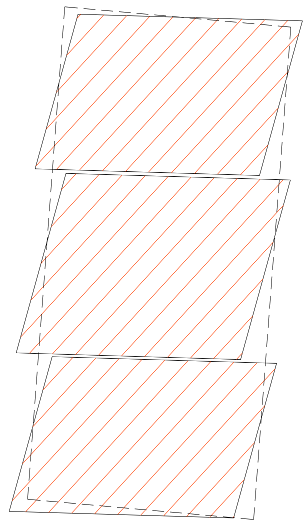


Figure 27. Relative deformation of metal sheet of shear wall

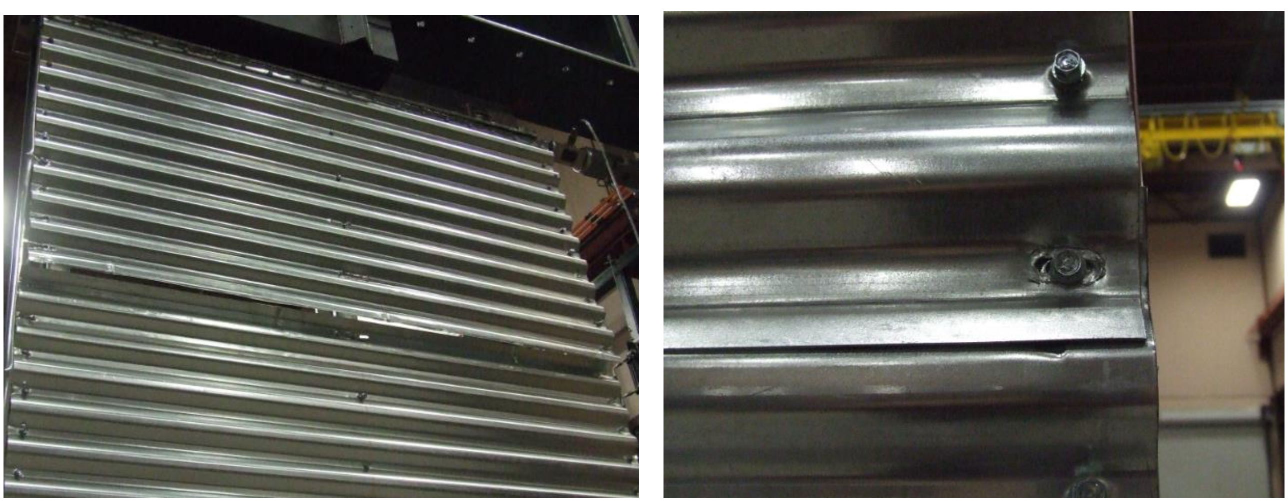


Figure 28. Failure mode of wall configuration Type 2

For Type 3 shear wall with six 6-in. diameter circular openings, one monotonic and two cyclic specimens were tested. The circular opening weakened the sheathing integrity and reduced the out-of-plane stiffness. In the test, relatively large out-of-plane movement of the corrugation portion mirrored by vertical diameter of the circular was observed. The relative deformation would be incremental with the increasing of the shear wall top displacement. The materials of sheet steel at top and bottom point in the circular opening perimeter yielded first and then passed harden stage and at last the steel elongation exceeded material capacity. And then the sheet was torn apart at top and bottom point of the circle due to out-of-plane deformation. The average shear wall strength of Type 3 tested wall was 3181 plf. which was greater than the highest recommendation value of 3080 plf. of 7/16-in. OSB sheathing shear wall from the AISI S213-07. But the stiffness degraded 37% and peak load dropped about 39.5% compared with no-opening shear wall.

Figure 29. Failure mode of wall configuration Type 3

Type 4 shear wall with six 4-in diameter circular opening had the failure mode as Type 3. In addition, screws on the lower sheet-to-sheet connection joint became loose. Even though the stiffness and peak load improved in comparison with walls in Type 3, the ductility factor reduced from 2.415 to 2.039 which make the opening configuration be dropped.

Figure 30. Failure mode of wall configuration Type 4

Type 5 shear wall with six 6-in vertical slits, has the same opening arrangement pattern as Type 3. The 6-in cutting caused the two portions of the sheathing along the slits free edge and make it unstable and be apt to buckle under in-plane shear force. Large relative out-of-plane movement of the sheet elements at the both side of vertical slits was observed. The rupture started from the two end points of slits and extended vertically up and down. The two adjacent slits aligned in vertical not connected together after both were stretched longer than their original dimension. Comparing to 6×6-in circular opening, the shear wall stiffness increased, but it didn't present an expected higher ductility factor.

Figure 31. Failure mode of wall configuration Type 5

The cutting pattern 24×3-in circular opening of shear wall Type 6 derived from the "equal opening area", i.e. twenty-four 3-in. diameter circle equal to six 6-in. diameter circle in area. Those twenty-four circular openings spread on the sheathing uniformly, so the stiffness reduction of the metal sheet at each opening location was not as higher as that of 6-in. circular holes. Same failure mode as that of 6-in holes wall was observed, and similar shear wall stiffness and ductility exhibited in this 24x3-in pattern wall.

Figure 32. Failure mode of wall configuration Type 6

With the concept of the cutting “small size but more quantity”, 24x3-in vertical slits cutting configuration was applied in shear wall test Type 7, which cutting arrangement was identical to 24x3-in holes in Type 6. Also, the stiffness reduction of the metal sheet at each cutting location was lower than that of 6-in vertical slits in shear wall Type 5. Same failure mode as that of 6-in vertical slits wall was found, but because the original short cutting did not weaken the wall a lot, the slits were extended progressively and the shear wall stiffness degraded gradually. And a good average ductility of 3.532 was achieved.

Figure 33. Failure mode of wall configuration Type 7

A high expectation was given to shear wall Type 8, 24x3-in. horizontal slits, but the shear wall behaved same as wall test No.7 in Type 1. The 5/8-in. bolts failed in tension and the wall did not exhibit any damage which means the short horizontal slits will not affect the shear wall behavior.

Figure 34. Failure mode of wall configuration Type 8

In order to improve the shear wall stiffness and at the same time remain good ductility, 12×2-in vertical slits were cut in shear wall Type 9. Same failure mode of wall with slits was observed. Firstly, the sheathing at slits was torn apart and it developed up and down. The torn slits extended so fast that slits aligned in vertical connected together at last. The long stretched slits (Figure 35) from the connected individual made a huge drop of the peak load 30% (Figure 36) between the two adjacent loops. The connected slits developed up to the overlap joint and down to the bottom track.

Figure 35. Failure mode of wall configuration Type 9

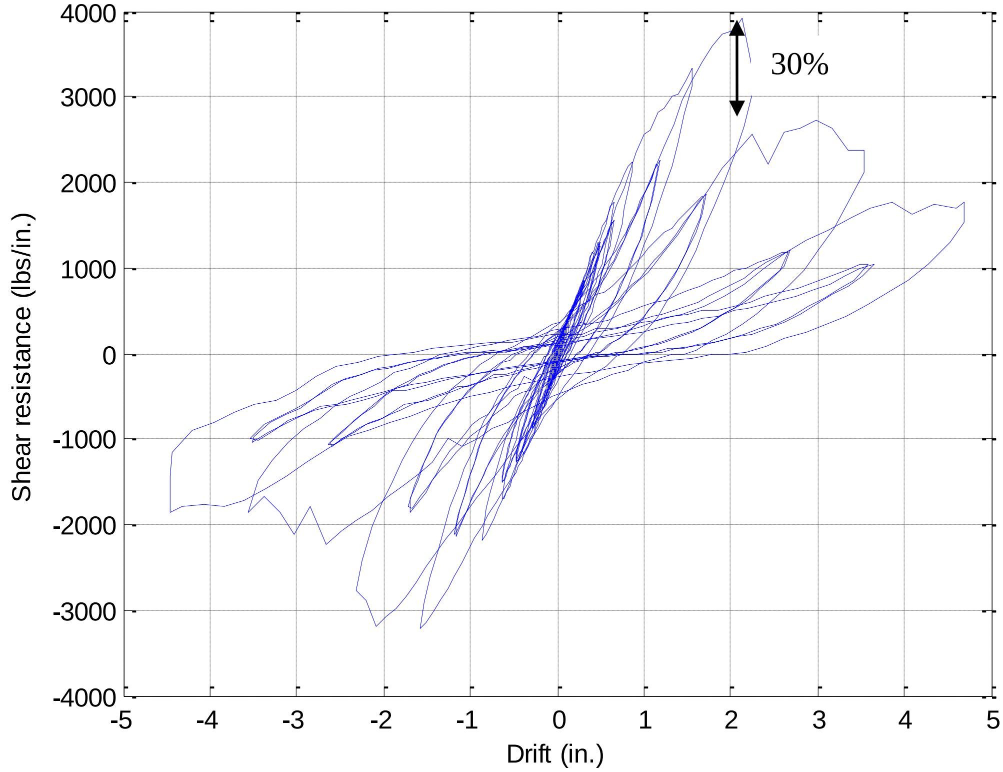


Figure 36. Hysteresis curve of wall configuration Type 9

Type 10, 24x1-in. vertical slits shear wall was tested in No. 16. Same as the failure mode in the no-opening shear wall test No.12 and 19, the metal sheathing of test No.16 buckled due to in-plane shear force and that resulted in the screws pulling over along the edge and middle studs and the sheet steel detached at those connection joints. The shear wall showed high peak load and stiffness but weak ductility. No slits was observed to be torn apart, it could conclude that vertical slits which can't be cut through the corrugation rib will not affect the shear wall behavior.

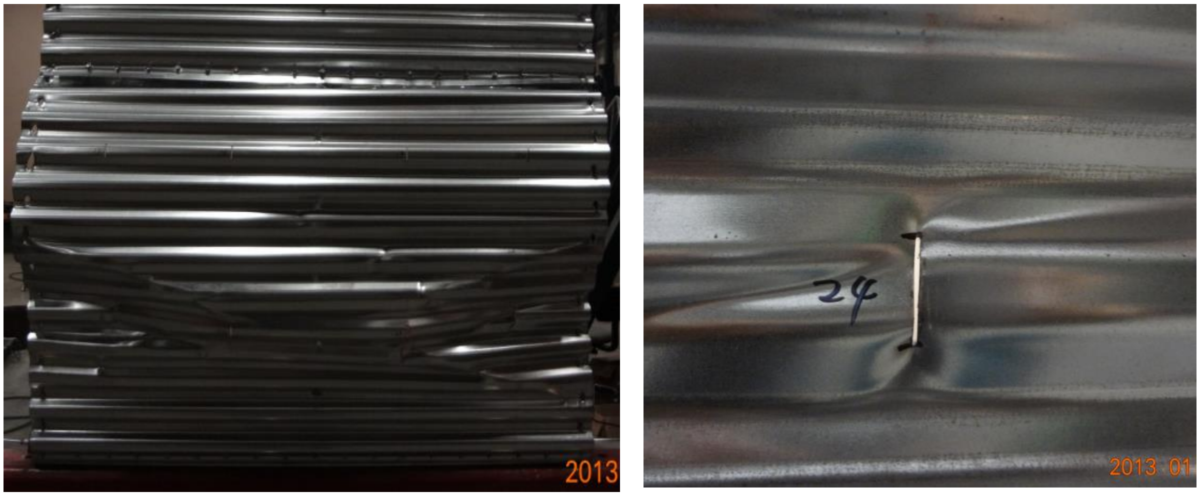


Figure 37. Failure mode of wall configuration Type 10

Type 11, 24x2-in. vertical slits shear wall included three tests No.15, 17 and 18. The metal sheathing ruptured due to relative out-of-plane movement of the sheet at both sides of the slits. Some screws along the edges and sheet connection joint became loose. Figure 38 shows the hysteresis curve of the specimen under reverse cyclic loading, the wall displayed a good character of energy dissipation and gradual stiffness degradation. An averaged peak load of 3125.6 plf. and an averaged ductility of 3.158 were obtained.

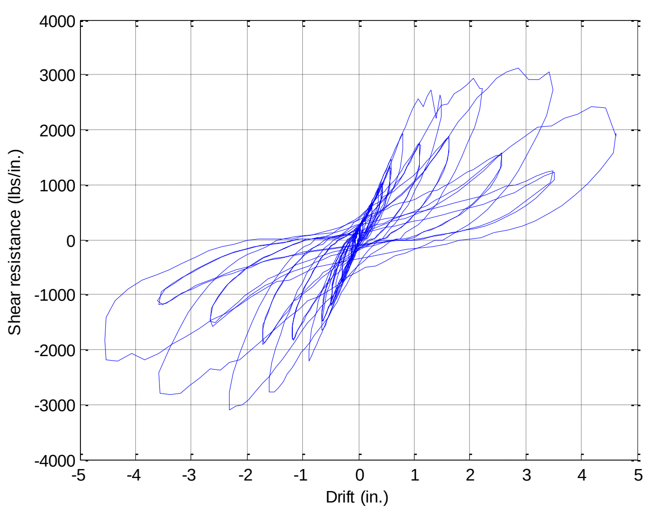


Figure 38. Hysteresis curve of test No. 17 of Type 11wall configuration

For comparison, Type 12, including one plywood and OSB sheathing shear walls were tested. Specimen of test No.20 failed in shear resulting in transverse fracture of sheathing horizontally. With increased displacement, bearing of the panel at the vertical edge of the panel fractured panel edge. Specimen of test No. 21, failed in fasteners along the edge and horizontal joints pulling through of screw head. The wood panel shear wall showed high shear strength, but also exhibit unanticipated low stiffness and ductility.

Figure 39. Failure mode of plywood wall of configuration Type 12

Figure 40. Failure mode of OSB of configuration Type 12

A limitation on the maximum inelastic lateral displacement of a shear wall may affect the determination of shear wall nominal strength. ASCE7-05 requires the inelastic drift for structures use shear wall lateral system less than 2.5% of wall height, i.e. $\Delta_{2.5\%}$ = 2.4-in for 8 ft. high shear wall. Figure 41 shows how to determine the nominal strength of the tested shear wall in terms of magnitude of peak displacement Δ_u and the inter-storey drift $\Delta_{2.5\%}$. In Figure (a), if the inelastic drift $\Delta_{2.5\%}$ (2.4-in) is greater than the shear wall peak displacement Δ_u, the nominal shear strength of the wall is just valued in the peak load F_{peak}. While in Figure (b), when the inelastic drift $\Delta_{2.5\%}$ falls between the interval 0 and Δ_u, the nominal strength of the wall should value in the load on the curve responding to the displacement of $\Delta_{2.5\%}$. And the modified summary of shear wall tested results was given in Table 10.

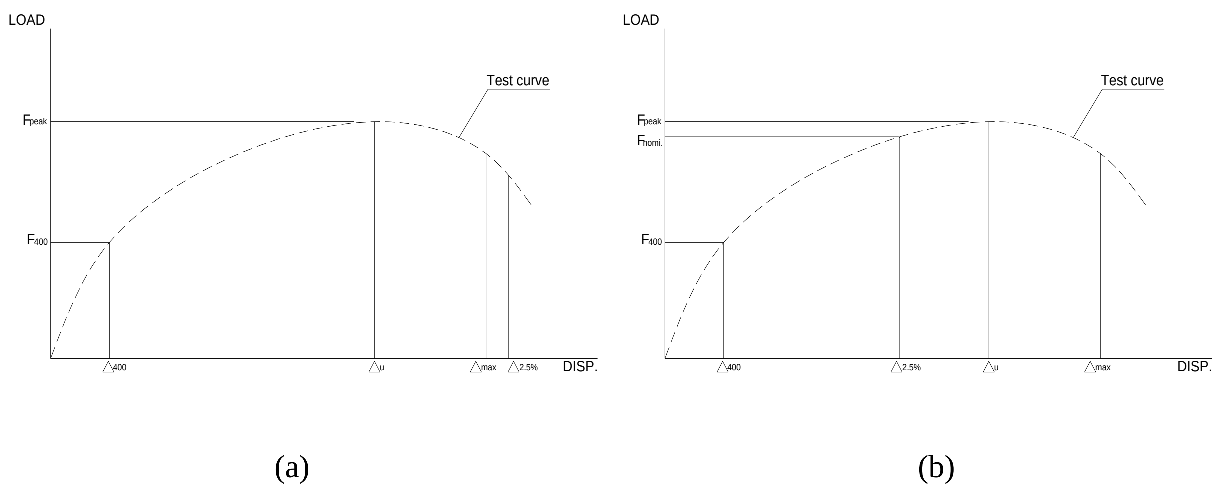


Figure 41. Shear wall load-displacement relationship curve

Table 10. Shear wall modified nominal strength

Wall configuration	test number	nominal strength Pn (lbf)	deflection (in)	average stiffness (lb/in.)
Type 1	No.2_no holes_monotonic	4152.5	2.326	9195
	No.12_no holes_monotonic	4640.0	2.400	10879
	No.7_no holes_cyclic	4146.3	2.400	10430
	No.19_no holes_cyclic	4782.5	2.363	10971
Type 2	No.1 no stitch screws	2107.1	2.400	8601
Type 3	No.3_6x6 holes_monotonic	2797.5	2.400	5399
	No.4_6x6 holes_cyclic	2930.0	2.333	6333
	No.8_6x6 holes_cyclic	2628.0	2.357	6892
Type 4	No.5_6x4 holes_cyclic	3732.5	2.283	8489
Type 5	No.6_6x6 slits_cyclic	2752.5	1.870	8045
Type 6	No.9_24x3 holes_cyclic	2633.8	2.360	5678
Type 7	No.10_24x3 slits_cyclic	2697.5	2.278	8568
	No.13_24x3 slits_cyclic	2890.0	2.345	8310
Type 8	No.11_24x3 horiz. slits_cyclic	4156.3	1.966	11132
Type 9	No.14_12x2 slits_cyclic	3568.8	1.861	11392
Type 10	No.16_24x1 slits_cyclic	4616.3	2.189	11129
Type 11	No.18_24x2 slits_monotonic	2965.0	2.400	8480
	No.15_24x2 slits_cyclic	3018.8	2.189	11126
	No.17_24x2 slits_cyclic	3013.0	2.189	9987
Type 12	No.20_plywood_cyclic	2915.0	2.278	6939
	No.21_OSB_cyclic	3166.3	2.267	8318

CHAPTER 6

SEISMIC RESPONSE MODIFICATION FACTOR FOR COLD- FORMED STEEL FRAME / CORRUGATED SHEET STEEL

6.1 Introduction

Seismic design codes in the United States were initiated in the late 1920's with some relatively simple equivalent static formulas. The development of earthquake code provisions proceeded somewhat intermittently until the Structure Engineers Association of California (SEAOC) in 1959-60 published its *Recommended Lateral Force Requirements and Commentary*, which was applicable to California seismic conditions. The SEAOC provisions recognized that the seismic forces induced in a structure are related to the structure's mode of deformation and fundamental period. Seismic codes in the United States and in many other countries have since been patterned after the SEAOC provisions (N.M. New mark and W.J. Hall 2007).

6.2 The Evolvement of Seismic Provisions in Corresponding Code.

The Seismic design evolution has developed a relationship of load versus strength and serviceability which was similar to other building loads addressed through the use of equivalent lateral load procedure. In 1959 published SEAOC Bluebook, the equation presented for building base shear V was:

$$V=KCW \tag{1}$$

In which W was the total dead load, C was related to the building's natural period, and K was a "horizontal force factor" related to the building system type:

Table 11. Horizontal Force Factor

Building Type	K-Horizontal Force Factor
Bearing Wall	1.33
Framing Systems not Classified	1.00
Dual Systems	0.80
Moment Resisting Frames	0.67

In 1985 Uniform Building Code (UBC), the equation expressed in:

$$V=ZIKCSW \tag{2}$$

In 1988 Uniform Building Code (UBC), the equation given as the followed form:

$$V=ZICW/R_w \tag{3}$$

In equations (2) & (3), the values related to the parameters Z, I, C, S was in a slight different (Table 12). And from which, the relationship between K and R was derived as:

$$K=8/R_w \tag{4}$$

The seismic force values were calculated by equation (3) by dividing forces that would be associated with elastic response by a response modification factor, often used as the symble “R”. The concept of a response modification factor was initiated on the basis of the premise that a well-detailed seismic resistant framing system could bear large inelastic deformation without collapse (ductile behavior) and could develop lateral strength in excess of their design strength. The R factor was assumed to represent the ratio of minimum loads required at the design level ground motion if the framing system were to work entirely elastically to the prescribed design forces at the significant yield level.

Table 12. Seismic parameters comparison

1985 UBC	1988 UBC
$V = ZIKCSW$	$V = ZICW/ R_w$
Z = 1, ¾, 3/8, 3/16	Z = 0.4, 0.3, 0.2, 0.075
I = 1.0, 1.25, 1.5	I = 1.0, 1.25
$C = 1/ 15 T^{.5}$	$C = 1.25 S/ T^{.67}$
$C < 0.12$	$C < 2.75$
$C \times S < 0.14$	$C/R_w > 0.075$
$0.67 \leq K \leq 1.33$	$4 \leq R_w \leq 12$

In the 2000 version of National Earthquake Hazards Reduction Program (NEHRP), the total lateral force, i.e. the base shear created by earthquake on a building was given by the formula:

$$V=IS_{DS}W/R \tag{5}$$

$$V=TIS_{D1}W/R \tag{6}$$

In which, W is the total dead load, I is the importance factor, S_{DS} & S_{D1} are the design spectral response acceleration at short periods and at 1 second respectively, T is structure period within the constant response velocity portion of the design spectrum. Equations (5) & (6), which remained the format of the response modification factor R to the denominator, are adopted by the current code ASCE-05 and IBC-06.

6.3 Ductility.

Most structures are not expected, or even designed, to remain elastic under violent ground motions. Rather, structures are expected to enter the inelastic region-the extent to which they behave inelastically can be defined by the ductility factor. Using the concept of equivalent energy elastic plastic model (EEEP) to calculate ductility was first proposed by Park (1989) and has been adopted by Kawai et al. (1997) in ASCE Structures Congress to analyze light gauge steel framed walls. The EEEP model results in an idealized bilinear shaped load deflection curve which can provide a realistic depiction of the experimental data in terms of energy dissipation levels. The EEEP model is based on the notation that the energy dissipated by the wall specimen during a monotonic or cyclic test is equivalent to the energy represented by the bilinear curve. The curve represents an ideal perfectly elastic-perfectly plastic shear wall system that is capable of dissipating an equivalent amount of energy as compared with the real shear wall. EEEP curve of each specimen under monotonic test was constructed based on the equivalent energy approach, as illustrated in Figure 42. The first step was to determine the displacement, Δ_{400}, and the matching resistance F_{400}. The Δ_{400} equaled the shear wall height H divided by 400, H/400, an estimation of the maximum service displacement level. Δ_{400} and F_{400} were used to define the stiffness of the elastic portion, K_e, of the bilinear EEEP curve.

$$K_e=F_{400}/\Delta_{400} \tag{7}$$

A horizontal line depicting the plastic portion of the EEEP curve was then positioned. The areas located above and below the EEEP curve, which enclosed by the EEEP curve and the real test curve were equal. The resulting plastic portion was then defined as the nominal shear strength, F_y. The failure load is the point on the envelope curve corresponding to the last data point with the absolute load equal or greater than $|0.8*F_{peak}|$ as illustrated in Figure 42. Δ_{max}, the maximum displacement, which corresponds to the failure load, was then defined accordingly. And the maximum elastic displacement, Δ_y, was defined by the intersection point of the EEEP curve elastic and plastic portion. As a result, the ductility, μ, could be obtained as the ratio of maximum displacement to the maximum elastic displacement.

$$\mu=\Delta_{max}/\Delta_y \tag{8}$$

In the case of cyclic test, a backbone curve was first constructed by connecting the peak point of each excursion using linear lines. Then the same EEEP procedure for monotonic test data was used to produce the EEEP bilinear curve and calculate the ductility for cyclic test result. The EEEP bilinear curves for representative cyclic tests are depicted in Figure 43.

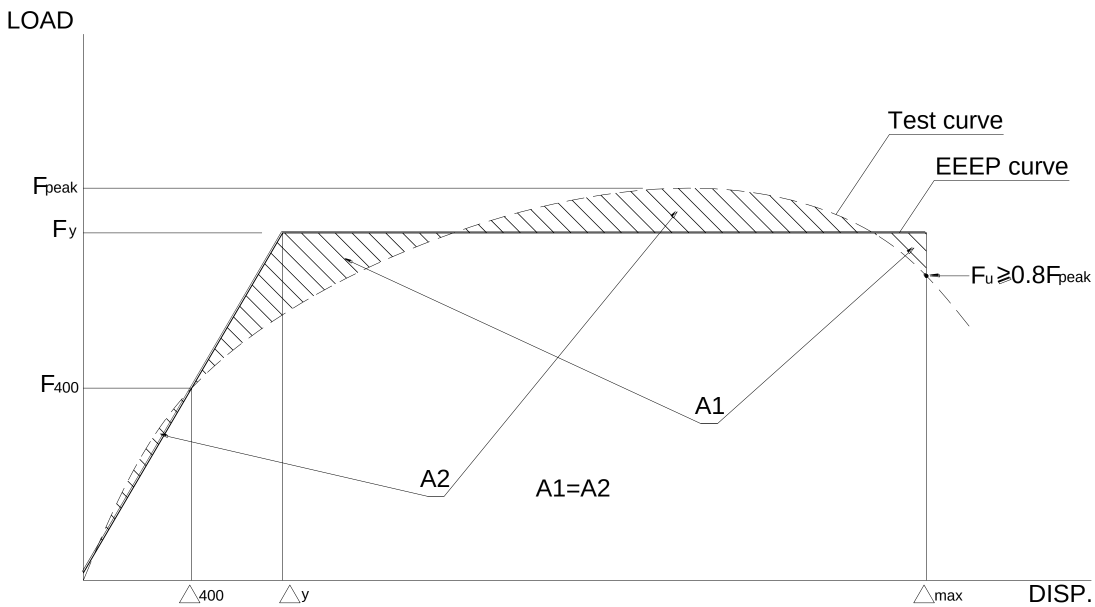


Figure 42. EEEP curve for monotonic test

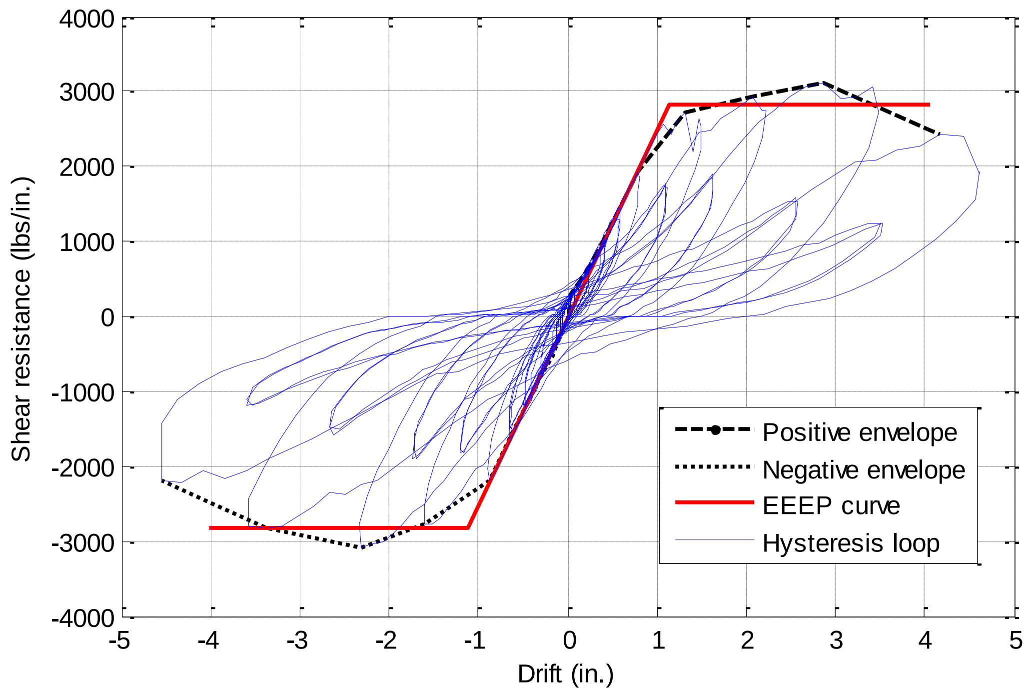


Figure 43. EEEP bilinear curves for cyclic test

6.4 Evaluation of Response Modification Factor.

A large portion of discussion of the ductility factor μ has been given above, the question is how does the ductility factor μ affect the behavior of framing system in a building, after all, it was not expressed as explicitly as the response modification factor, R, included in base shear equations for seismic action.

Nemark and Hall (1982) derived the relationship between the ductility factor μ and the response modification factor R, according to which elastic response spectra can be readily modified to reflect inelastic behavior (Figure 44 & 45).

$$R = \mu \qquad Hz < 2 \text{ i.e. } T > 0.5 \text{ sec} \tag{9}$$

$$R = \sqrt{2\mu - 1} \qquad 2 \leq Hz \leq 8 \text{ i.e. } 0.125 \leq T \leq 0.5 \text{ sec} \tag{10}$$

$$R = 1 \qquad Hz > 33 \text{ i.e. } T < 0.03 \text{ sec} \tag{11}$$

Expression of Equation (11) permits no strength reduction in highly stiff systems which possess limited ductility capacity. In Equation (9), it shows that the system behaves very flexible, and the maximum relative displacement tends toward the maximum ground displacement. For any given ground acceleration time history the inelastic strength is attained from the elastic strength demand divided by the displacement ductility ratio. Framing behavior indicated by Equation (10) will fall between the two extremities stated by Equation (9) and (11).

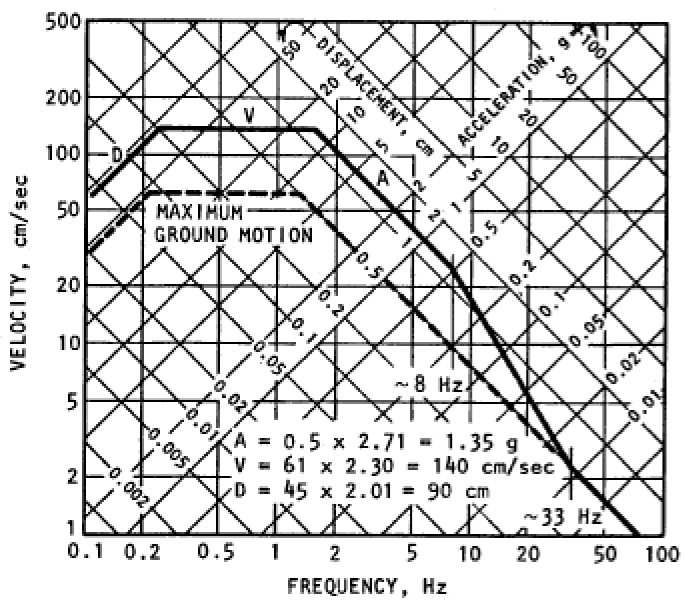


Figure 44. Idealized elastic design spectrum, horizontal motion (Nemark and Hall)

(ZPA=0.5g, 5% damping)

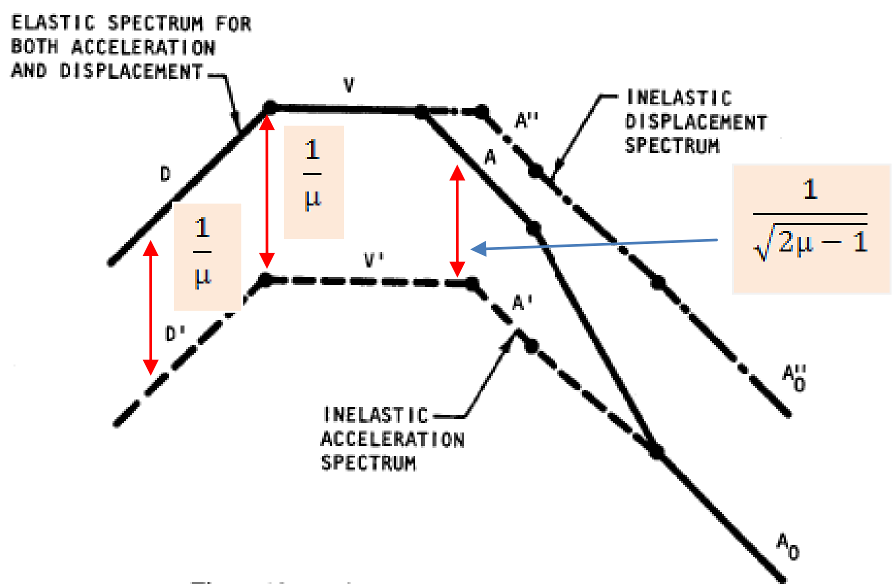


Figure 45. Combined elastic and inelastic response spectra (Nemark and Hall)

Besides *Newmark and Hall*, other researchers like *Krawinkler and Nassar*, *Miranda and Bertero*, *Vidic et al.* also derived their formulas which established the relationship between modification factor R and ductility factor μ, but all in all, the response modification factor depends not only on the characteristics of the system, but also on the ground acceleration time history. For a given ground motion, R is a function of the period of vibration T of the structure, the damping, the type of hysteretic behavior and the level of displacement ductility ratio. For a given acceleration time history the response modification factor is primarily influenced by the period of vibration and the level of inelastic deformation, and to a much lesser degree by the damping and hysteretic behavior of the system (Miranda 1994).

6.5 Determination of the Response Modification Factor for Shear Wall Sheathed with Corrugated Sheet Steel.

It was agreed and verified by researcher that response modification factor R is the function of building natural period T and ductility ratio μ. The equations proposed by *Krawinkler and Nassar*, *Miranda and Bertero*, *Vidic et al.* were quite complicated due to double variants of T and μ which were combined together to express the function. For a quick evaluation of the behavior and conducting a practical design of the shear walls, the simplified expressions proposed by Nemark and Hall (1982) could be employed in this research. So the problem will be focusing on knowing what will be the physical periods the building experienced under the earthquake. Boudreault (2005) given a summaries about the natural periods which based on the past studies and calculation estimates for cold-formed steel building (Table 13).

Table 13. Statistic of Natural Period for cold-formed steel buildings

Building Type	Reference	Natural Period T (sec)
One, one and a half, and two-storey North American residential house	Soltis et al. (1981)	0.06 to 0.25
Two and three-storey North American residential house	Sugiyama (1984)	0.14 to 0.32
Residential House (1999a)	Gad et al.	0.25
Low rise wood frame structure	Foliente and Zacher (1994)	0.05 to 0.1
Residential houses (Univ. of BC code estimate)	Folz and Filiatrault (2001a)	0.18
Typical 8'x4' shear wall (NBCC 1995 estimate)	Zhao (2002)	0.20

From above reference information and considering the properties of the corrugated metal sheathing and the potential application of shear wall in multi-storey commercial building, the natural period would converge between the interval (0.1s, 0.5s). As a result, Equation (10) will be applied to ductility factor that was acquired with EEEP technique to calculate R, the response modification factor. Table 14 shows the ductility factors and response modification factor for the 21 tests performed in this research.

Table 14. Response modification factor R and ductility factor μ

wall configuration	Opening configuration	Test protocal	Test Number	Nominal sheathing thickness	Fastener spacing	Ductility μ	R
Type 1	no opening	Monotonic-ASTM E564	No.2	27 mil	2.5"/5"	1.511	1.422
		Monotonic-ASTM E564	No.12	27 mil	2.5"/5"	2.051	1.761
		Cyclic-CUREE	No.7	27 mil	2.5"/5"	1.644	1.513
		Cyclic-CUREE	No.19	27 mil	2.5"/5"	2.123	1.802
Type 2	no seam screws	Cyclic-CUREE	No.1	27 mil	2.5"/5"	4.277	2.748
Type 3	6x6" circular holes	Monotonic-ASTM E564	No.3	27 mil	2.5"/5"	1.678	1.535
		Cyclic-CUREE	No.4	27 mil	2.5"/5"	1.679	1.535
		Cyclic-CUREE	No.8	27 mil	2.5"/5"	2.415	1.957
Type 4	6x4" circular holes	Cyclic-CUREE	No.5	27 mil	2.5"/5"	2.039	1.754
Type 5	6x6" vertical slit	Cyclic-CUREE	No.6	27 mil	2.5"/5"	2.204	1.846
Type 6	24x3" circular holes	Cyclic-CUREE	No.9	27 mil	2.5"/5"	2.485	1.992
Type 7	24x3" vertical slit	Cyclic-CUREE	No.10	27 mil	2.5"/5"	3.699	2.530
		Cyclic-CUREE	No.13	27 mil	2.5"/5"	3.365	2.394
Type 8	24x3" horizontal slit	Cyclic-CUREE	No.11	27 mil	2.5"/5"	1.534	1.438
Type 9	12x2" vertical slit	Cyclic-CUREE	No.14	27 mil	2.5"/5"	2.128	1.804
Type 10	24x1" vertical slit	Cyclic-CUREE	No.16	27 mil	2.5"/5"	1.295	1.261
Type 11	24x2" vertical slit	Monotonic-ASTM E564	No.18	27 mil	2.5"/5"	3.090	2.276
		Cyclic-CUREE	No.15	27 mil	2.5"/5"	3.646	2.508
		Cyclic-CUREE	No.17	27 mil	2.5"/5"	3.027	2.248
Type 12	plywood	Cyclic-CUREE	No.20	15/32 in.	2.5"/5"	1.964	1.711
	OSB	Cyclic-CUREE	No.21	7/16 in.	2.5"/5"	2.488	1.994

As shown in Table 14, shear walls with opening configuration of 24x3 in. vertical slits, 24x2 in. vertical slits, and no seam screws were observed to provide relatively higher values of the ductility factors which are greater than 3.0 and response modification factors with an average value of 2.45.

CHAPTER 7

CONCLUSION AND RECOMMENDATION

Corrugated sheet steel CFS shear walls in various opening configurations were experimentally examined and numerically analyzed for two goals: (1) to investigate the shear capacity of the shear wall under racking load caused by earthquake and wind, and (2) study the behavior of shear walls with different opening pattern and explore the ductility factor μ and derive response modification factor R of corrugated metal sheet shear wall.

North American Standard for Cold-Formed Steel Framing – Lateral Design 2007 (AISI S213-07) provided the nominal strength (Table 2) for shear walls under in-plane load, meanwhile requires “the nominal shear strength for light-framed wall systems for buildings, where the seismic response modification coefficient, R, used to determine the lateral forces is taken greater than 3. Considering the combined factors of nominal strength, stiffness and ductility of the tested specimens in this research, shear wall without opening and wall with 24x2-in vertical slits were interested based on their performance. Compared to the nominal strength under in-plane loads for shear wall from Standard AISI S213 showed in Table 2, shear wall without opening exhibited strength capacity 4782.5 plf which is 55.3% higher than the published value 3080 plf of 7/16 in. OSB shear wall. Shear wall with 24x2-in vertical slits demonstrated same nominal strength as the published shear capacity. However the response modification factor R of no-opening shear wall is only 1.802 and 2.508 for 24x2-in vertical slits wall, which are both less than the required value 3.0.

The corrugated no-opening shear wall could be used as the lateral system in building constructions even if it showed low ductility character. Based on the equivalent energy and the using of EEEP curve, shear wall yielding strength could be calculated as higher as 4030.5 plf which could be used as the design strength of shear wall working elastically under earthquake.

Shear wall with 24x2-in vertical slits presented higher lateral force resistance capacity, shear wall stiffness and exhibited the behavior of ductility to some extent but lower than the stipulated value in Standard AISI S213-07.

Based on the experimental research, the following further studies are recommended for the shear walls with 24x2-in vertical slits and other opening configurations

- This research verified that less quantity number, big size cutting reduced much of the wall stiffness. While more quantity of small size cutting make the wall behave well on stiffness, nominal strength and ductility. Therefore further study could be focus on shear wall with 24x2-in vertical slits, 24x3-in vertical slits and 24x2-in circular opening.
- Undoubtedly, Finite Element Model could capture the structural behavior of cold-formed steel shear wall. FEA should be emphasized to predict the performance of shear wall with different opening configuration. Meanwhile, connection test of screw seam-connection, panel to frame connection and frame to frame connection should be conducted to provide accurate connection data for FE simulation.
- To probe a suitable relationship between ductility factor μ and response modification factor R and building natural period T, therefore a practical response modification factor R could be derived based on the behavior of tested specimens. And conclude a correct response modification factor R for corrugated shear wall with certain cutting pattern on the sheathing.
- The circular holes of the sheathing in this research were cut manually with plasma cutter. This work depended on the operator’s individual skill. But no matter how skilled of the worker, the quality of the holes perimeter could not as good as machine cutting edge,

therefore stress concentration was caused and this imperfection made the premature failure of the sheathing. In the future construction of the shear wall with holes, pre-punched holes in the manufacturer’s mill should be applied.

- Develop a preliminary design and detailing manual. Upon the analysis and test results, the design manual should be applicable for shear walls with different corrugated sheathing.

A series of 21 full-scale tests were carried out to evaluate the capacity of cold-formed steel framed shear wall with corrugated sheet steel having different cutting openings. The derived test data provided a basic optimized study of the corrugation opening configuration. In the future study, more tests need be performed to verify the conclusion of this research and much data should be acquired to validate the ductility factor and response modification factor.

APPENDIX A

DATA SHEET S FOR CORRUGATED SHEET SHEAR WALL TESTS

Opening Type 1: No cutting-opening shear wall. **Test No. 02**

Test date: Dec. 18, 2012

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft. Studs: 350S162-68, 33ksi Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Monotonic-ASTM

Test results:

+Peak load: 4152.5plf.

Lateral displacement of wall top at +peak load: 2.326 in.

-Peak load: NA

Lateral displacement of wall top at -peak load: NA

Average peak load: 4152.5plf.

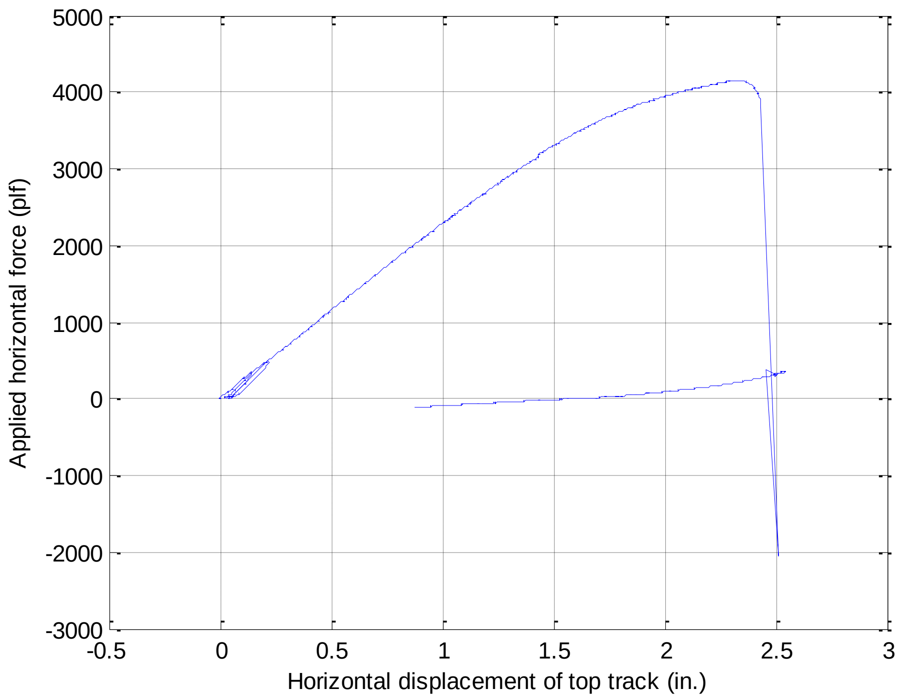
Average lateral displacement of wall top: 2.326 in.

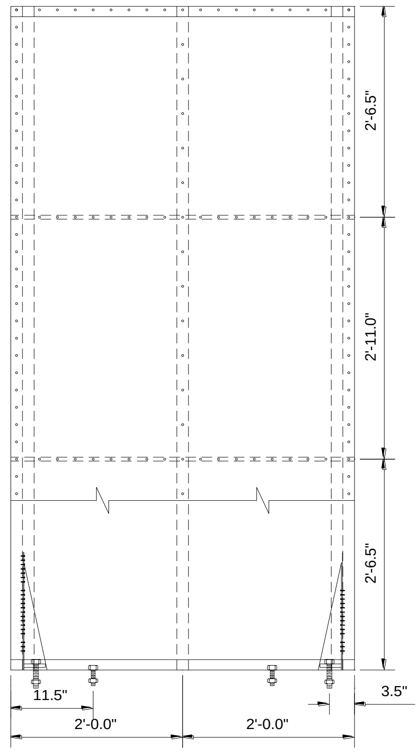
Observed Failure Mode: specimen failed in the broken at hold down bolt.

Screw Pull Out: None

Sheathing Tear: None

Screw Pull Over: None




2'-6.5"
2'-11.0"
2'-6.5"
11.5"
3.5"
2'-0.0"
2'-0.0"

Opening Type 1: No cutting-opening shear wall. **Test No. 12**

Test date: Jan. 15, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft. Studs: 350S162-68, 33ksi Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Monotonic-ASTM

Test results:

+Peak load: 5007.5plf.

Lateral displacement of wall top at +peak load: 3.032 in.

-Peak load: NA

Lateral displacement of wall top at -peak load: NA

Average peak load: 5007.5plf.

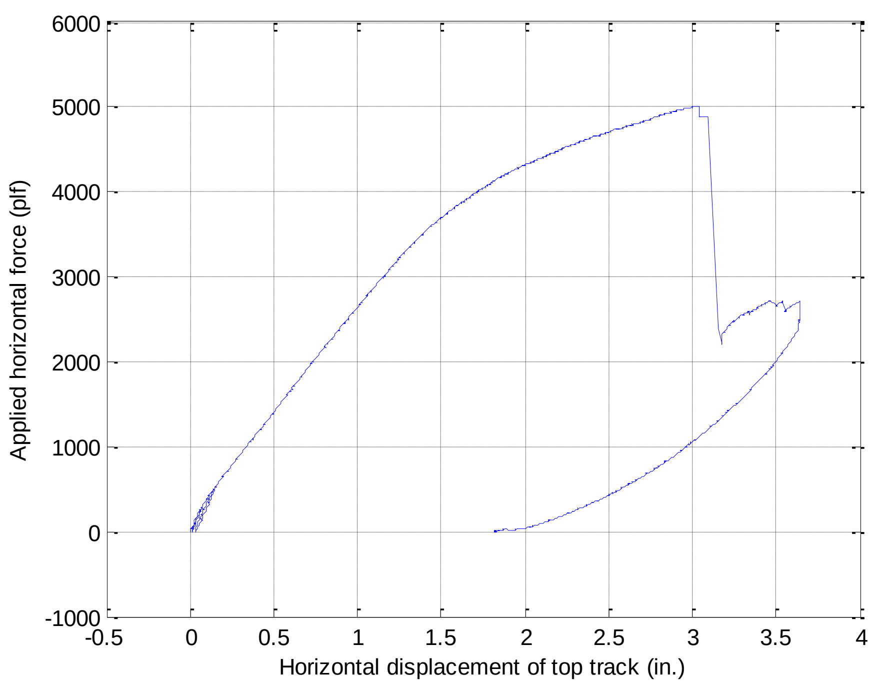
Average lateral displacement of wall top: 3.032 in.

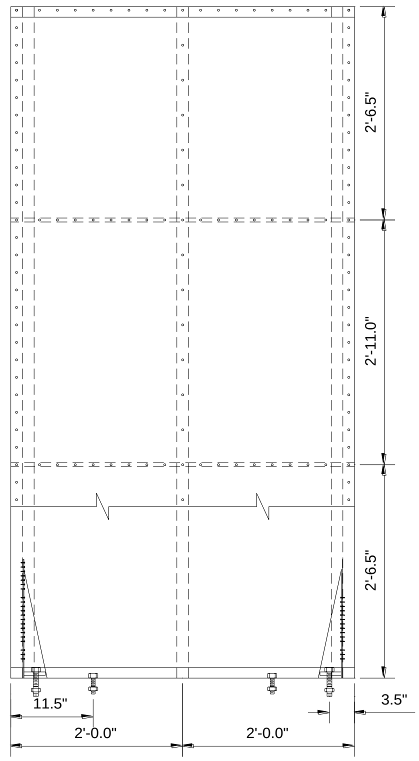
Observed Failure Mode: the sheathing buckled, screws in the middle studs were pulled over.

Screw Pull Out: None

Sheathing Tear: None

Screw Pull Over: Yes




2'-6.5"
2'-11.0"
2'-6.5"
11.5"
3.5"
2'-0.0"
2'-0.0"


2013 01 16

Opening Type 1: No cutting-opening shear wall. **Test No. 07**

Test date: Dec. 27, 2012

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft. Studs: 350S162-68, 33ksi Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 4265plf.

Lateral displacement of wall top at +peak load: 2.49 in.

-Peak load: 4312.5plf

Lateral displacement of wall top at -peak load: 2.78 in.

Average peak load: 4288.8plf.

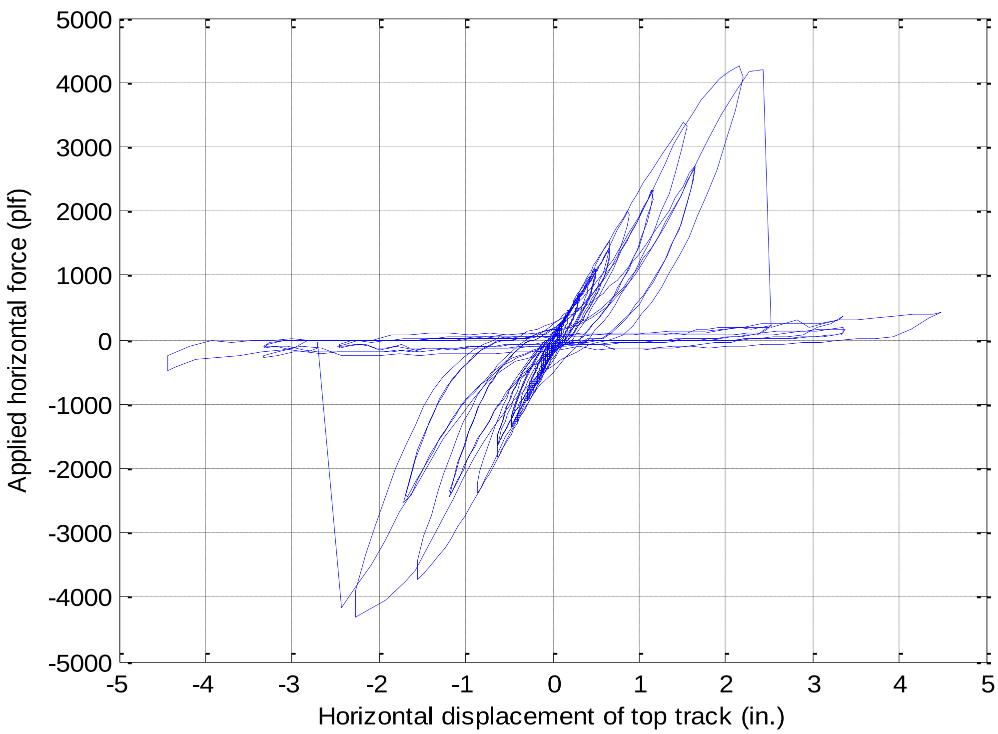
Average lateral displacement of wall top: 2.635 in.

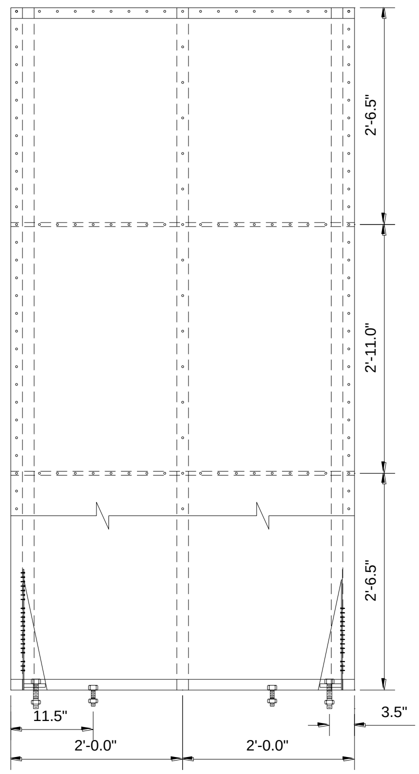
Observed Failure Mode: specimen failed in the broken at hold down bolts.

Screw Pull Out: None

Sheathing Tear: None

Screw Pull Over: None





2'-6.5"
2'-11.0"
2'-6.5"
11.5"
3.5"
2'-0.0"
2'-0.0"

2012 12 2

Opening Type 1: No cutting-opening shear wall. **Test No. 19**

Test date: Jan. 24, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft. Studs: 350S162-68, 33ksi Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 5257.5plf.

Lateral displacement of wall top at +peak load: 2.8 in.

-Peak load: 4807.5plf

Lateral displacement of wall top at -peak load: 2.236 in.

Average peak load: 5032.5plf.

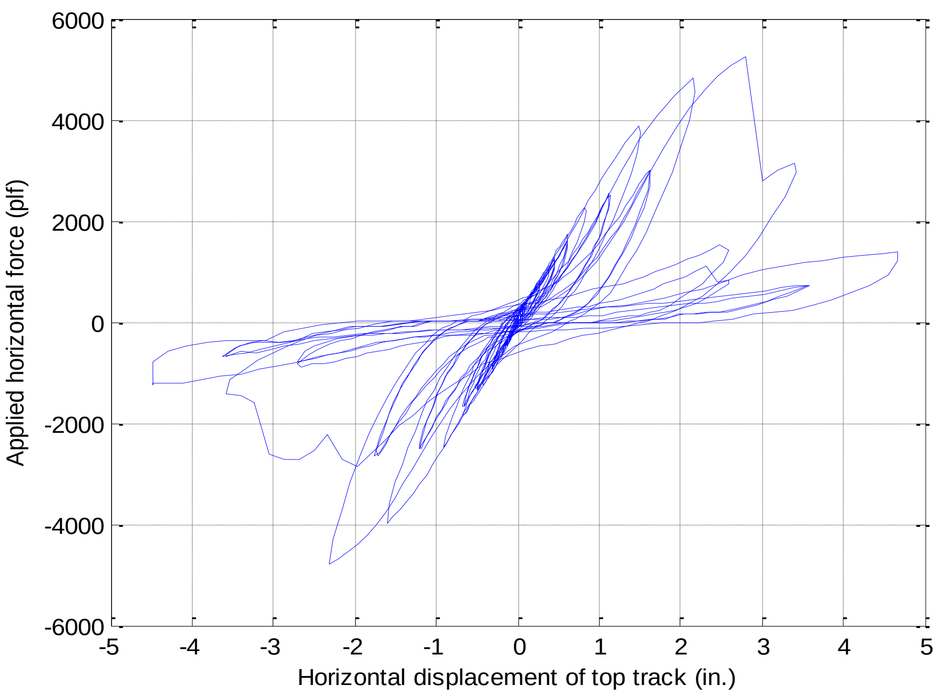
Average lateral displacement of wall top: 2.563 in.

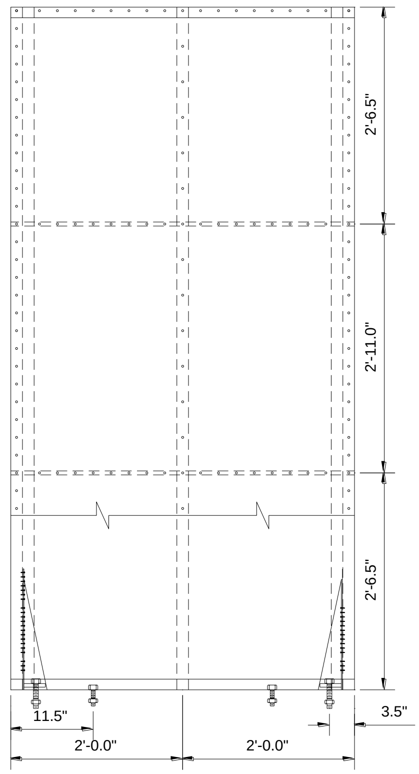
Observed Failure Mode: the sheathing buckled, screws in the middle studs were pulled over.

Screw Pull Out: Yes

Sheathing Tear: None

Screw Pull Over: Yes




2'-6.5"
2'-11.0"
2'-6.5"
11.5"
3.5"
2'-0.0"
2'-0.0"


2013 01 25

Opening Type 2: No seam stitch-screws shear wall. **Test No. 1**

Test date: May 03, 2012

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft. Studs: 350S162-68, 33ksi Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 2276.3plf.

Lateral displacement of wall top at +peak load: 2.585 in.

-Peak load: 2102.3plf.

Lateral displacement of wall top at -peak load: 2.598 in.

Average peak load: 2189.3plf.

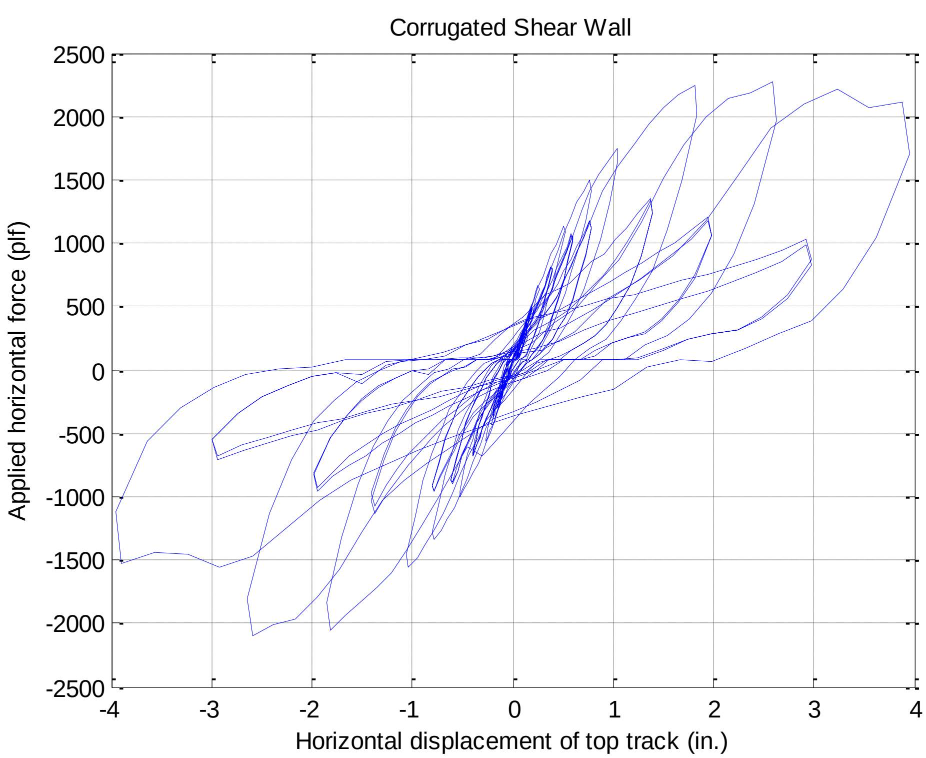
Average lateral displacement of wall top: 2.592 in.

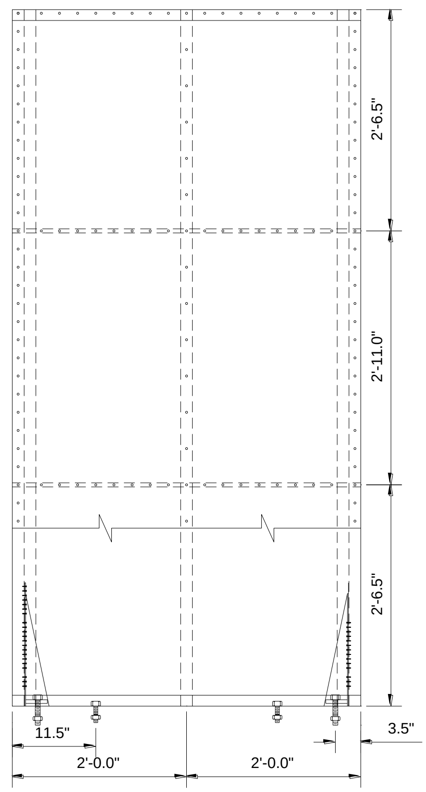
Observed Failure Mode: The upper sheet steel was gouged slot hole due to the reverse cyclic movement, Screws in the boundary studs were pulled over.

Screw Pull Out: None

Sheathing Tear: Yes

Screw Pull Over: Yes




2'-6.5"
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2'-6.5"
11.5"
3.5"
2'-0.0"
2'-0.0"

Opening Type 3: 6x6-in circular holes.   **Test No. 3**

Test date: Dec. 19, 2012

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft.   Studs: 350S162-68, 33ksi   Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4” hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Monotonic-ASTM

Test results:

+Peak load: 3222.5plf.

Lateral displacement of wall top at +peak load: 3.097 in.

-Peak load: NA

Lateral displacement of wall top at -peak load: NA

Average peak load: 3222.5plf.

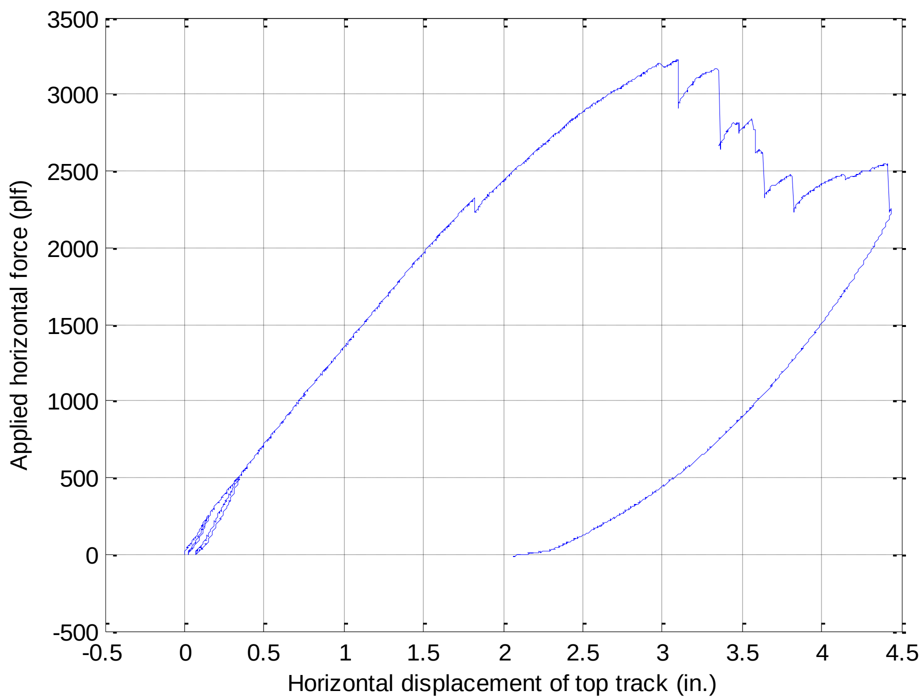
Average lateral displacement of wall top: 3.097 in.

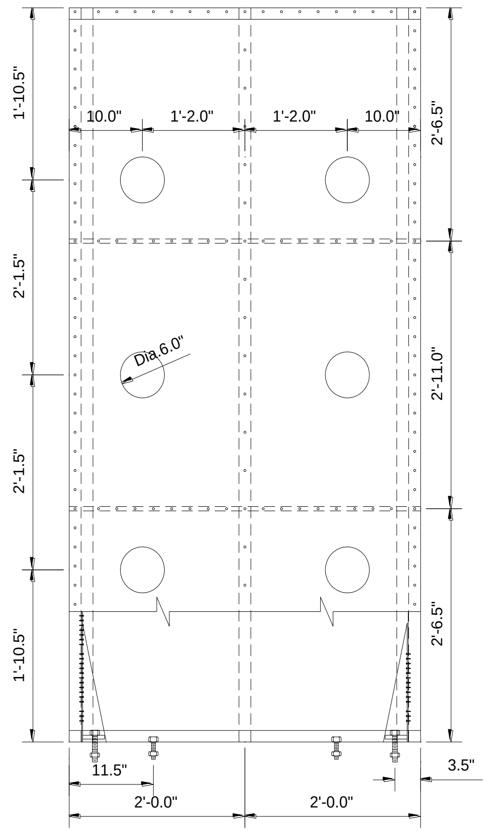
Observed Failure Mode: large relative out-of-plane movement of the sheet elements at the both sides of circles, resulting in the repture of sheathing.

Screw Pull Out: None

Sheathing Tear: Yes

Screw Pull Over: None





1'-10.5"
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1'-2.0"
10.0"
2'-6.5"
2'-11.0"
2'-6.5"
Dia 6.0"
11.5"
3.5"
2'-0.0"
2'-0.0"

2012 12 20

Opening Type 3: 6x6-in circular holes.   **Test No. 4**

Test date: Dec. 19, 2012

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft.   Studs: 350S162-68, 33ksi   Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 3295.0plf.

Lateral displacement of wall top at +peak load: 2.82 in.

-Peak load: 3002.5plf.

Lateral displacement of wall top at -peak load: 2.266 in.

Average peak load: 3148.8plf.

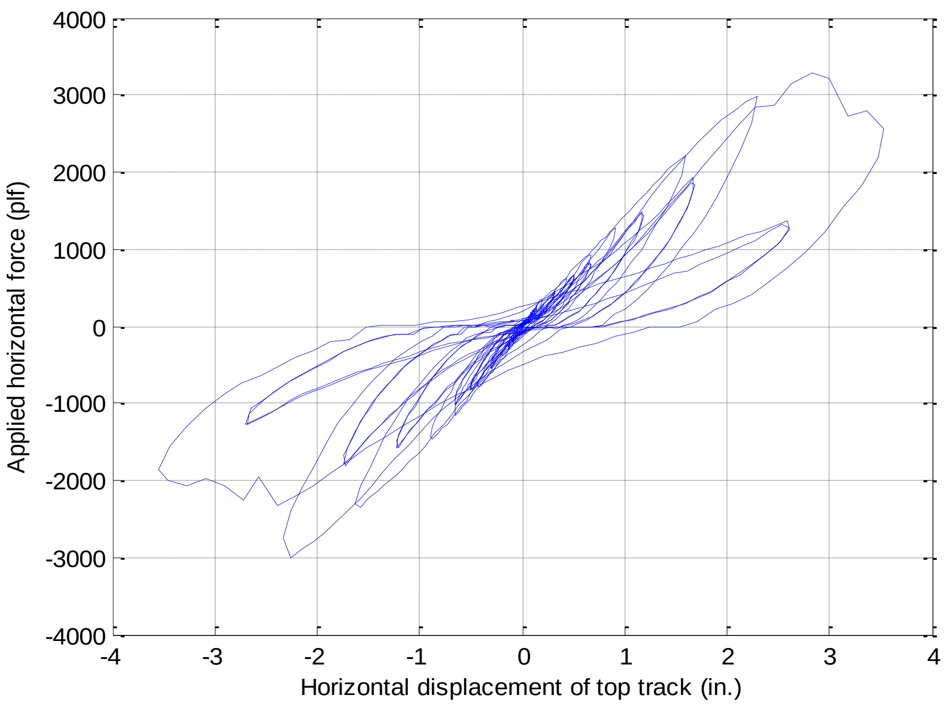
Average lateral displacement of wall top: 2.543 in.

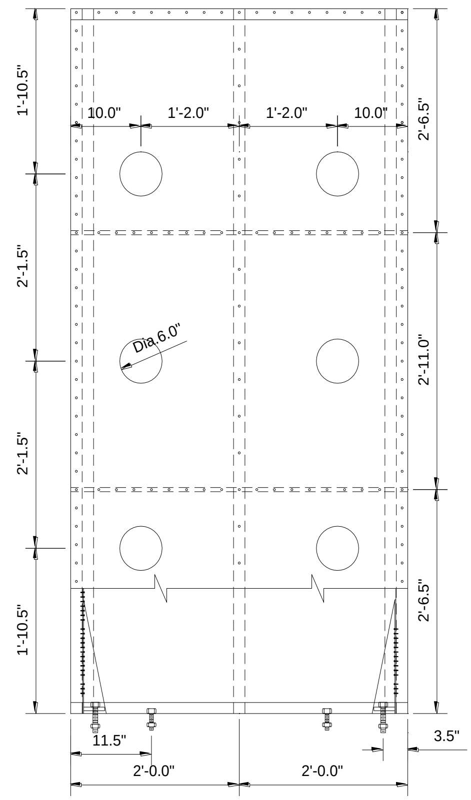
Observed Failure Mode: large relative out-of-plane movement of the sheet elements at the both sides of circles, resulting in the repture of sheathing.

Screw Pull Out: None

Sheathing Tear: Yes

Screw Pull Over: None




1'-10.5"
2'-1.5"
2'-1.5"
1'-10.5"
10.0"
1'-2.0"
1'-2.0"
10.0"
2'-6.5"
2'-11.0"
2'-6.5"
Dia.6.0"
11.5"
3.5"
2'-0.0"
2'-0.0"


2012 12 20


2012

Opening Type 3: 6x6-in circular holes.   **Test No. 8**

Test date: Dec. 28, 2012

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft.   Studs: 350S162-68, 33ksi   Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 3027.5plf.

Lateral displacement of wall top at +peak load: 3.027 in.

-Peak load: 2817.5plf.

Lateral displacement of wall top at -peak load: 2.314 in.

Average peak load: 2922.5plf.

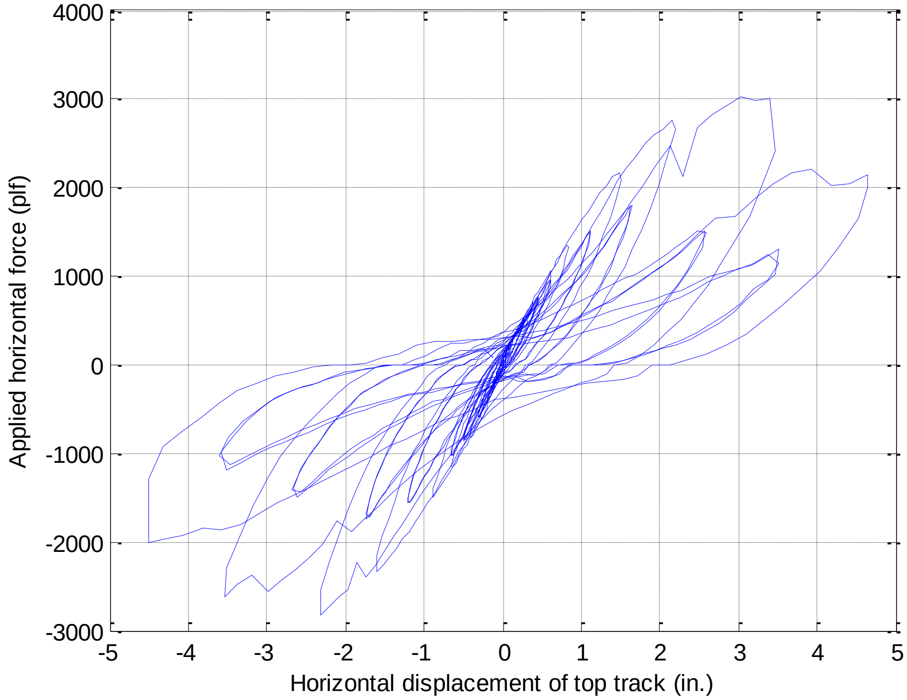
Average lateral displacement of wall top: 2.671 in.

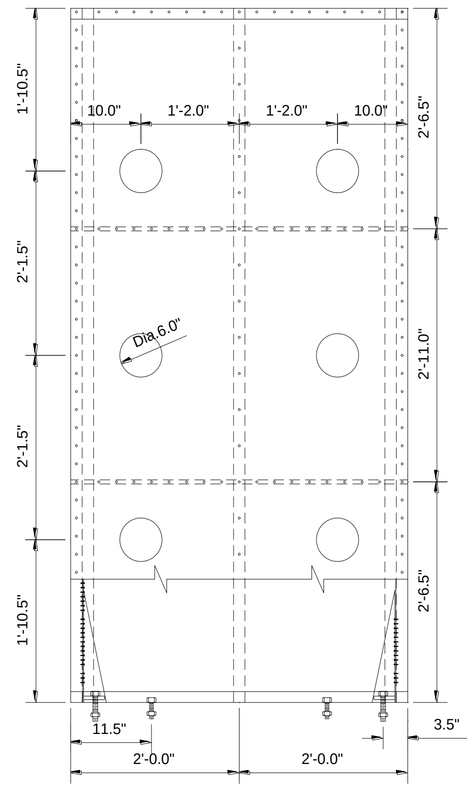
Observed Failure Mode: large relative out-of-plane movement of the sheet elements at the both sides of circles, resulting in the repture of sheathing.

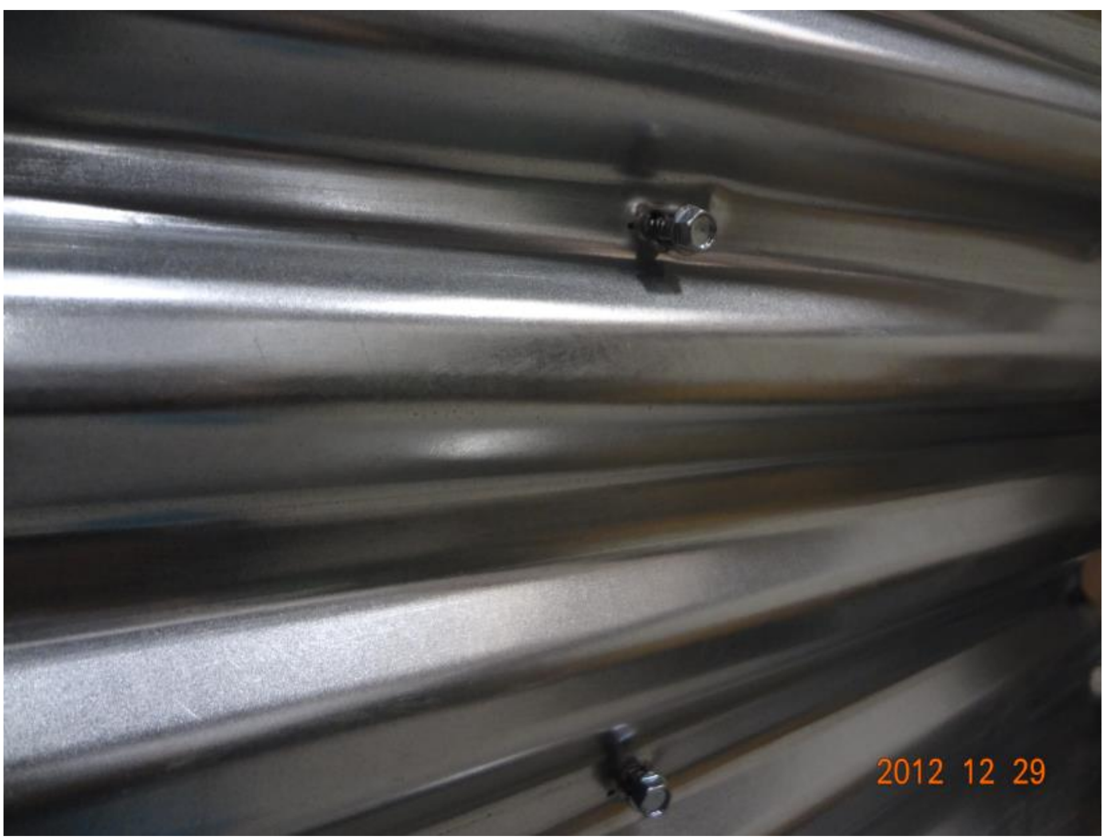
Screw Pull Out: Yes

Sheathing Tear: Yes

Screw Pull Over: None




1'-10.5"
10.0"
1'-2.0"
1'-2.0"
10.0"
2'-6.5"
2'-1.5"
Dia.6.0"
2'-11.0"
2'-1.5"
2'-6.5"
1'-10.5"
11.5"
3.5"
2'-0.0"
2'-0.0"


2012 12 29

Opening Type 4: 6x4-in circular holes. **Test No. 5**

Test date: Dec. 20, 2012

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft. Studs: 350S162-68, 33ksi Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 3882.5plf.

Lateral displacement of wall top at +peak load: 2.865 in.

-Peak load: 3582.5plf.

Lateral displacement of wall top at -peak load: 2.167 in.

Average peak load: 3732.5plf.

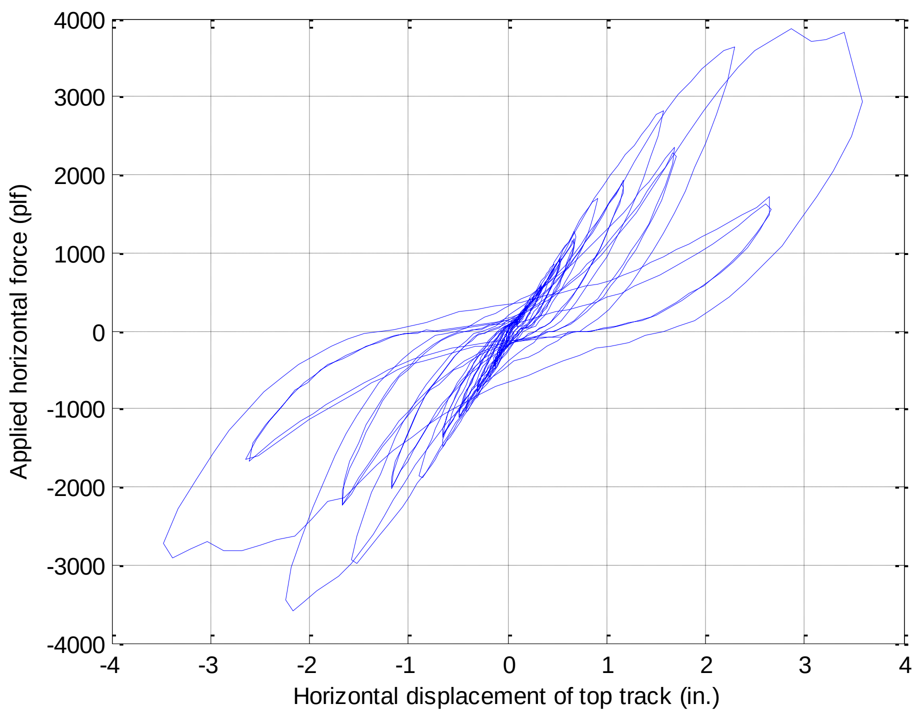
Average lateral displacement of wall top: 2.516 in.

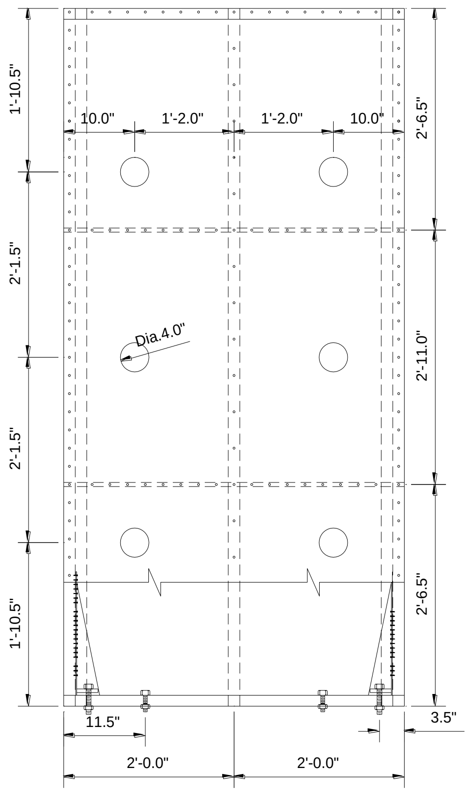
Observed Failure Mode: large relative out-of-plane movement of the sheet elements at the both sides of circles, resulting in the repture of sheathing. some screws were pulled over through head.

Screw Pull Out: None

Sheathing Tear: Yes

Screw Pull Over: Yes





1'-10.5"
10.0"
1'-2.0"
1'-2.0"
10.0"
2'-6.5"
2'-1.5"
Dia.4.0"
2'-11.0"
2'-1.5"
1'-10.5"
2'-6.5"
11.5"
3.5"
2'-0.0"
2'-0.0"

2012 12 21

Opening Type 5: 6x6-in vertical slits.   **Test No. 6**

Test date: Dec. 21, 2012

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft.   Studs: 350S162-68, 33ksi   Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4” hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 2865plf.

Lateral displacement of wall top at +peak load: 2.153 in.

-Peak load: 2640plf.

Lateral displacement of wall top at -peak load: 1.586 in.

Average peak load: 2752.5plf.

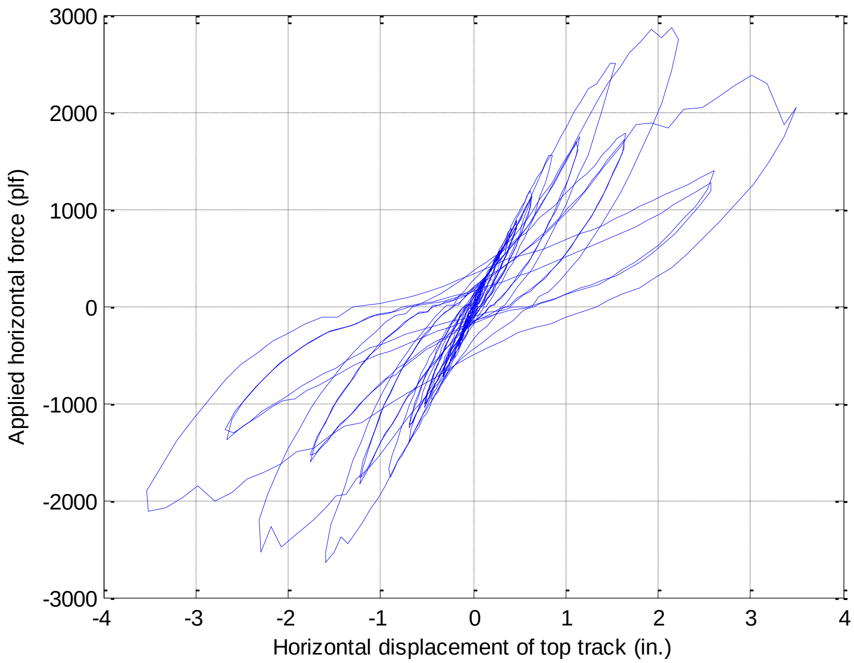
Average lateral displacement of wall top: 1.87 in.

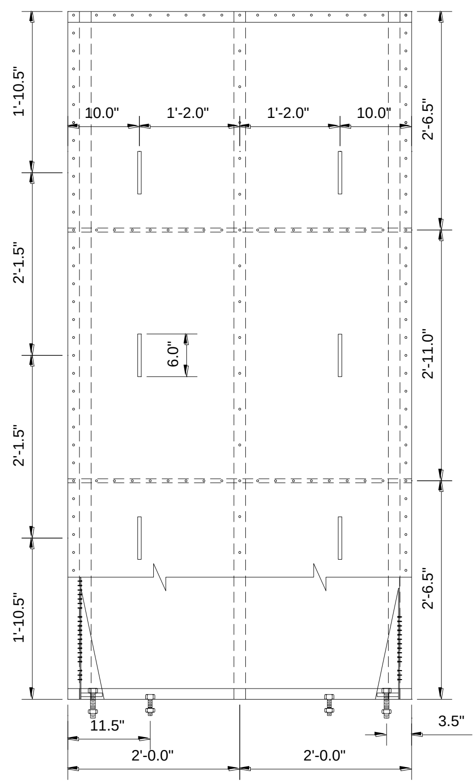
Observed Failure Mode: large relative out-of-plane movement of the sheet elements at the both side of vertical slits, resulting in the repture of sheathing.

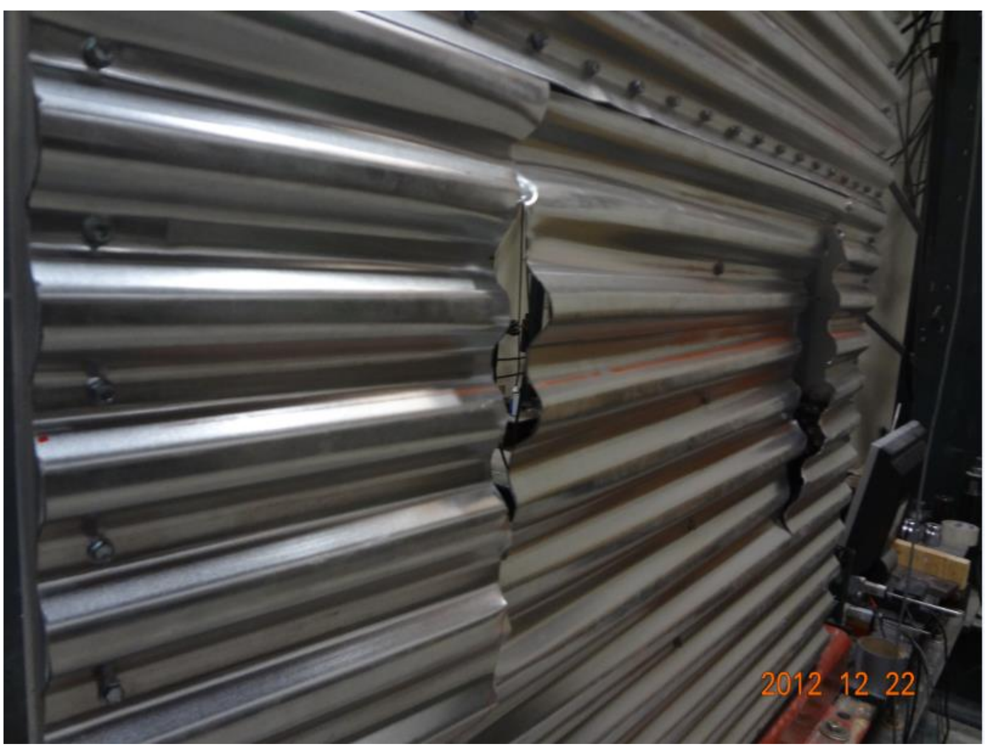
Screw Pull Out: None

Sheathing Tear: Yes

Screw Pull Over: None




1'-10.5"
10.0"
1'-2.0"
1'-2.0"
10.0"
2'-6.5"
2'-1.5"
6.0"
2'-11.0"
2'-1.5"
1'-10.5"
2'-6.5"
11.5"
3.5"
2'-0.0"
2'-0.0"


2012 12 22

Opening Type 6: 24x3-in circular holes.   **Test No. 9**

Test date: Jan. 08, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft.   Studs: 350S162-68, 33ksi   Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 3027.5plf.

Lateral displacement of wall top at +peak load: 3.461 in.

-Peak load: 2850plf.

Lateral displacement of wall top at -peak load: 3.187 in.

Average peak load: 2938.8plf.

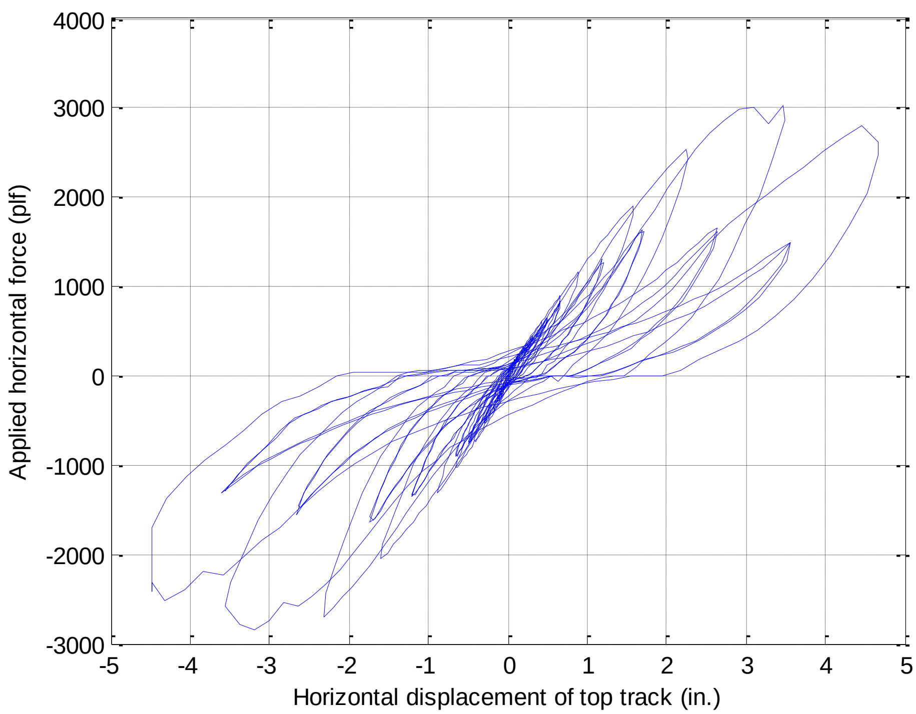
Average lateral displacement of wall top: 3.324 in.

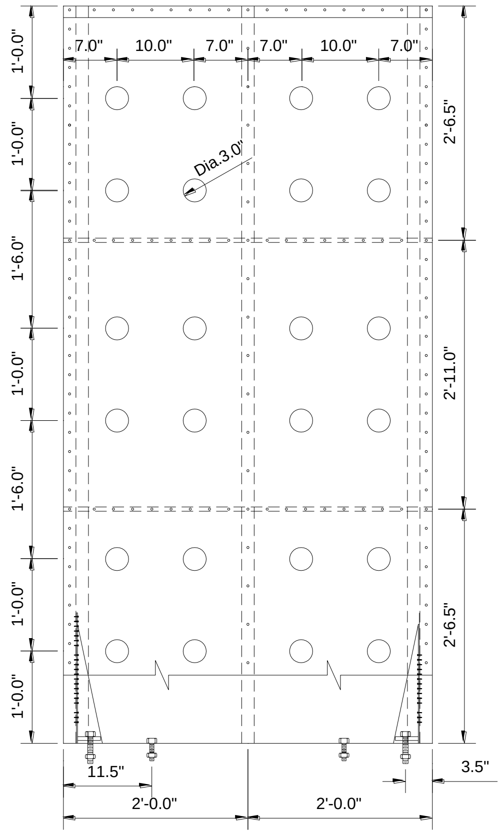
Observed Failure Mode: large relative out-of-plane movement of the sheet elements at the both side of circles, resulting in the repture of sheathing. screws along the lower seam joint became loose.

Screw Pull Out: None

Sheathing Tear: Yes

Screw Pull Over: Yes




1'-0.0"
7.0"
10.0"
7.0"
7.0"
10.0"
7.0"
1'-0.0"
Dia.3.0"
2'-6.5"
1'-6.0"
1'-0.0"
2'-11.0"
1'-6.0"
1'-0.0"
2'-6.5"
1'-0.0"
11.5"
3.5"
2'-0.0"
2'-0.0"


2013 01 09

Opening Type 7: 24x3-in vertical slits. **Test No. 10**

Test date: Jan. 09, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft. Studs: 350S162-68, 33ksi Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 2950plf.

Lateral displacement of wall top at +peak load: 3.223 in.

-Peak load: 2925plf.

Lateral displacement of wall top at -peak load: 3.308 in.

Average peak load: 2937.5plf.

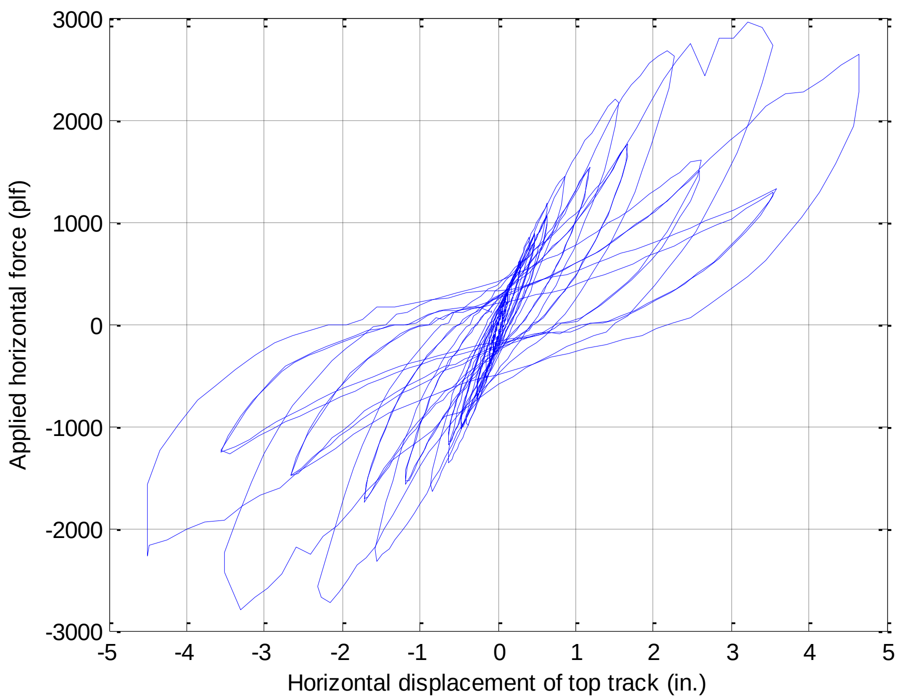
Average lateral displacement of wall top: 3.266 in.

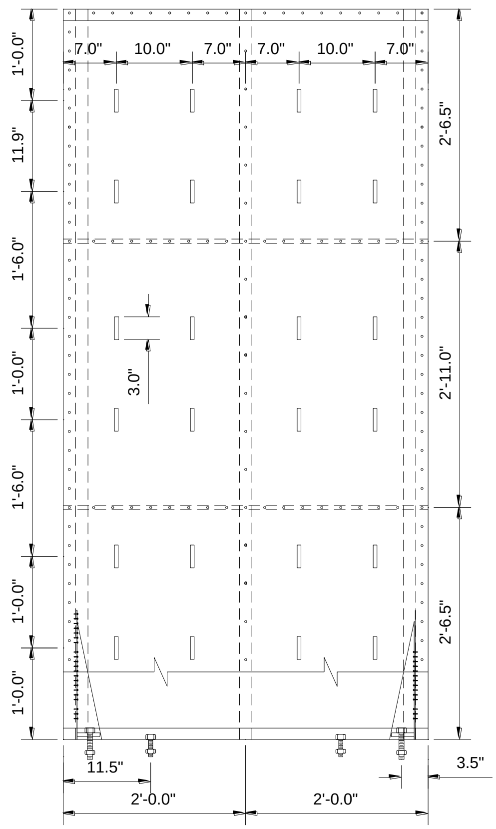
Observed Failure Mode: large relative out-of-plane movement of the sheet elements at the both side of vertical slits, resulting in the repture of sheathing. screws along the lower seam joint became loose.

Screw Pull Out: None

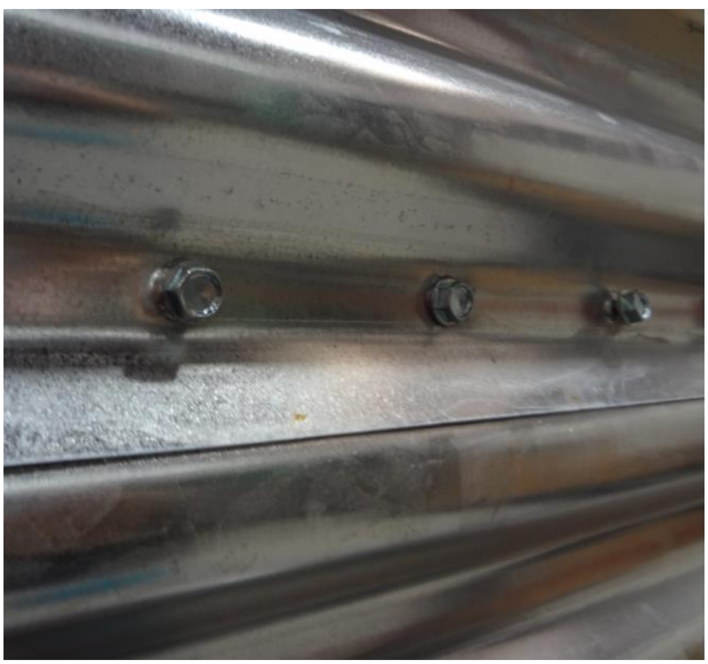
Sheathing Tear: Yes

Screw Pull Over: None




7.0"
10.0"
7.0"
7.0"
10.0"
7.0"
1'-0.0"
11.9"
1'-6.0"
1'-0.0"
1'-6.0"
1'-0.0"
1'-0.0"
3.0"
2'-6.5"
2'-11.0"
2'-6.5"
11.5"
3.5"
2'-0.0"
2'-0.0"


2013 01 10



Opening Type 7: 24x3-in vertical slits. **Test No. 13**

Test date: Jan. 16, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft. Studs: 350S162-68, 33ksi Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 3180plf.

Lateral displacement of wall top at +peak load: 2.598 in.

-Peak load: 2747.5plf.

Lateral displacement of wall top at -peak load: 2.29 in.

Average peak load: 2963.8plf.

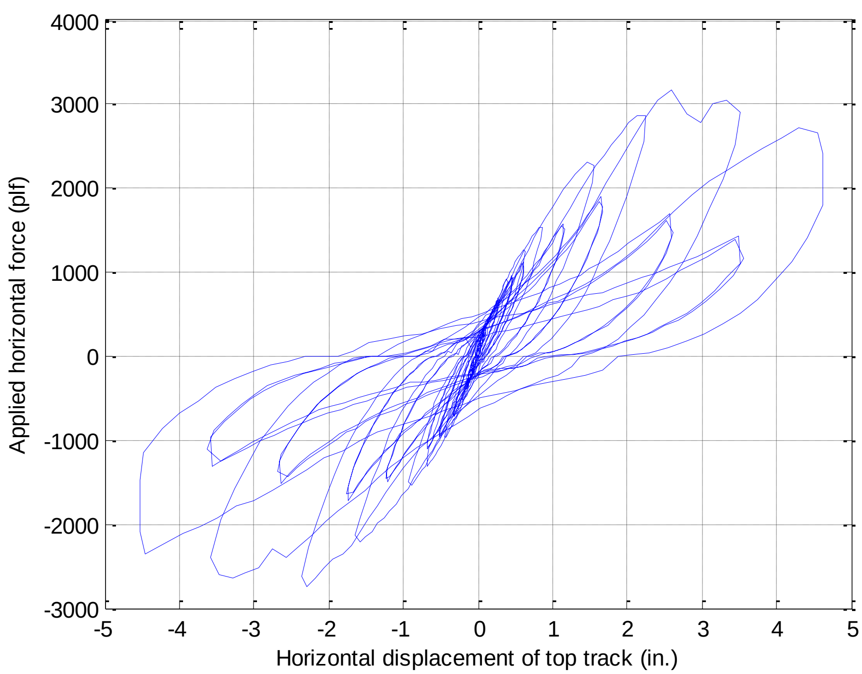
Average lateral displacement of wall top: 2.444 in.

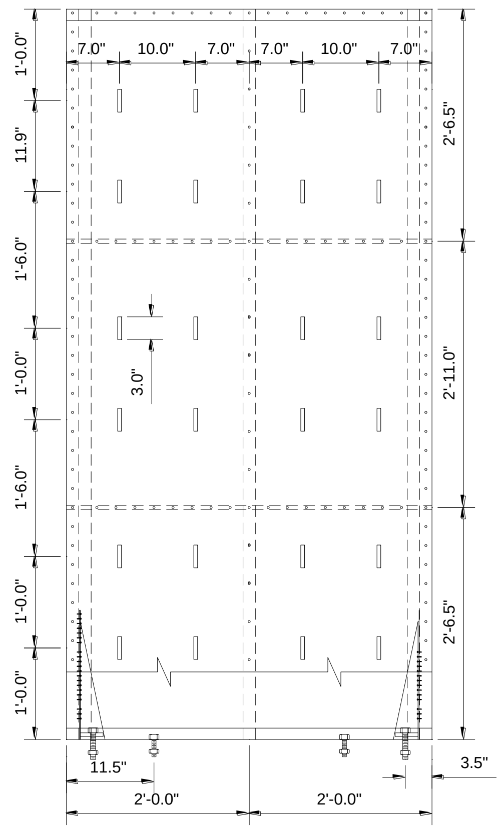
Observed Failure Mode: large relative out-of-plane movement of the sheet elements at the both side of vertical slits, resulting in the repture of sheathing. screws along the lower seam joint became loose.

Screw Pull Out: None

Sheathing Tear: Yes

Screw Pull Over: None





7.0"
10.0"
7.0"
7.0"
10.0"
7.0"
1'-0.0"
11.9"
1'-6.0"
1'-0.0"
1'-6.0"
1'-0.0"
1'-0.0"
3.0"
2'-6.5"
2'-11.0"
2'-6.5"
11.5"
3.5"
2'-0.0"
2'-0.0"

2013 01 17


2013 01 17

Opening Type 8: 24x3-in horizontal slits.   **Test No. 11**

Test date: Jan. 10, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft.   Studs: 350S162-68, 33ksi   Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 3180plf.

Lateral displacement of wall top at +peak load: 2.598 in.

-Peak load: 2747.5plf.

Lateral displacement of wall top at -peak load: 2.29 in.

Average peak load: 2963.8plf.

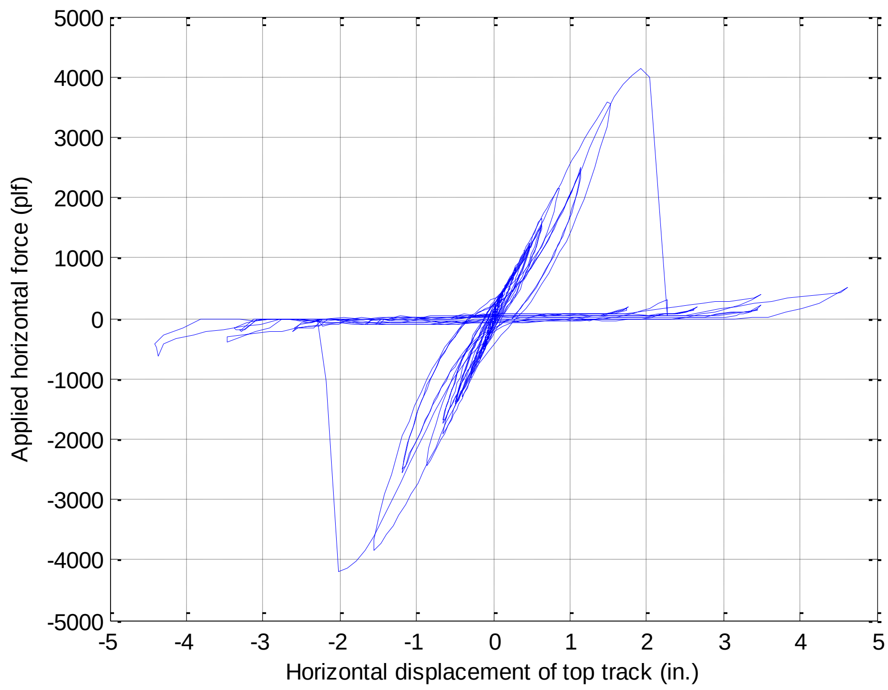
Average lateral displacement of wall top: 2.444 in.

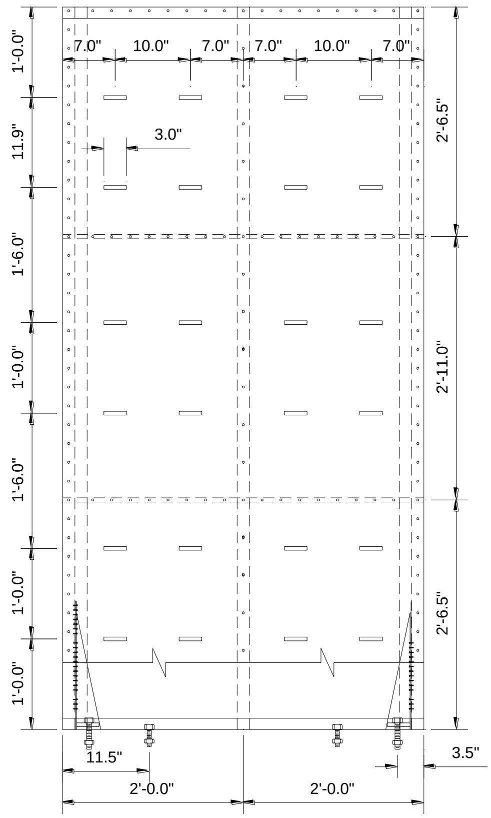
Observed Failure Mode: specimen failed in the broken at hold down bolts. No other failure was observed.

Screw Pull Out: None

Sheathing Tear: None

Screw Pull Over: None




7.0"
10.0"
7.0"
7.0"
10.0"
7.0"
3.0"
1'-0.0"
11.9"
1'-6.0"
1'-0.0"
1'-6.0"
1'-0.0"
1'-0.0"
2'-6.5"
2'-11.0"
2'-6.5"
11.5"
3.5"
2'-0.0"
2'-0.0"


2013 01 11

Opening Type 9: 12x2-in vertical slits.   **Test No. 14**

Test date: Jan. 18, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft.   Studs: 350S162-68, 33ksi   Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 3920plf.

Lateral displacement of wall top at +peak load: 2.138 in.

-Peak load: 3217.5plf.

Lateral displacement of wall top at -peak load: 1.584 in.

Average peak load: 3568.8plf.

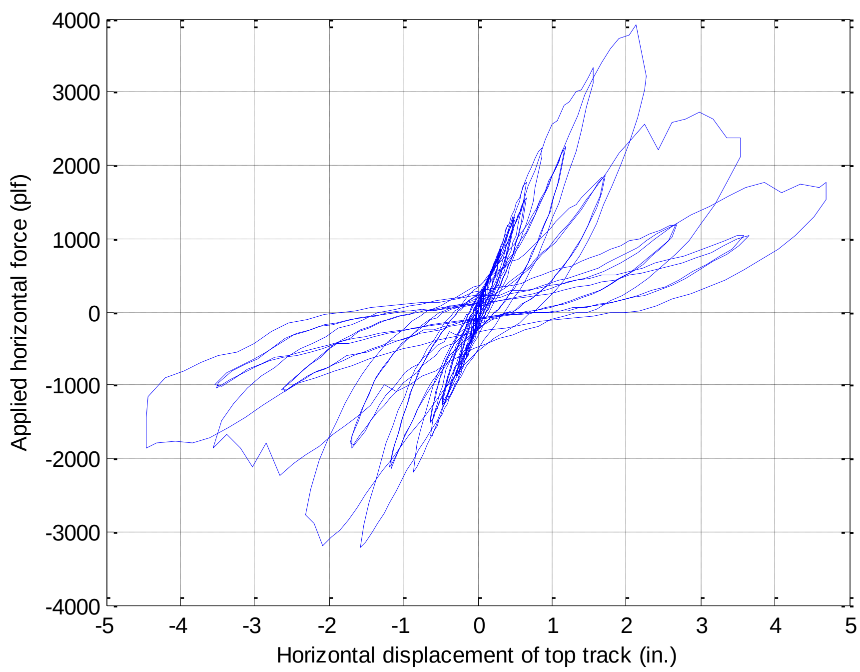
Average lateral displacement of wall top: 1.861 in.

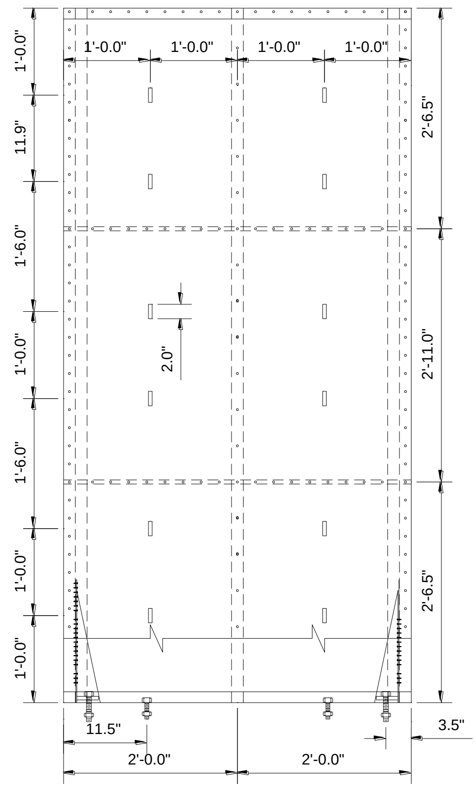
Observed Failure Mode: large relative out-of-plane movement of the sheet elements at the both side of vertical slits, resulting in the repture of sheathing. The slits which were aligned vertically connected together.

Screw Pull Out: None

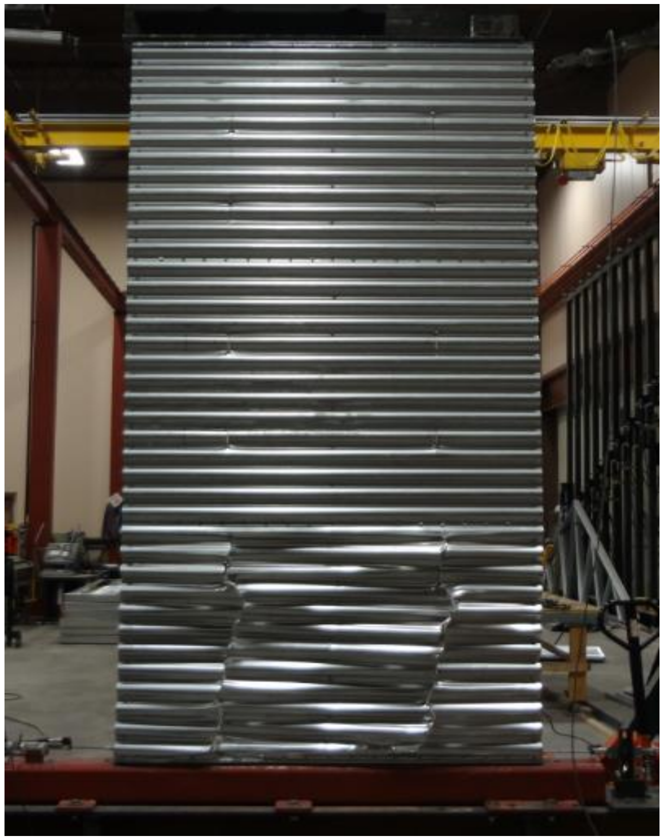
Sheathing Tear: Yes

Screw Pull Over: None




1'-0.0"
1'-0.0"
1'-0.0"
1'-0.0"
1'-0.0"
11.9"
1'-6.0"
1'-0.0"
2.0"
1'-6.0"
1'-0.0"
1'-0.0"
2'-6.5"
2'-11.0"
2'-6.5"
11.5"
3.5"
2'-0.0"
2'-0.0"


2013 01 19




2013

Opening Type 10: 24x1-in vertical slits.   **Test No. 16**

Test date: Jan. 18, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft.   Studs: 350S162-68, 33ksi   Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 3920plf.

Lateral displacement of wall top at +peak load: 2.138 in.

-Peak load: 3217.5plf.

Lateral displacement of wall top at -peak load: 1.584 in.

Average peak load: 3568.8plf.

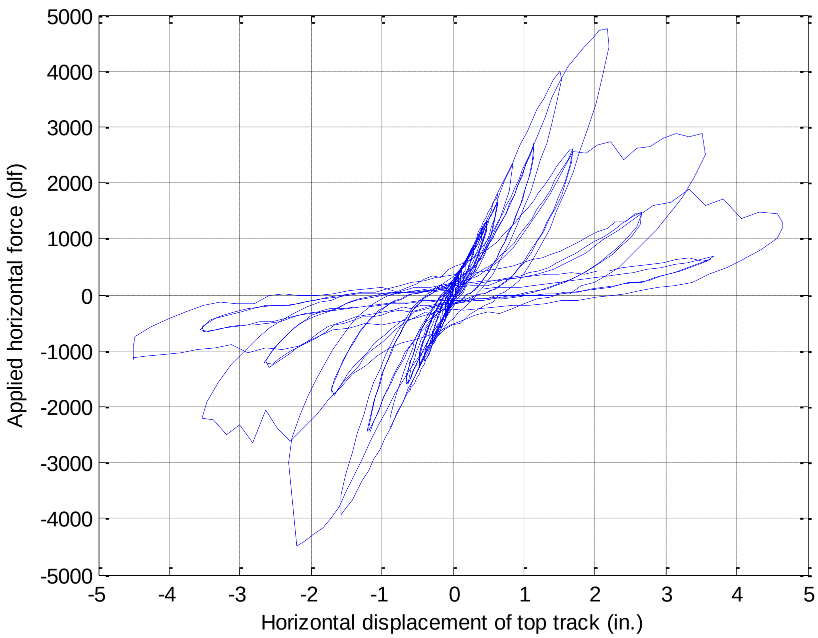
Average lateral displacement of wall top: 1.861 in.

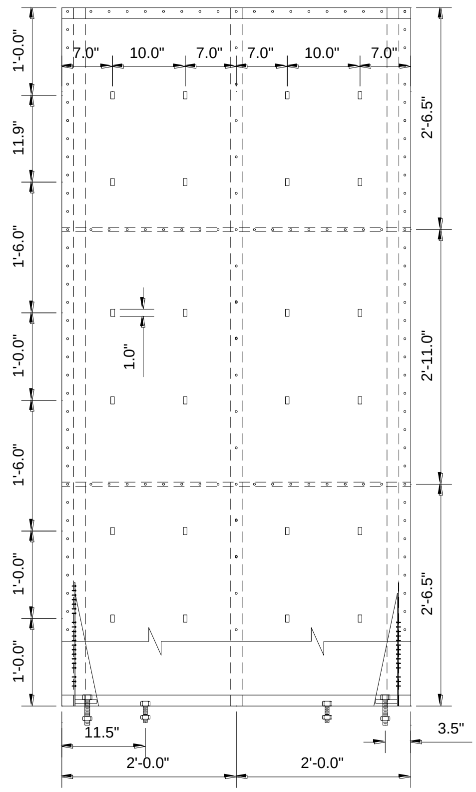
Observed Failure Mode: the lower sheet steel overall buckled and unzipped.

Screw Pull Out: Yes

Sheathing Tear: None

Screw Pull Over: Yes




1'-0.0"
11.9"
1'-6.0"
1'-0.0"
1'-6.0"
1'-0.0"
1'-0.0"
7.0"
10.0"
7.0"
7.0"
10.0"
7.0"
2'-6.5"
2'-11.0"
2'-6.5"
1.0"
11.5"
3.5"
2'-0.0"
2'-0.0"


2013 01 23


19
20
2013 01 23

Opening Type 11: 24x2-in vertical slits. **Test No. 18**

Test date: Jan. 23, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft. Studs: 350S162-68, 33ksi Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Monotonic-ASTM

Test results:

+Peak load: 3092.5plf.

Lateral displacement of wall top at +peak load: 3.741 in.

-Peak load: NA.

Lateral displacement of wall top at -peak load: NA

Average peak load: 3092.5plf.

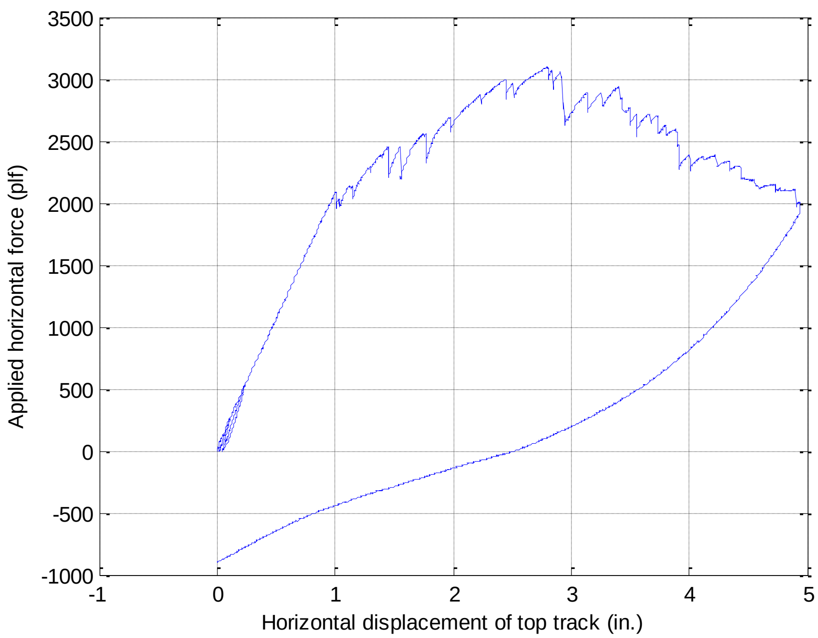
Average lateral displacement of wall top: 3.741 in.

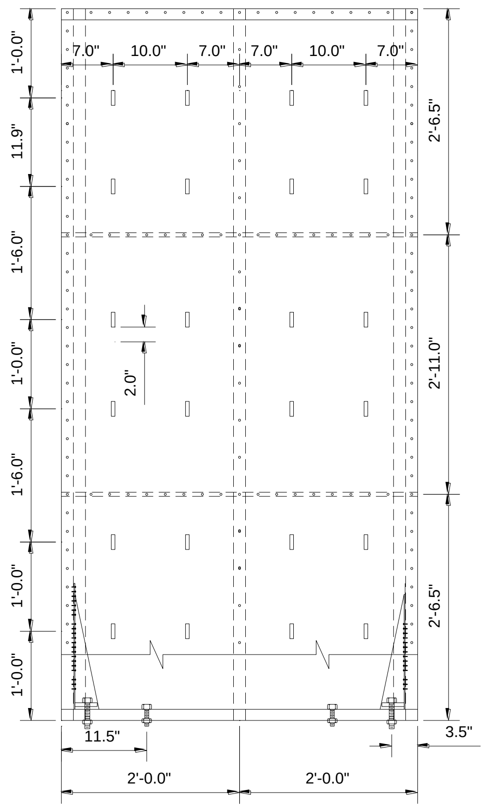
Observed Failure Mode: large relative out-of-plane movement of the sheet elements at the both side of vertical slits, resulting in the repture of sheathing. Some screws became loose.

Screw Pull Out: None

Sheathing Tear: Yes

Screw Pull Over: None




1'-0.0"
7.0"
10.0"
7.0"
7.0"
10.0"
7.0"
11.9"
2'-6.5"
1'-6.0"
1'-0.0"
2.0"
2'-11.0"
1'-6.0"
1'-0.0"
2'-6.5"
1'-0.0"
11.5"
3.5"
2'-0.0"
2'-0.0"


2013 0

Opening Type 11: 24x2-in vertical slits.   **Test No. 15**

Test date: Jan. 18, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft.   Studs: 350S162-68, 33ksi   Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 3207.5plf.

Lateral displacement of wall top at +peak load: 3.465 in.

-Peak load: 2982.5plf

Lateral displacement of wall top at -peak load: 2.15 in.

Average peak load: 3095plf.

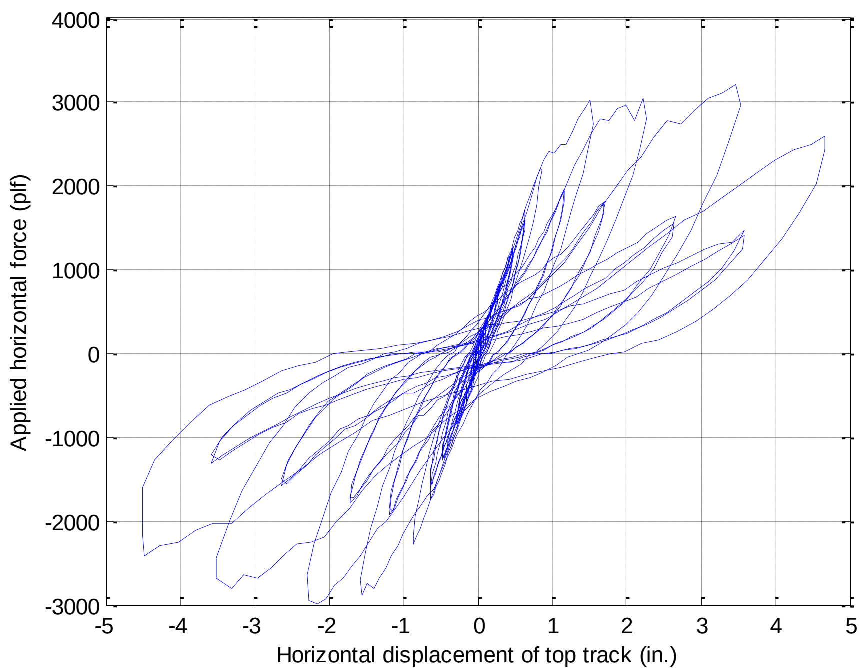
Average lateral displacement of wall top: 2.808 in.

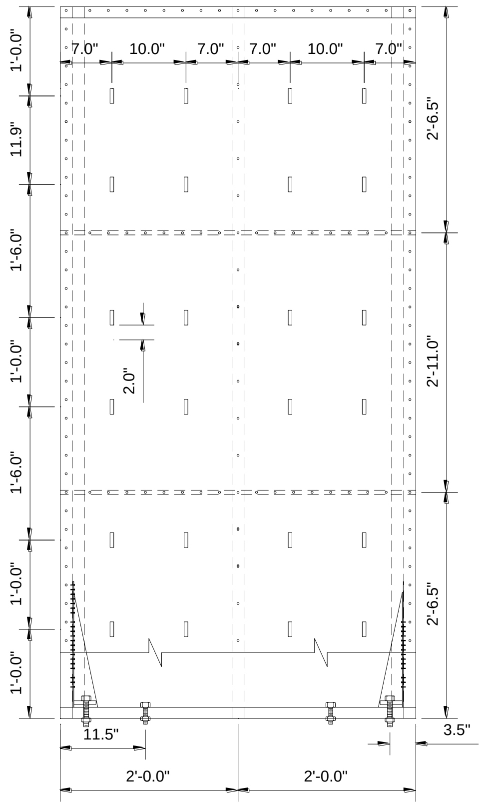
Observed Failure Mode: large relative out-of-plane movement of the sheet elements at the both side of vertical slits, resulting in the repture of sheathing. Some screws became loose.

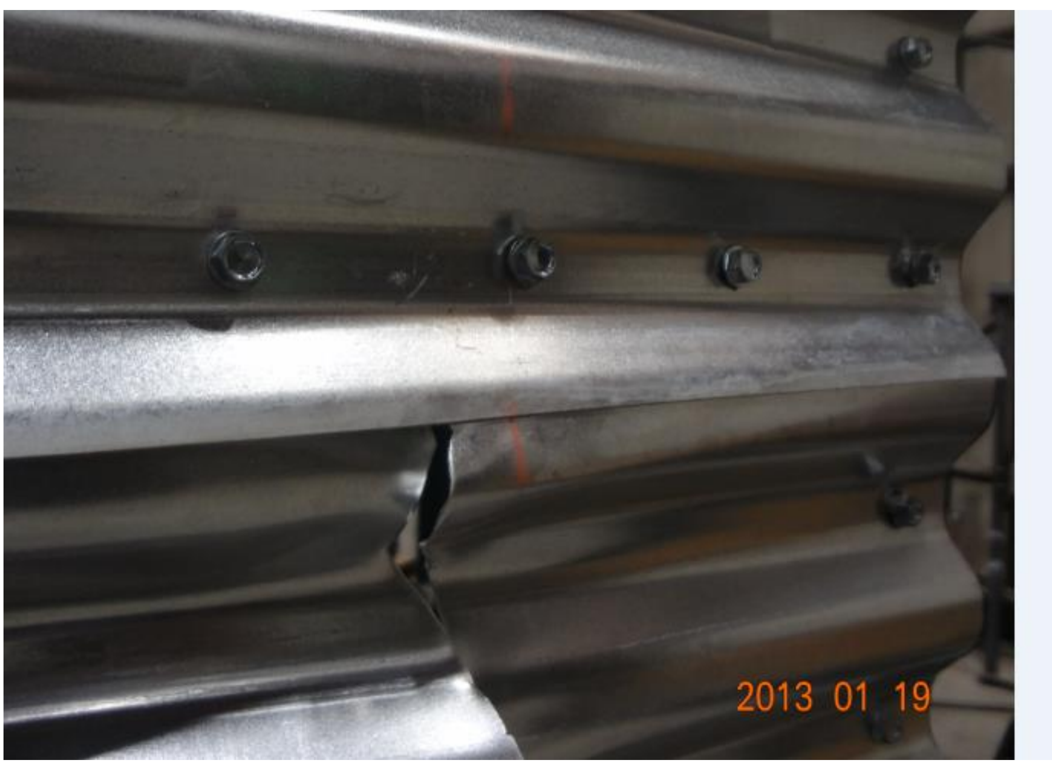
Screw Pull Out: None

Sheathing Tear: Yes

Screw Pull Over: None




1'-0.0"
7.0"
10.0"
7.0"
7.0"
10.0"
7.0"
11.9"
2'-6.5"
1'-6.0"
1'-0.0"
2.0"
2'-11.0"
1'-6.0"
1'-0.0"
2'-6.5"
1'-0.0"
11.5"
3.5"
2'-0.0"
2'-0.0"


2013 01 19

Opening Type 11: 24x2-in vertical slits.   **Test No. 17**

Test date: Jan. 23, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft.   Studs: 350S162-68, 33ksi   Tracks: 350T150-68, 33ksi

Steel sheathing : 0.027 in. 90ksi, Vulcraft 0.6C decking

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 3110plf.

Lateral displacement of wall top at +peak load: 3.43 in.

-Peak load: 3095plf

Lateral displacement of wall top at -peak load: 3.397 in.

Average peak load: 3102.5plf.

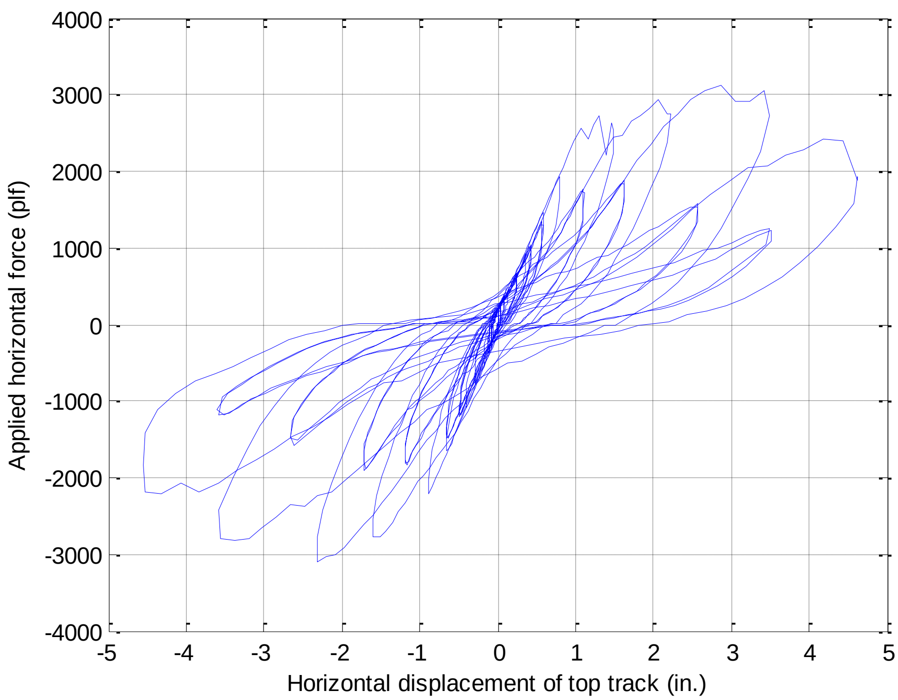
Average lateral displacement of wall top: 3.414 in.

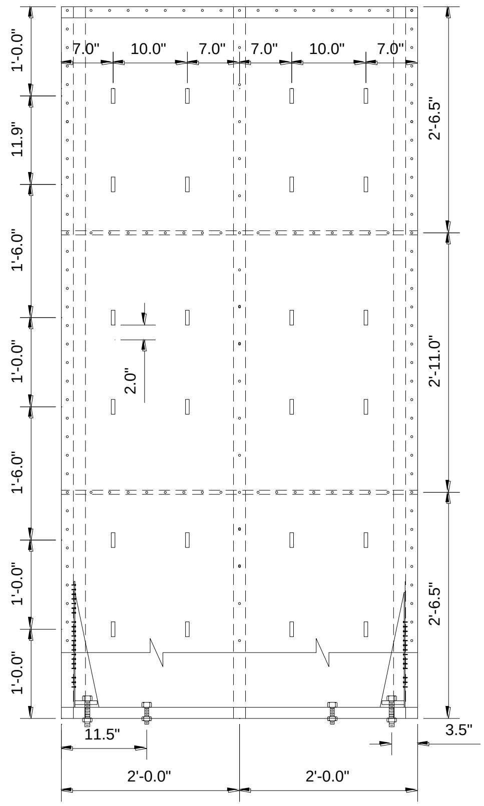
Observed Failure Mode: large relative out-of-plane movement of the sheet elements at the both side of vertical slits, resulting in the repture of sheathing. Some screws became loose.

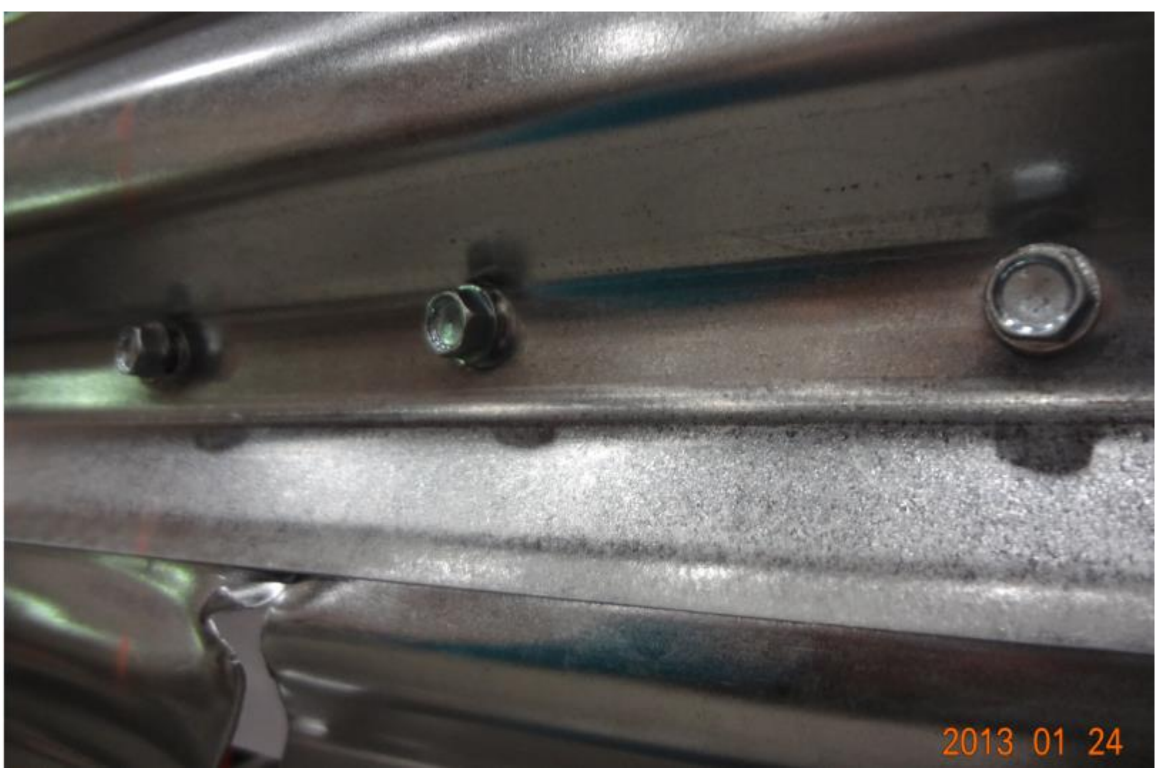
Screw Pull Out: None

Sheathing Tear: Yes

Screw Pull Over: None




1'-0.0"
7.0"
10.0"
7.0"
7.0"
10.0"
7.0"
11.9"
2'-6.5"
1'-6.0"
1'-0.0"
2.0"
2'-11.0"
1'-6.0"
1'-0.0"
2'-6.5"
1'-0.0"
11.5"
3.5"
2'-0.0"
2'-0.0"


2013 01 24


19
20
2013 01 24

Opening Type 12: plywood **Test No. 20**

Test date: Jan. 25, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft. Studs: 350S162-68, 33ksi Tracks: 350T150-68, 33ksi

Steel sheathing : 15/32 in. plywood

Fastener: #12x 1-1/4” hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD15S

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 3505plf.

Lateral displacement of wall top at +peak load: 3.263 in.

-Peak load: 2927.5plf

Lateral displacement of wall top at -peak load: 2.337 in.

Average peak load: 3216.3plf.

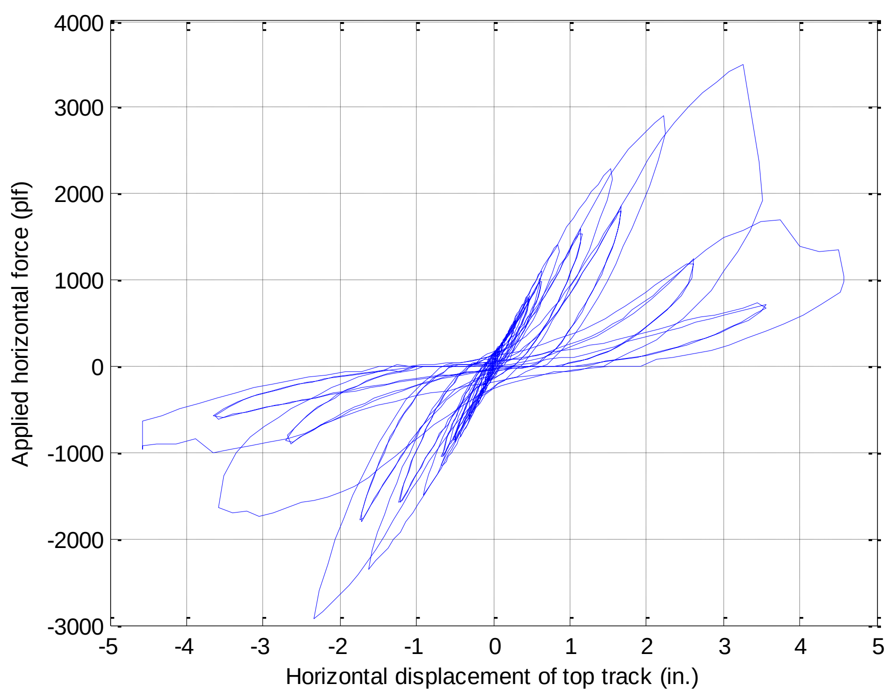
Average lateral displacement of wall top: 2.8 in.

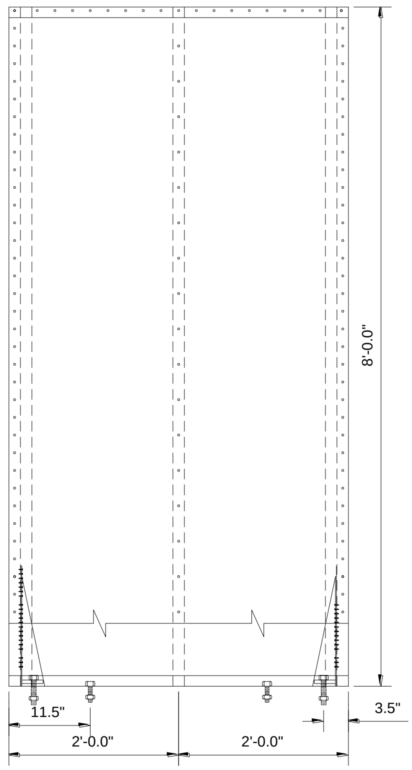
Observed Failure Mode: shear failure resulting in rupture of sheathing horizontally. Then sheathing was beared around screws on the boundary stud.

Screw Pull Out: None

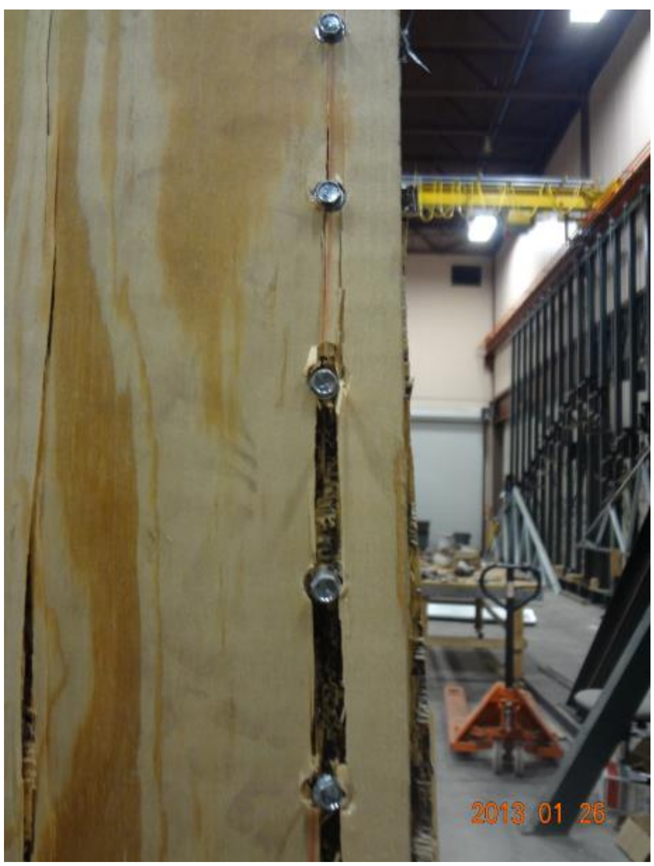
Sheathing Tear: Yes

Screw Pull Through: Yes




8'-0.0"
11.5"
3.5"
2'-0.0"
2'-0.0"


2013 01 26


2013 01 26

Opening Type 12: OSB **Test No. 21**

Test date: Jan. 28, 2013

Specimen Configuration:

Wall dimensions : 8 ft. x 4 ft. Studs: 350S162-68, 33ksi Tracks: 350T150-68, 33ksi

Steel sheathing : 7/16 in. OSB

Fastener: #12x 1-1/4" hex head washer self-drilling screws, 2.5/5.0 in. spacing.

Hold-down: Simpson Strong Tie S/HD159

Test protocol: Cyclic-CUREE

Test results:

+Peak load: 3595plf.

Lateral displacement of wall top at +peak load: 3.336 in.

-Peak load: 3282.5plf

Lateral displacement of wall top at -peak load: 3.542 in.

Average peak load: 3438.8plf.

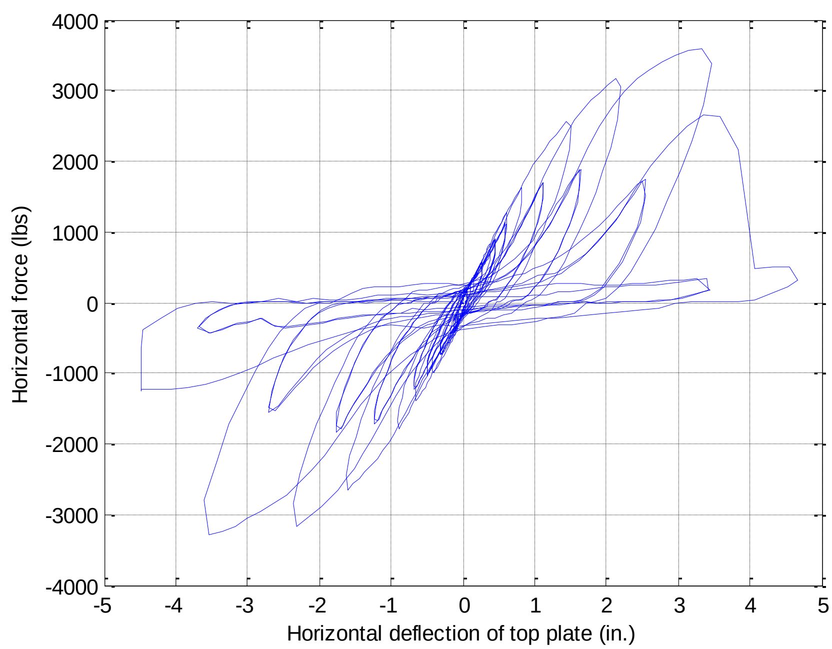
Average lateral displacement of wall top: 3.439 in.

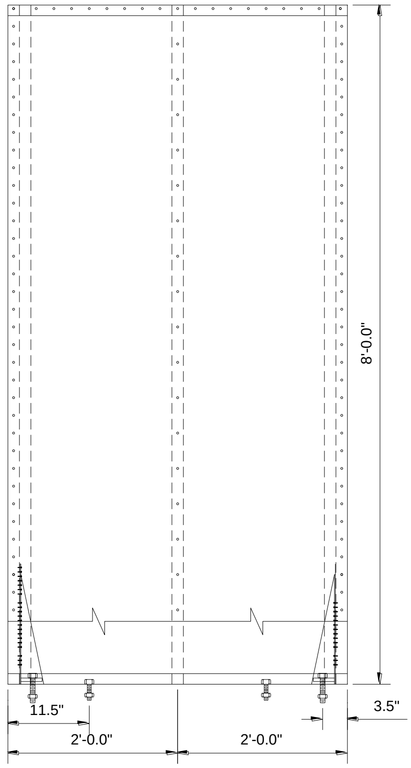
Observed Failure Mode: screws were pulled through head along bounday stud and bottom track.

Screw Pull Out: None

Sheathing Tear: Yes

Screw Pull Through: Yes



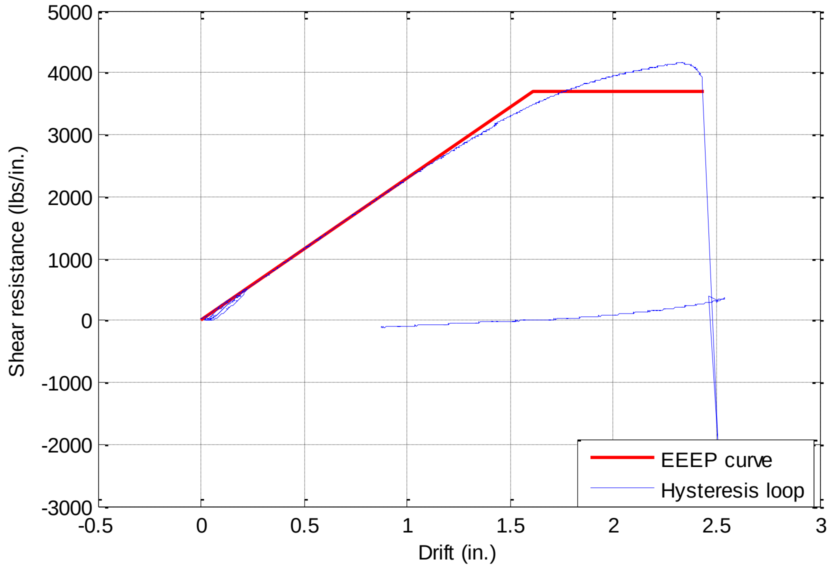
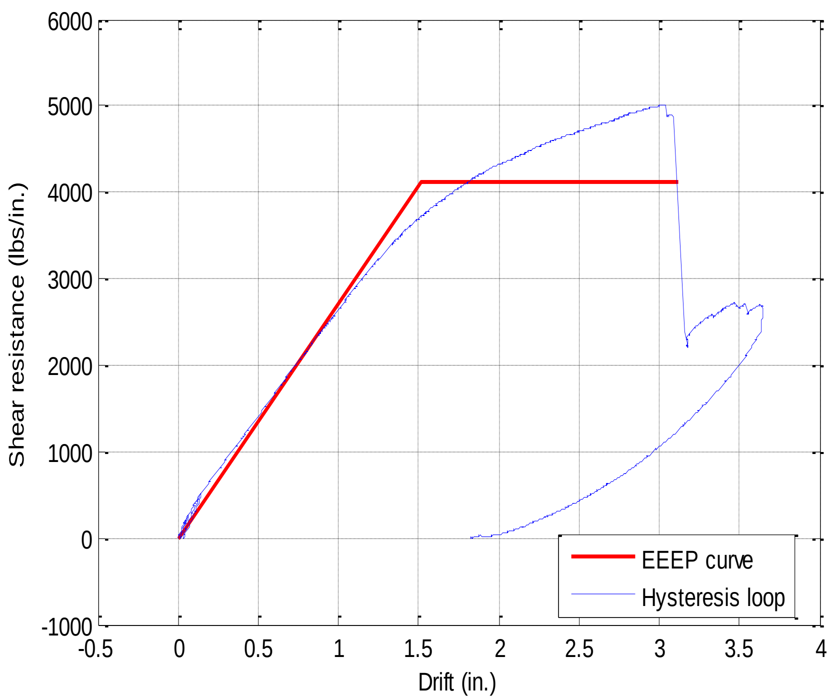

8'-0.0"
3.5"
11.5"
2'-0.0"
2'-0.0"

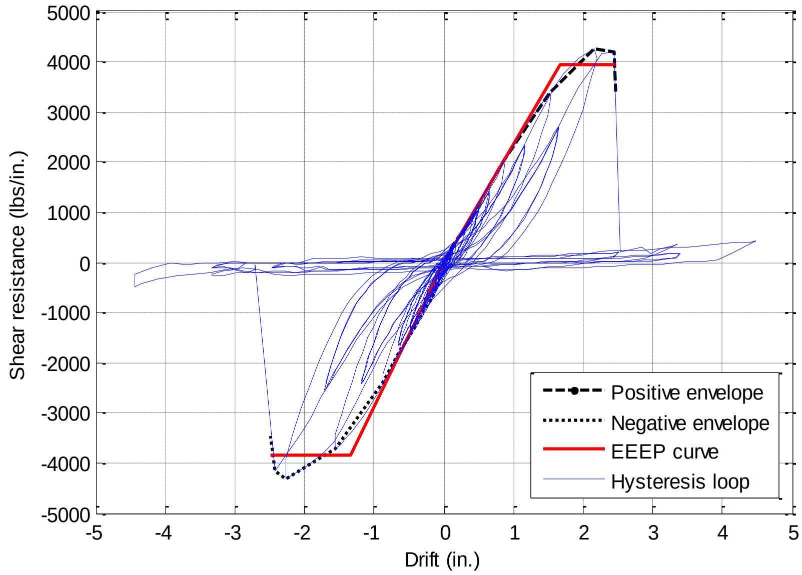
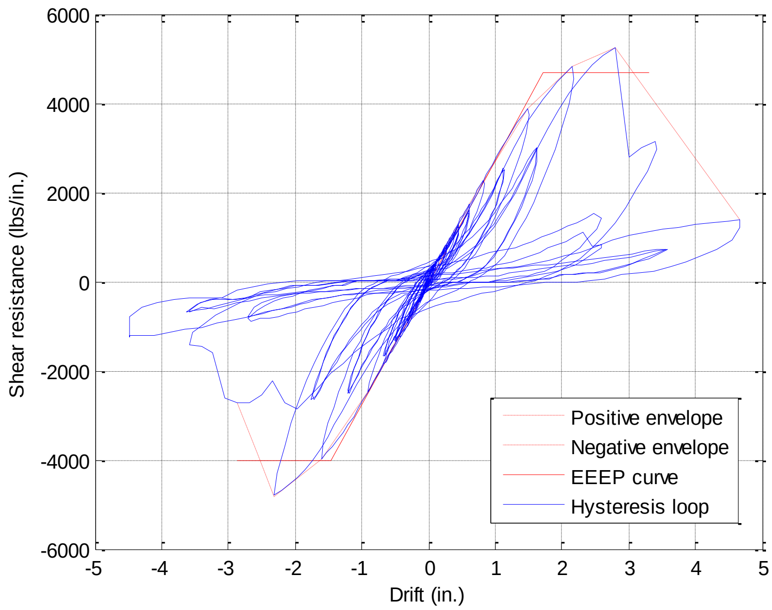

2013 01 29

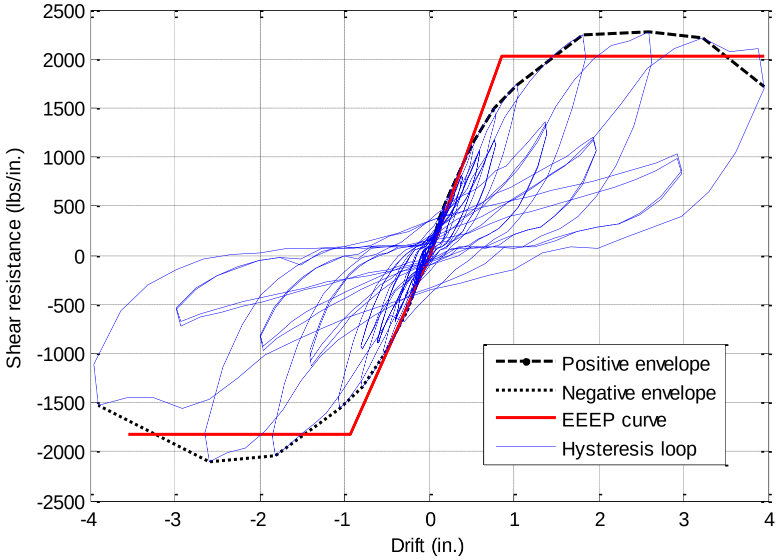
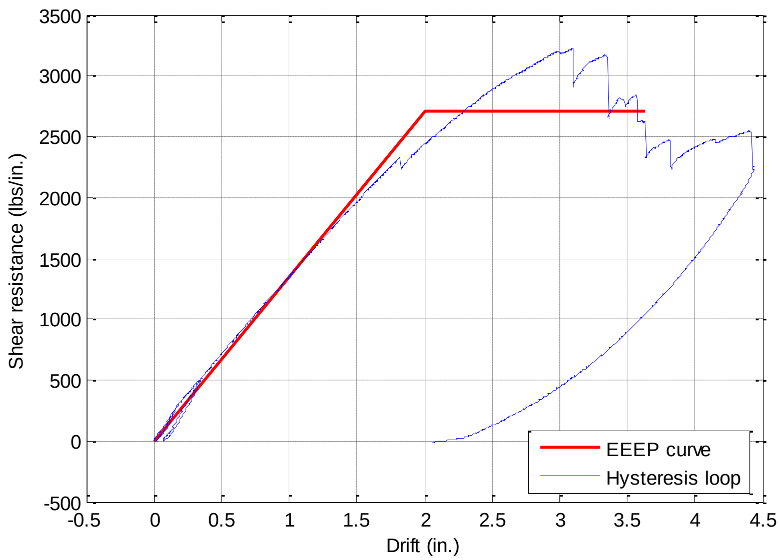

2013 01 29

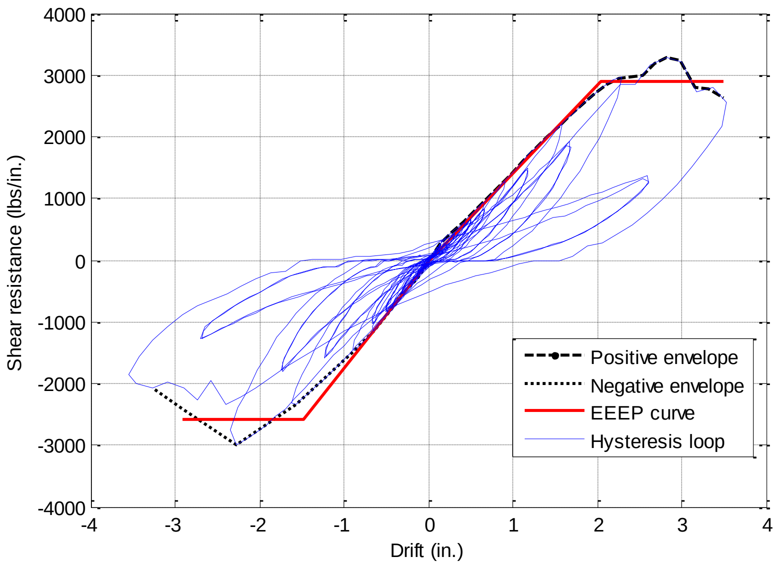
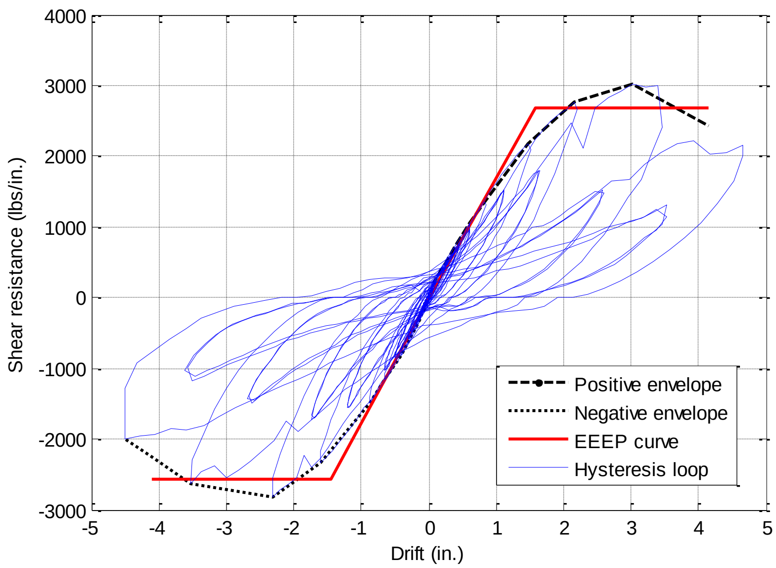
APPENDIX B

COMBINED HYSTERESIS CURVE, ENVELOPE CURVE, EEEP CURVE AND DUCTILITY FACTOR

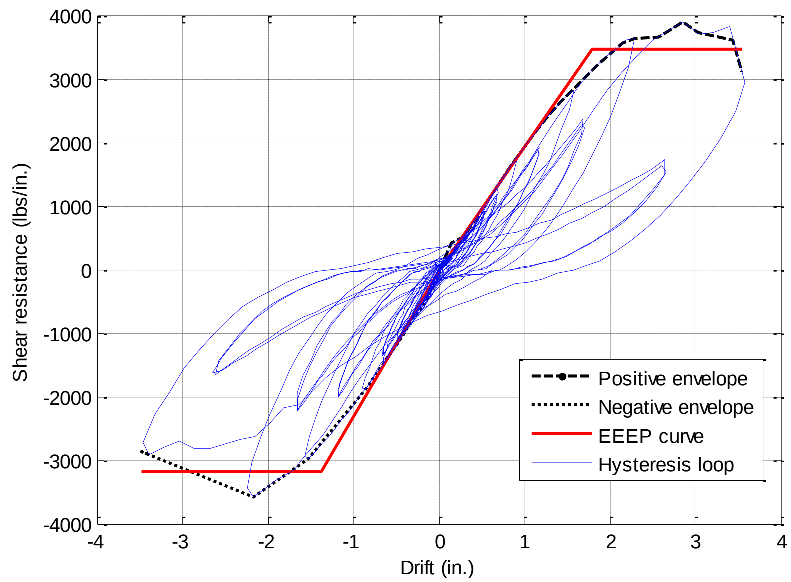
Label	Ductility factor	Graph
Type 1: No-opening No.02	1.511	
Type 1: No-opening No.12	2.051	

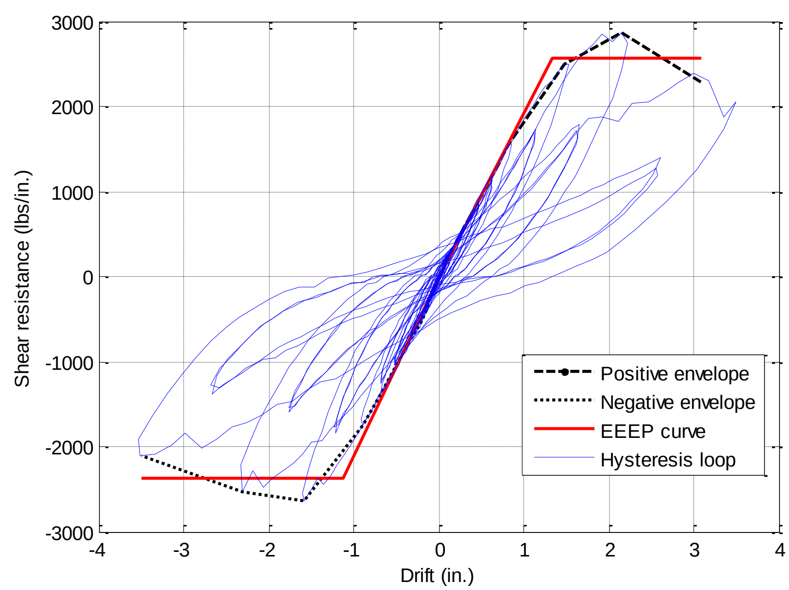
Type 1: No-opening No.07	1.644	
Type 1: No-opening No.19	2.123	

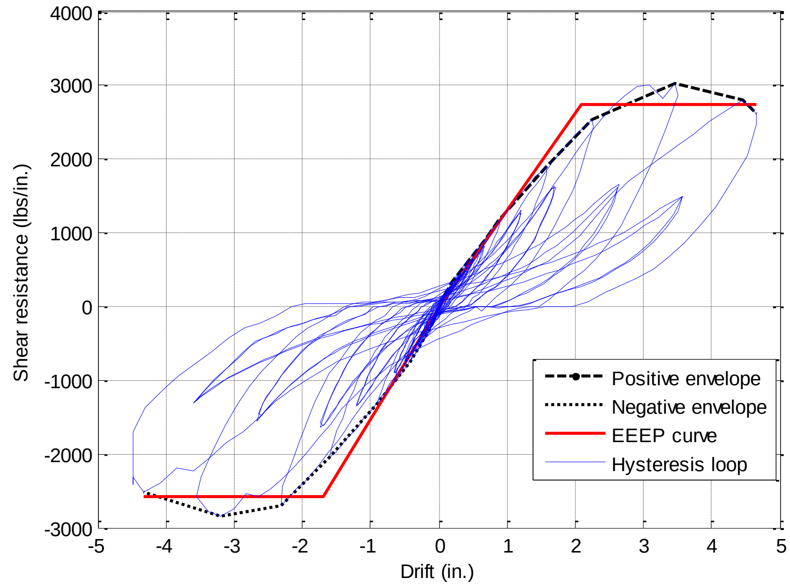
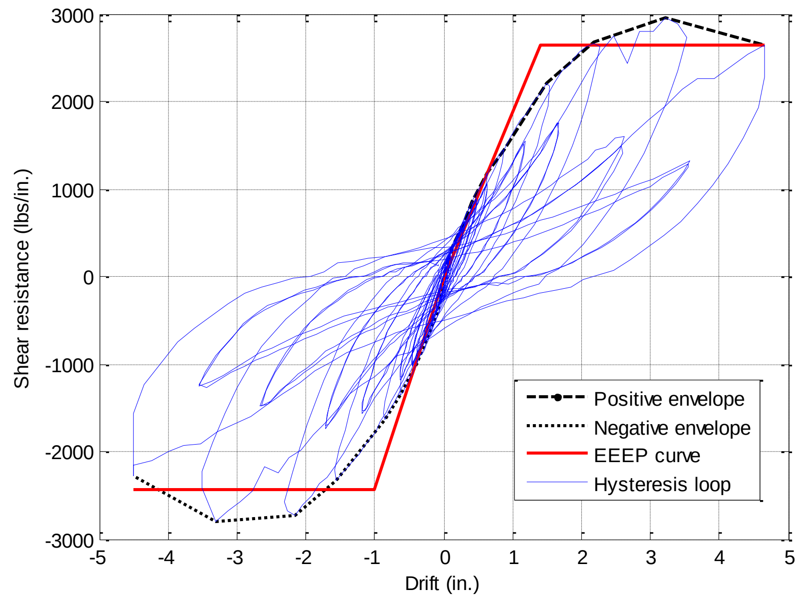
Type 2: No-seam stitch screws No.01	4.277	
Type 3: 6x6 in. circular holes No.03	1.678	

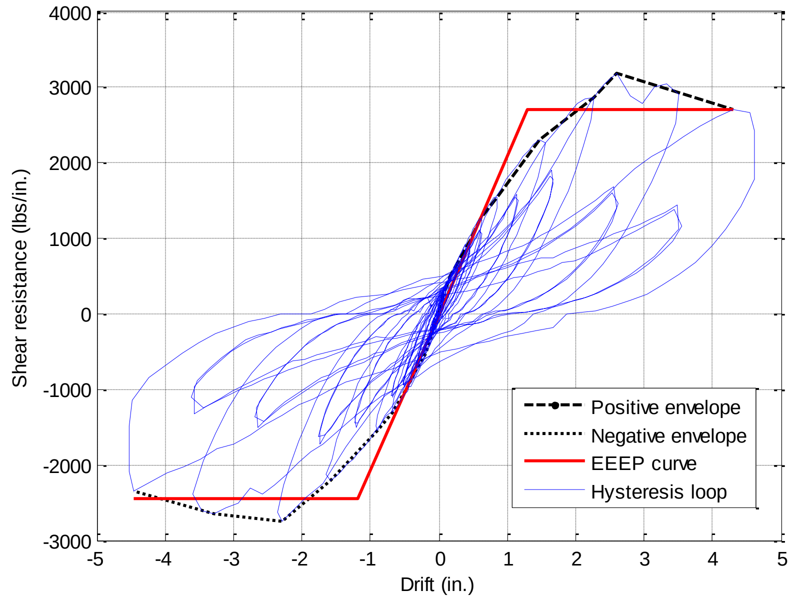
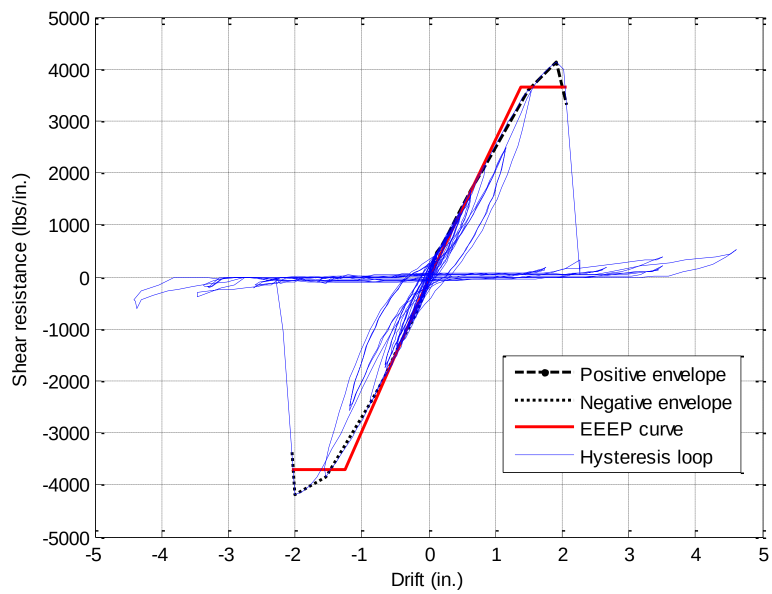
Type 3: 6x6 in. circular holes No.04	1.679	
Type 3: 6x6 in. circular holes No.08	2.415	

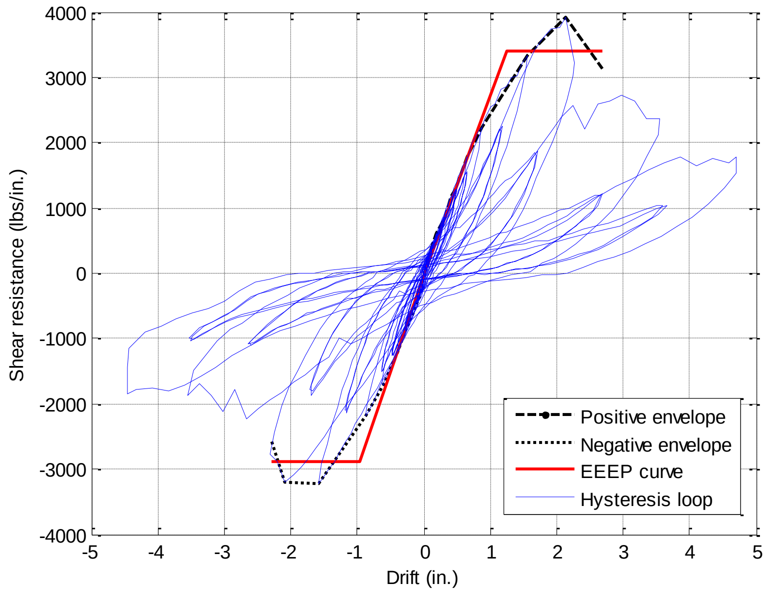
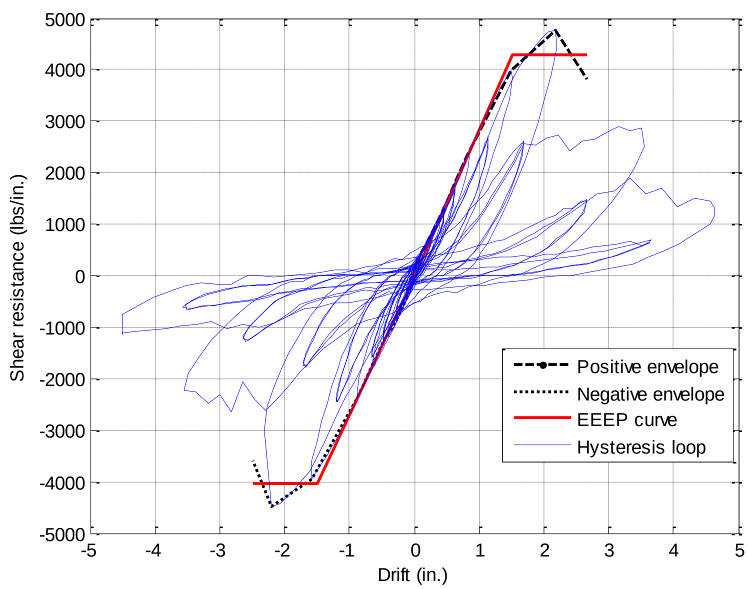
<table>
<tr><td>Type 4:
6x4 in.
circular
holes

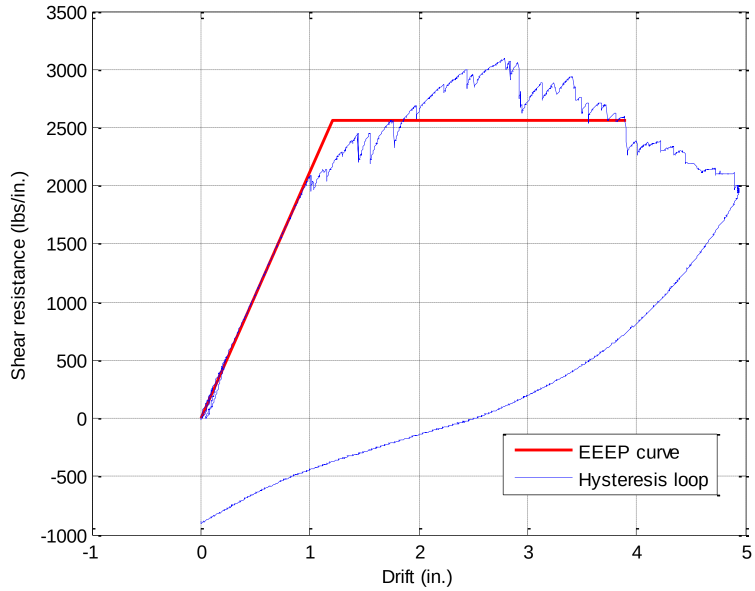
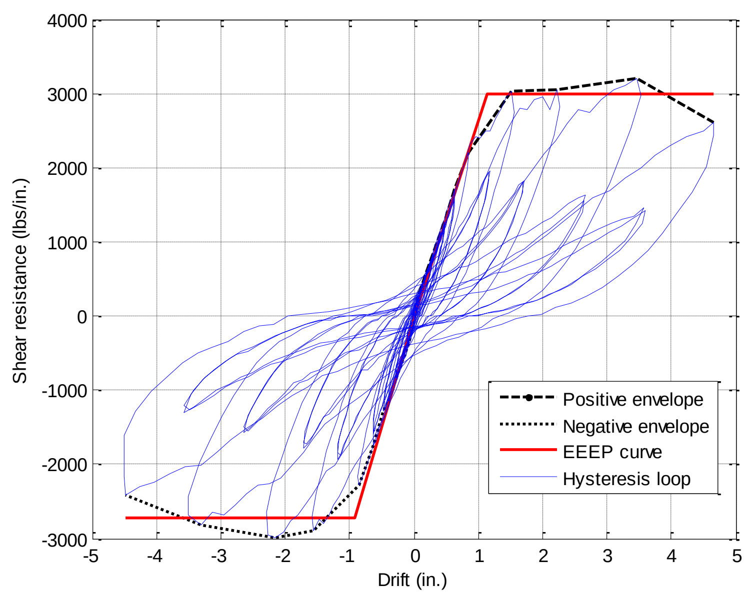
No.05</td><td>2.039</td><td>
</td></tr>
<tr><td>Type 5:
6x6 in.
vertical
slits

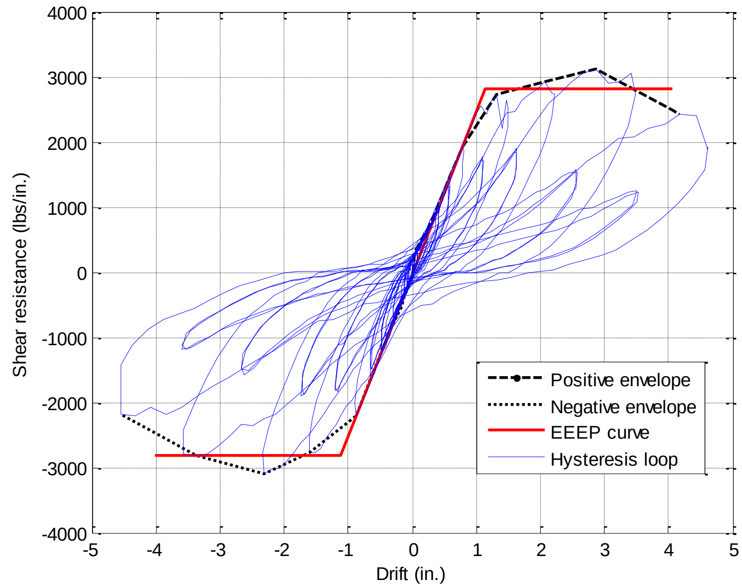
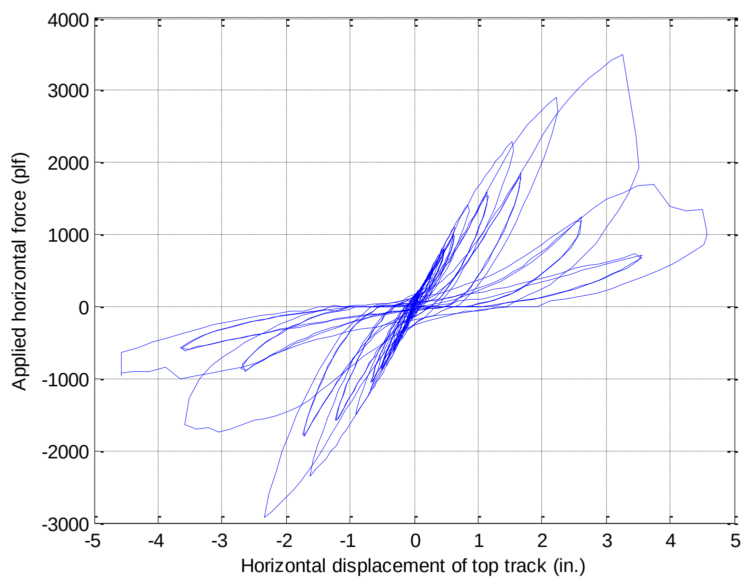
No.06</td><td>2.204</td><td>
</td></tr>
</table>

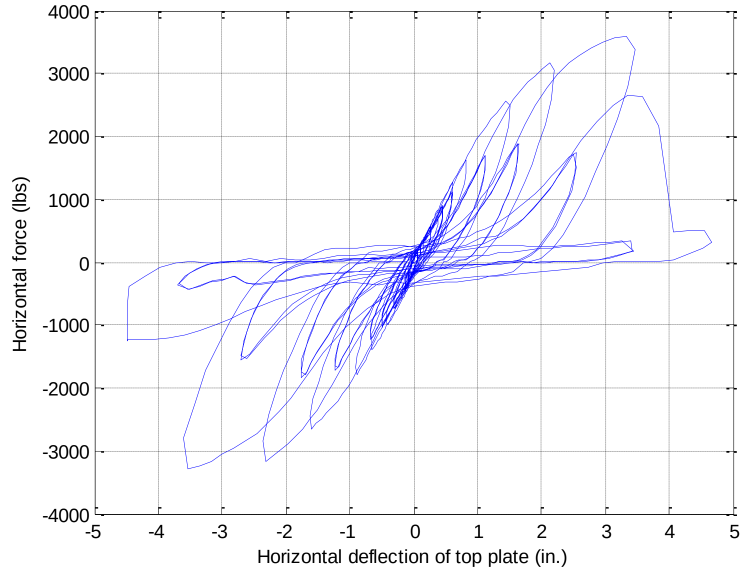
Type 6: 24x3 in. circular holes No.09	2.485	
Type 7: 24x3 in. vertical slits No.10	3.699	

Type 7: 24x3 in. vertical slits No.13	3.365	
Type 8: 24x3 in. horizontal slits No.11	1.534	

Type 9: 12x2 in. vertical slits No.14	2.128	
Type 10: 24x1 in. vertical slits No.16	1.295	

Type 11: 24x2 in. vertical slits No.18	3.090	
Type 11: 24x2 in. vertical slits No.15	3.646	

Type 11: 24x2 in. vertical slits No.16	3.207	
Type 12: 15/32 in. plywood No.20	-	

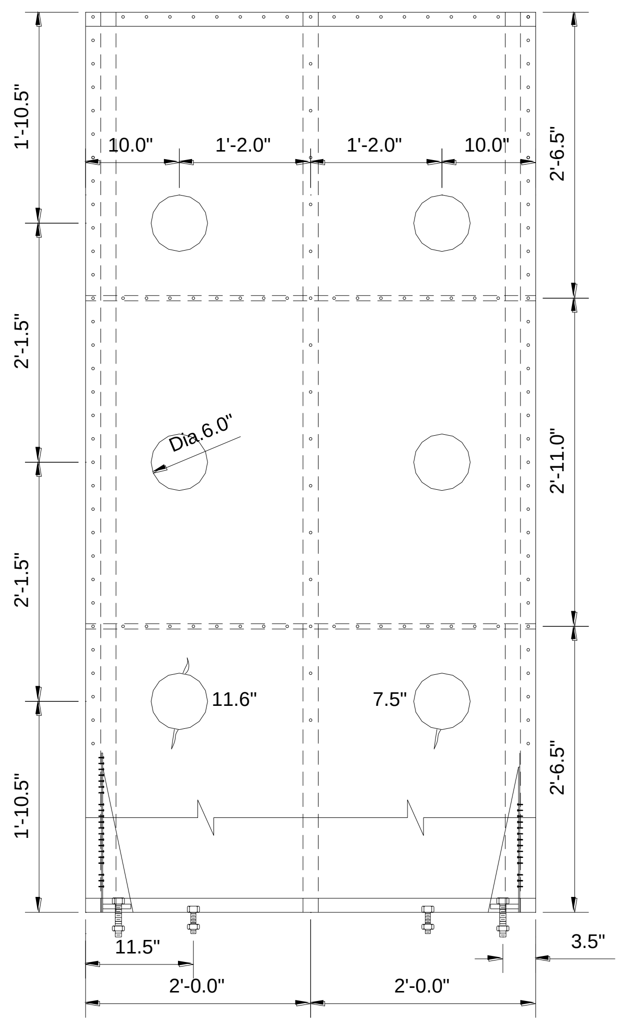
Type 12: 7/16 in. OSB No.21	-	

APPENDIX C

FINAL LENTH OF RUPTURED HOLES AND SLITS

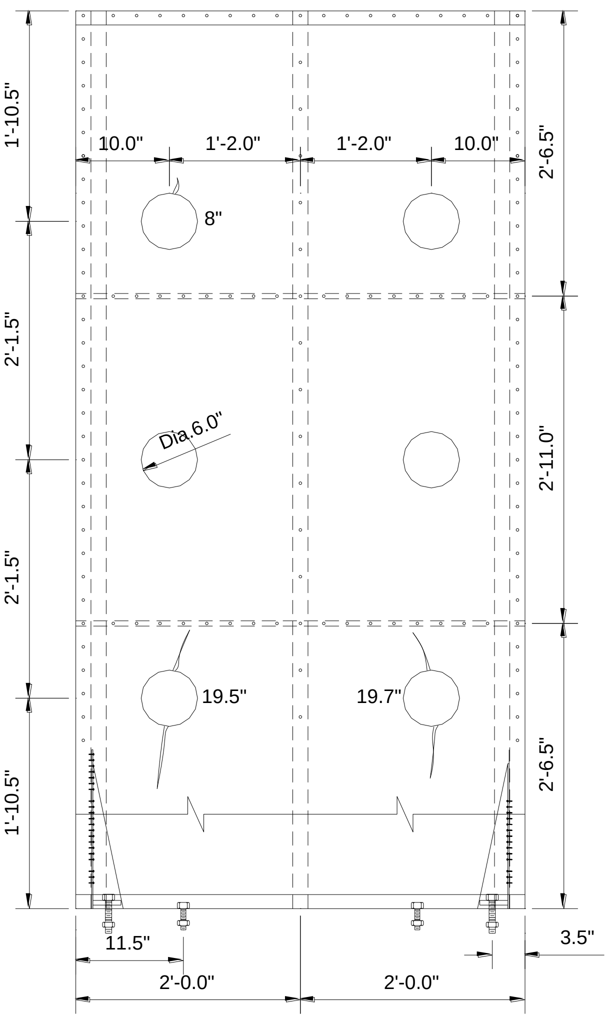
Type 3: 6x6-in circular holes

No. 03



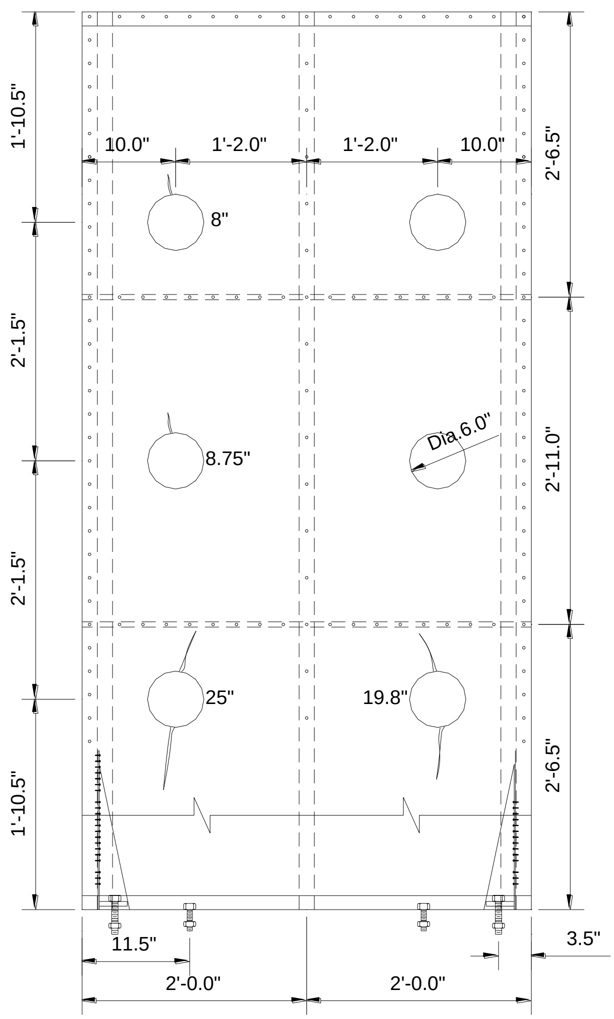
Type 3: 6x6-in circular holes

No. 04



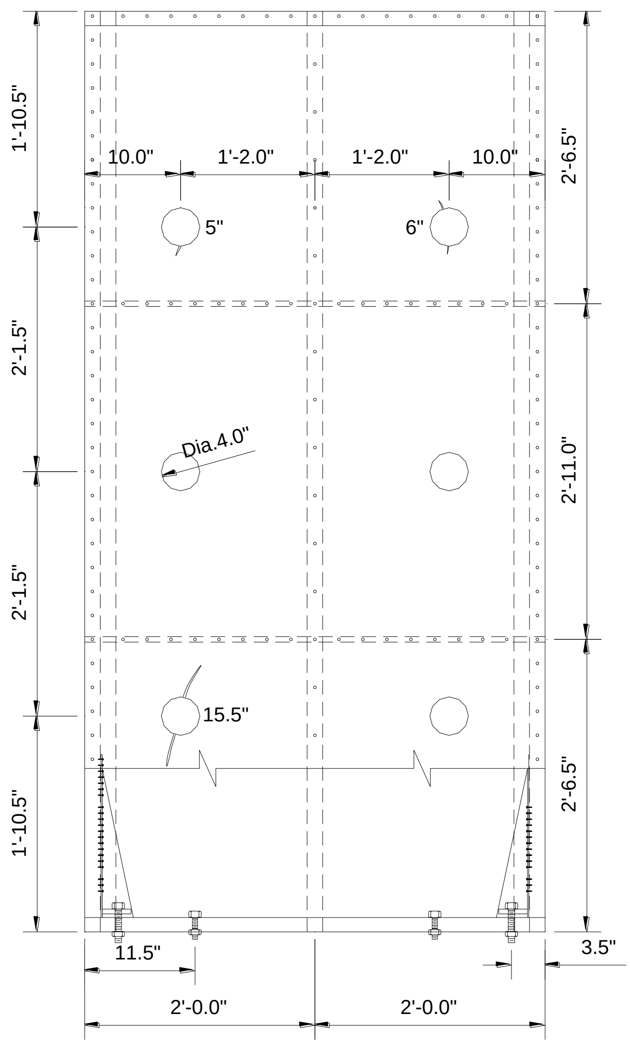
Type 3: 6x6-in circular holes

No. 08



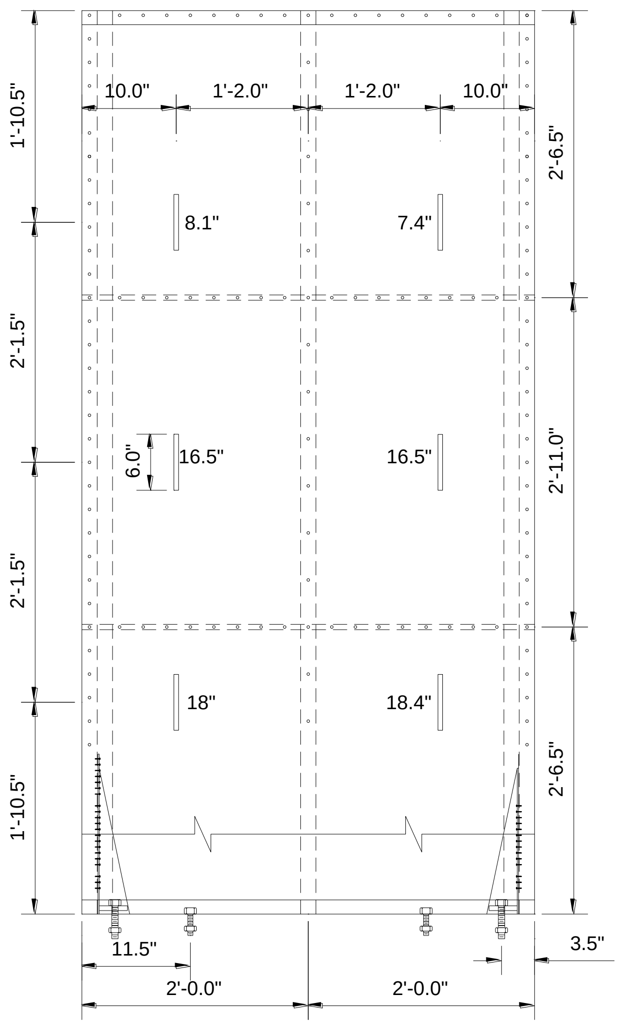
Type 4: 6x4-in circular holes

No. 05



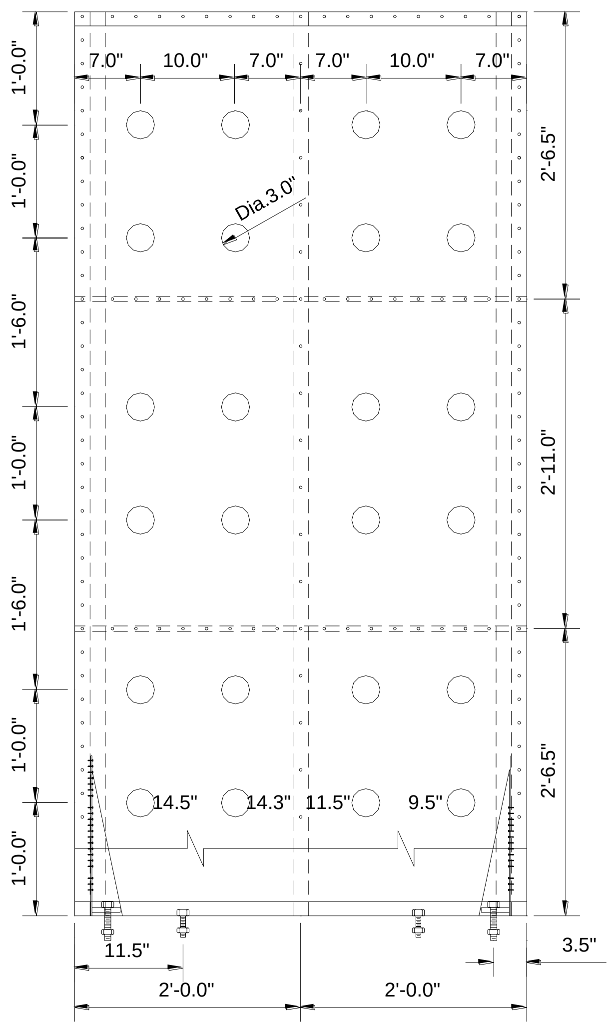
Type 5: 6x6-in vertical slits

No. 06



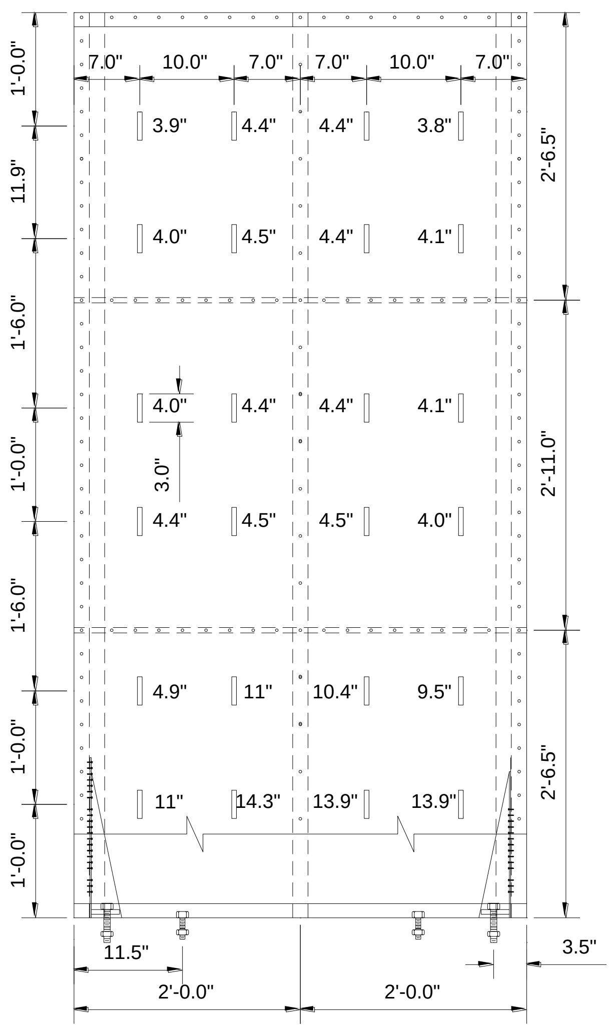
Type 6: 24x3-in circular holes

No. 09



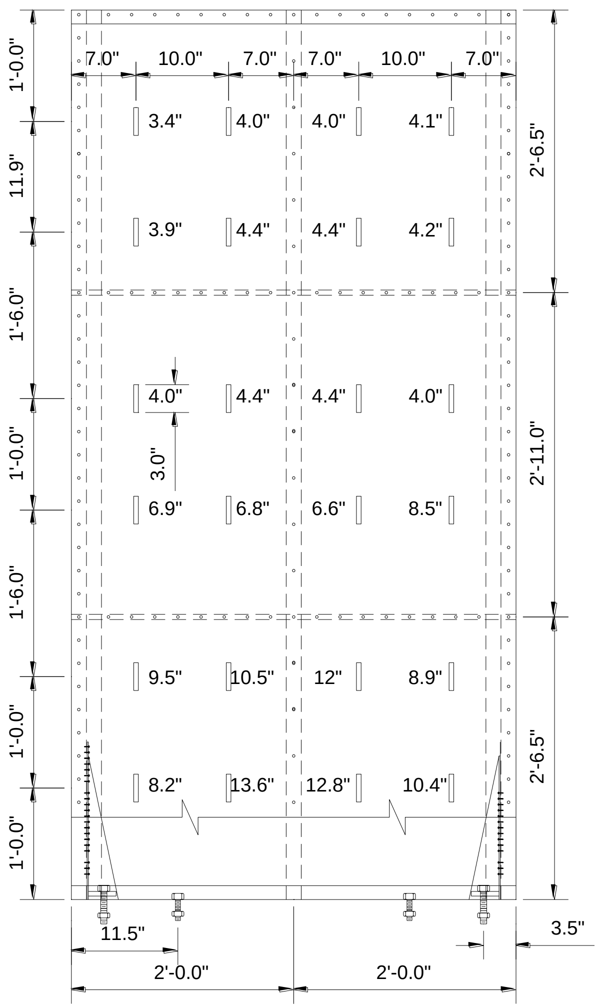
Type 7: 24x3-in vertical slits

No. 10



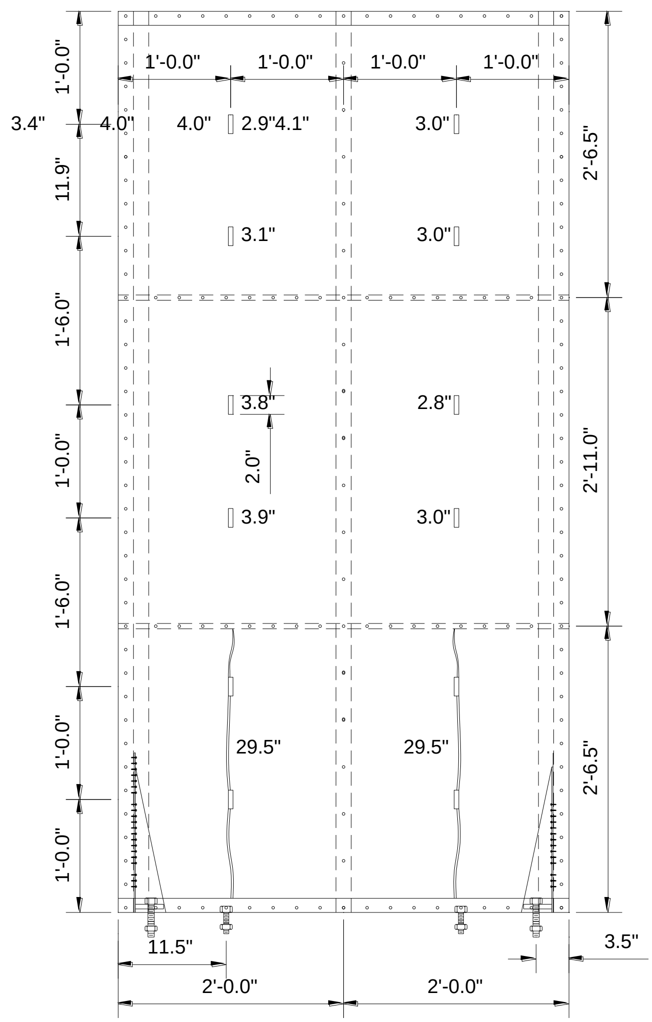
Type 7: 24x3-in vertical slits

No. 13



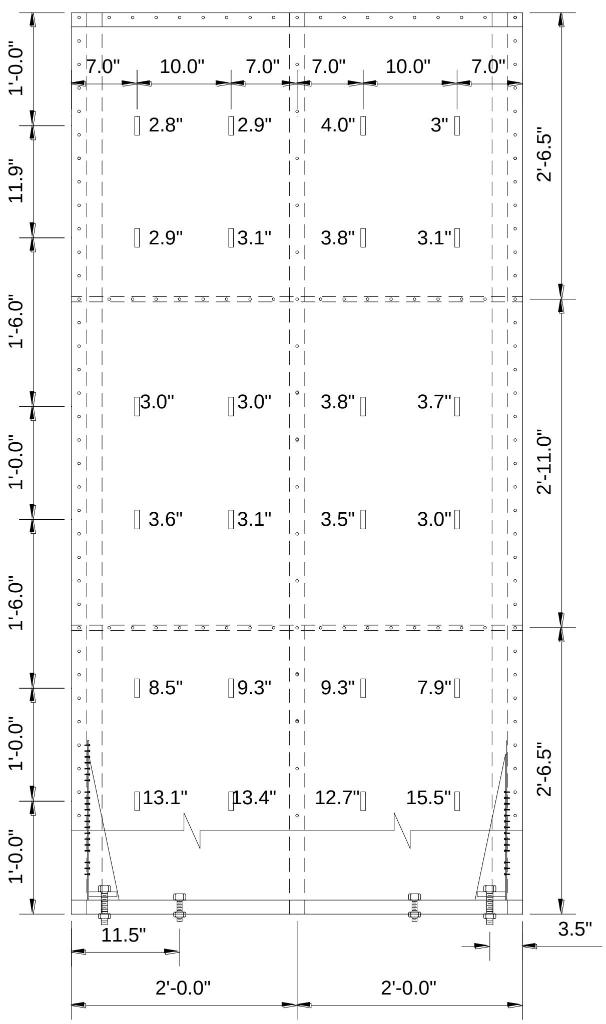
Type 9: 12x2-in vertical slits

No. 14



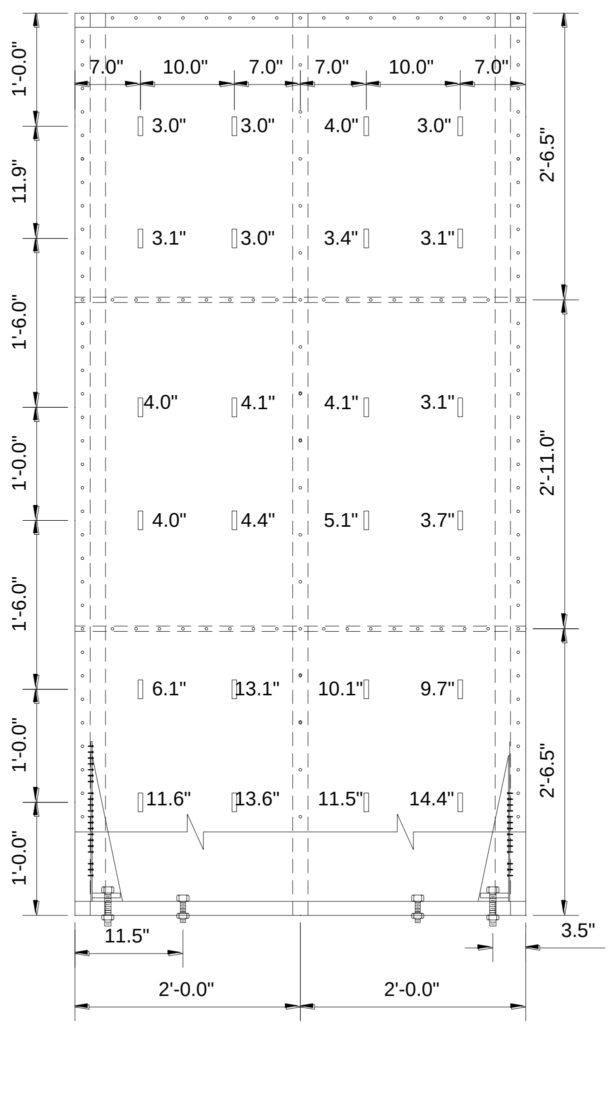
Type 11: 24x2-in vertical slits

No. 15



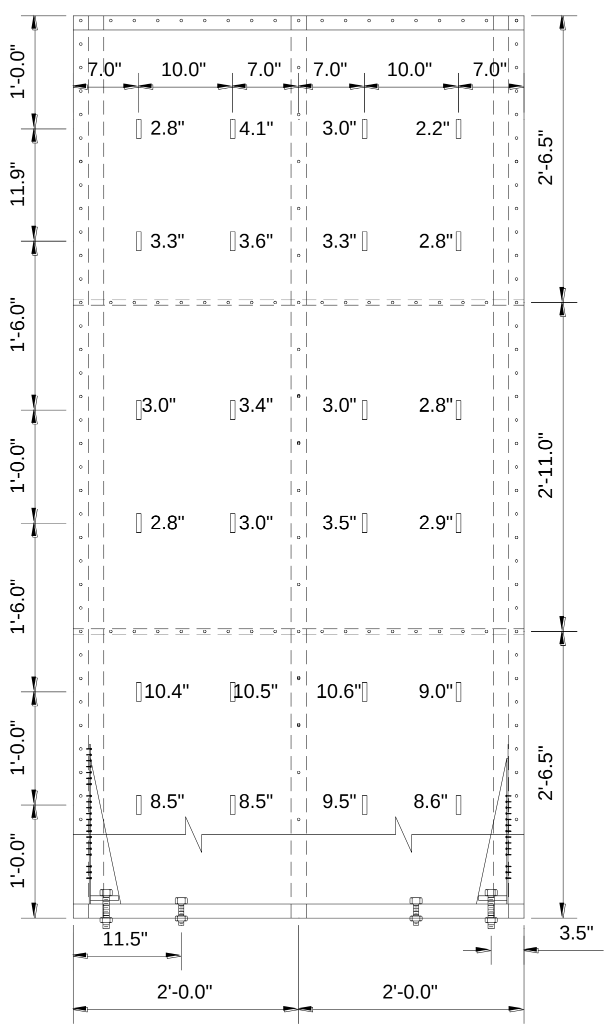
Type 11: 24x2-in vertical slits

No. 17



Type 11: 24x2-in vertical slits

No. 18



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