



Cold-formed steel shear walls in ledger-framed buildings

P. Liu¹, K.D. Peterman², C. Yu³, B.W. Schafer⁴

Abstract

The objective of this paper is to detail recent cyclic testing on Oriented Strand Board (OSB) sheathed cold-formed steel (CFS) shear walls. The shear walls are designed for a two-story ledger-framed building that will undergo full-scale shake table testing as part of the National Science Foundation funded Network for Earthquake Engineering Simulation (NEES) project: CFS-NEES (www.ce.jhu.edu/bschafer/cfsnees). While the overall goal of the project is to enable performance-based seismic design for cold-formed steel framed buildings, the specific goal of this work is to determine how practical building details impact the stability, stiffness, strength, and ductility of cold-formed steel shear walls. Monotonic and cyclic (CUREE protocol) tests are conducted on 4'×9' (1.22m×2.74 m) and 8'×9' (2.44m×2.74 m) shear walls utilizing back-to-back chord studs and OSB sheathing on the exterior. Practical building details studied include the impact of (a) ledger track, which is attached to the top of the interior face of the wall, (b) gypsum, which is attached to the interior face of the wall below the ledger track, (c) locations of panel seams, both horizontal and vertical in the OSB, and (d) the impact of differing stud dimensions for the field studs. In each case the impact on stability (particularly of the chord studs), stiffness (both initial and degraded), strength, and ductility are assessed. The information will be used to assess current design procedures utilizing AISI-S213-07 and to develop improved nonlinear models of the CFS-NEES model building so that full building system performance can be assessed.

1. Introduction

Cold-formed steel has been widely used as a construction material for low and mid-rise buildings due to its light weight and low cost. Common lateral force resisting systems (LFRS) for cold-formed steel construction consist of specifically detailed sheathed and strap braced walls, and other systems (AISI-S213-07). In an effort to improve performance-based seismic design of cold-formed steel systems the CFS-NEES project has recently detailed a two-story archetype building (Madsen et al. 2011). The shear walls in the building utilize back-to-back chord studs, and OSB sheathing. The details of the shear walls deviate modestly from tested configurations in AISI-S213-07 in that (a) a ledger or rim track is attached to the interior face of the walls, (b)

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gypsum board is attached to the interior face of the walls, and (c) OSB panel seams do not always fall at stud or track locations and thus strapping must be used for shear transfer in the walls, and (d) the field studs do not always match the grade or thickness of the chord studs. Given these variations, and a general desire to better understand actual building performance for the CFS-NEES building, it was decided to complete a shear wall test series that specifically investigated the impact of these variations on stability, stiffness, strength, and ductility.

Cold-formed steel shear walls have seen significant study. Notably, based on the work of Serrete (1996, 1997, 2002) the North American Standard for Cold-Formed Steel Framing: Lateral Design (AISI S213-07) provides nominal strength for three different types of sheathing: 15/32 in. “Structural 1” sheathing, 7/16 in. Oriented Strand Board (OSB), and 0.018-0.027 in. steel sheet. Tabled values are based on maximum aspect ratio, fastener spacing at the panel edge, and stud and track thickness. More recently, Rogers (Branston 2004; Branston et al. 2004; Branston, Chen 2004; Chen et al. 2004; Boudreault 2005) has developed a large multi-year shear wall testing program. Among the many aspects studied is the significance of the cyclic loading protocol including comparisons between Sequential Phased Displacement (SPD) and the Consortium of Universities for Research in Earthquake Engineering (CUREE) protocols. In general, the dominant failure modes involved the sheathing-to-steel connection and a combination of fastener pull-through, tear-out, and bearing. A large variety of different sheathing materials and details have been explored. However, the impact of ledger track, panel seams, and grade and thickness of the field studs has not been studied. The test program presented herein is an attempt to broaden the field to include practical details consistent with multi-story CFS buildings.

2. Test Program

2.1 Test setup

Monotonic and cyclic tests were conducted on a 16 ft x 12 ft adaptable testing frame at the University of North Texas. Figure 1 depicts a drawing of the test frame with a 4 ft x 9 ft shear wall specimen.

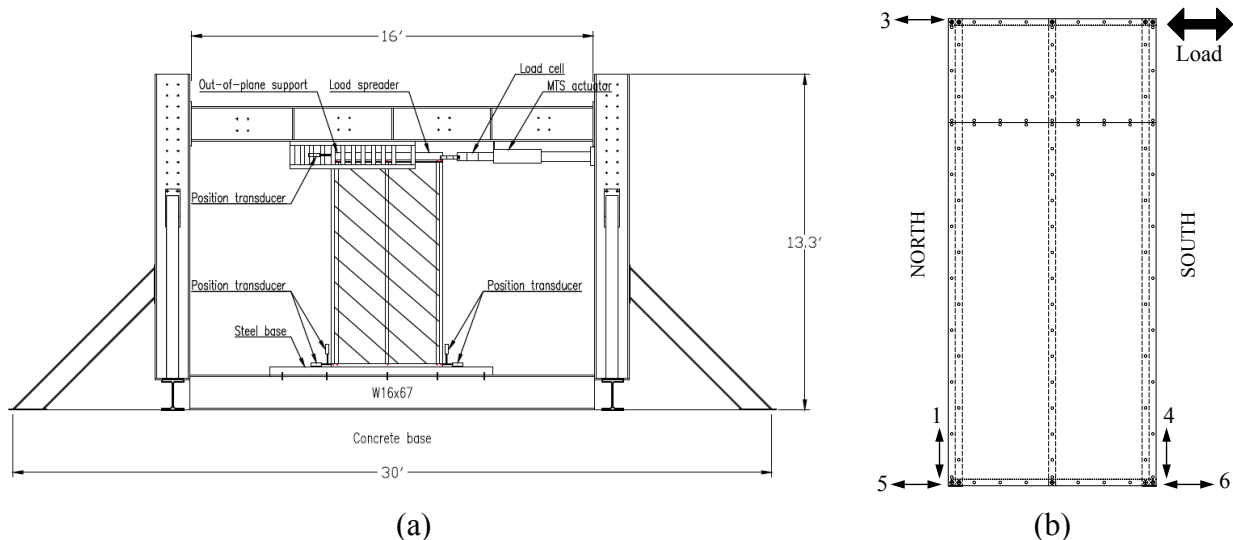


Figure 1: (a) Schematic of testing rig with specimen (b) Sensor plan where numbers correspond to position transducers measuring along the direction indicated by the arrows.

Specimens are bolted to the test frame via the steel base, according to the plan detailed in Figure 2. Both hold downs and anchor bolts restricted the specimen from lateral displacement at the bottom of the test frame. At the top of the specimen, a WT shape acts as a load spreader, transferring horizontal force from the hydraulic actuator to the specimen. To ensure effective load transfer, the WT section was attached to the specimen with No. 10 x 1" hex washer head self-drilling screws along two lines, spaced every 3 inches. To keep the shear wall in plane during the test, a series of steel rollers (labeled 'out-of-plane support' in Figure 1) are arranged in series to restrict the motion of the load spreader as it displaced.

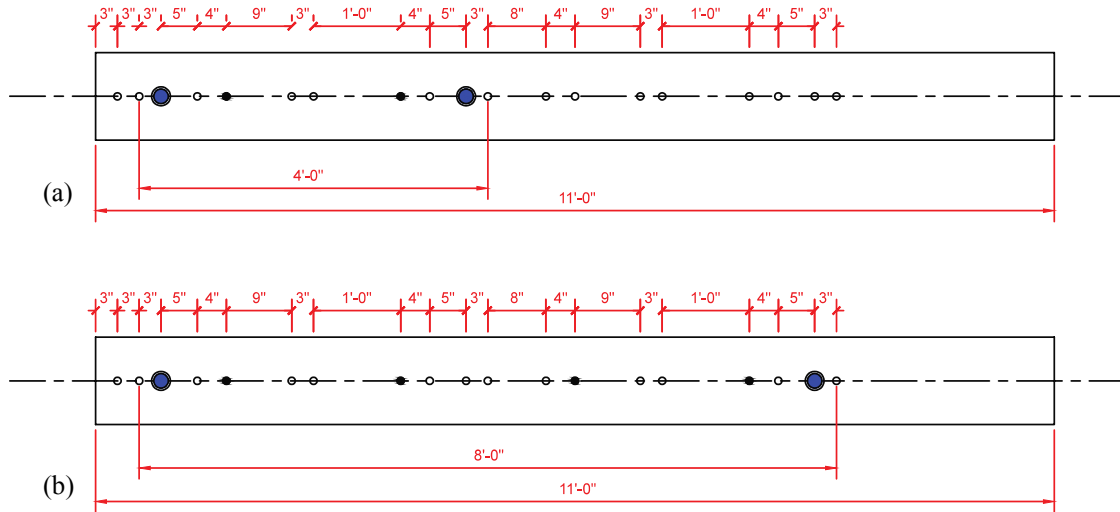


Figure 2: Anchor placement along test frame base (top view) for (a) 4 ft wide walls and (b) 8 ft wide walls. Large (blue) dots indicate hold down position and small black dots represent anchor position.

The test frame is equipped with a 35 kip hydraulic actuator with ± 5 inch stroke. A 20 kip load cell is pinned to the specimen and actuator. Five position transducers are employed to measure deflection of the shear wall in the north, south, and lateral directions (Figure 1b). Each chord stud is outfitted with vertically- and horizontally-oriented position transducers to capture the tension and compression experienced by the wall. One position transducer at the top of the wall records specimen lateral displacement.

2.2 Test Method

Both monotonic and cyclic tests are performed under displacement control. Monotonic tests follow ASTM E564-06, "Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings." This protocol involves a preload of approximately 10% of the estimated ultimate load to be applied and held for five minutes to seat all connections. This preload is then removed and the specimen is loaded to one-third (incremental load) of the estimated ultimate load. Again, the specimen is loaded and unloaded, this time to two-thirds of ultimate load. The loading continues in this manner until ultimate load is attained.

For cyclic loading, the CUREE protocol was employed in accordance with ASTM E2126, "Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings." A constant cyclic frequency of 0.2 Hz was chosen for the cyclic test as well as a reference displacement based on the results from monotonic tests. Figure 3 depicts the CUREE protocol used for this test series.

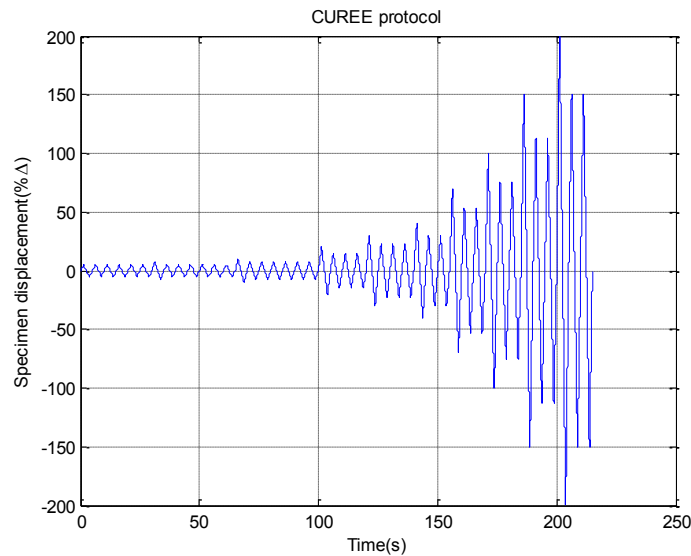


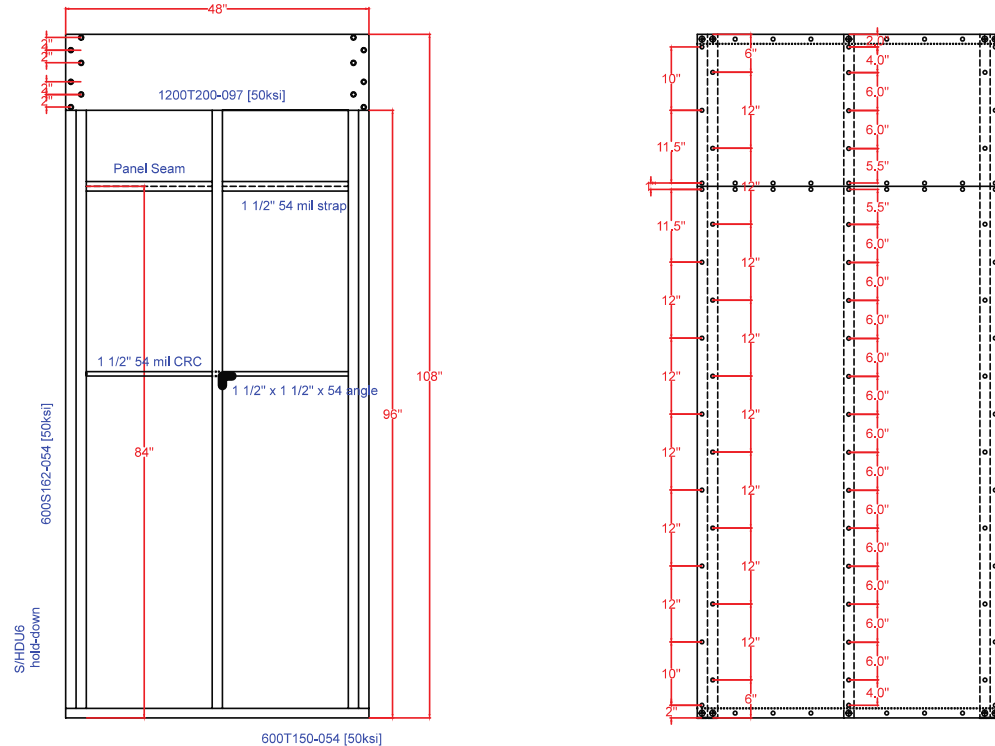
Figure 3: CUREE protocol with cyclic frequency of 0.2 Hz.

2.3 Specimen

The baseline specimen consisted of either 4 ft x 9 ft or 8 ft x 9 ft walls framed with 600S162-54 (50ksi) studs spaced 24 in. o.c. and connected with No. 10 flathead screws to 600T150-54 (50ksi) track (member nomenclature per AISI-S200-07). Studs were spaced at 24 in. o.c. and braced with CRC as detailed in Figure 4a. Chord studs consisted of back-to-back studs assembled with pairs of No. 10 flathead screws spaced every 12 in.. Simpson Strong-Tie S/HDU6 hold downs were attached on the inward face at the bottom of the chord studs. Twelve No. 14 fasteners attached the hold downs to the chord studs.

In most cases (see Table 1) 7/16 in. OSB was attached with No. 8 flat head fasteners (1-15/16 in. long) at 6 in. o.c. to the stud and track. When seams of the OSB do not fall on a stud, taugt 1 1/2 in. wide 54 mil strap was used as shown in Figure 4a. When the rim track/ledger is present (again see Table 1) a 1200T200-097 (50ksi) track is attached with No. 10 fasteners to the top 12 in. of the interior face of the wall. When gypsum board is present 5/8 in. gypsum board is attached with No. 6 fasteners at 6 in. o.c. to the lower 8 ft. (below the ledger).

As previously discussed, connection of the shear wall to the testing rig is accomplished in the top track by dragging in the applied shear through two lines of No. 10 fasteners spaced 3 in. o.c. (labeled load spreader) in Figure 1a. The bottom track is connected to the steel base (Figure 1a) by 5/8 in. diameter A325 bolts at the hold down locations and directly through the track every 2 ft. along the wall (as detailed in Figure 2a).



(a) Back (interior) view (b) Front (exterior) view
Figure 4: Shear wall specimen drawing (details are the same for 8 ft wide shear walls)

2.4 Test Matrix

The CFS-NEES building dictated specimen construction and design. Most construction variables were studied, including the effect of the ledger (rim track), interior gypsum sheathing (B. Sheathing in Table 1), and panel seam location for both 4 ft and 8 ft walls. Stiffness, strength, and ductility were investigated. The experimental test matrix with selected results is produced in Table 1. It is worth noting that for specimens with a vertical seam not located along a field stud, an additional field stud is added at the seam location. In addition, test 15 employed 33 mil studs only in the field (chord studs remained back-to-back 600S162-54 (50ksi)).

Table 1: Test matrix and selected results

Test quantity	Wall Size	Load Type	F. Sheathing	B. Sheathing	Stud	Ledger	H. Seam	V. Seam	Peak Load	Peak Disp.
unit	ftxft	mono/cyclic	OSB	Gypsum	600S162-xx	1200T200-97	ft	ft	P_{ave}	Δ_{ave}
			✓/–	✓/–	1/1000 in.	✓/–			pLf	in
1c	4x9	Monotonic	✓	–	54	✓	8'up	–	1225	2.96
2	4x9	Cyclic	✓	–	54	✓	8'up	–	1102	2.82
3	4x9	Cyclic	✓	✓	54	✓	8'up	–	1111	2.67
4	4x9	Cyclic	✓	–	54	–	8'up	–	1004	2.40
5	4x9	Cyclic	✓	–	54	✓	7'up	–	987	2.39
6	4x9	Cyclic	✓	–	54	–	7'up	–	1031	2.24
7*	4x9	Cyclic	✓	–	54	–	8'up	1'over	897	2.23
8*	4x9	Cyclic	✓	–	54	–	8'up	2'over	982	3.33
9	4x9	Cyclic	✓	–	54	–	8'up	2'over	906	3.56
10	4x9	Cyclic	✓	–	54	–	4.5'up	–	950	2.94
11c	8x9	Monotonic	✓	–	54	✓	8'up	–	593	1.00
11r	8x9	Monotonic	✓	–	54	✓	8'up	–	1089	2.42
12	8x9	Cyclic	✓	–	54	✓	8'up	–	1156	1.96
13	8x9	Cyclic	✓	✓	54	✓	8'up	–	1232	1.91
14	8x9	Cyclic	✓	–	54	–	8'up	–	1023	1.94
15	8x9	Cyclic	✓	–	33	–	8'up	–	861	1.64
16	8x9	Cyclic	–	✓	54	✓	8'up	–	231	1.47

Notes: CUREE protocol employed for cyclic testing, *additional field stud 1' over from side

3. Test Results

Test results are summarized in Table 2 and Figures 5-7, and discussed in the following.

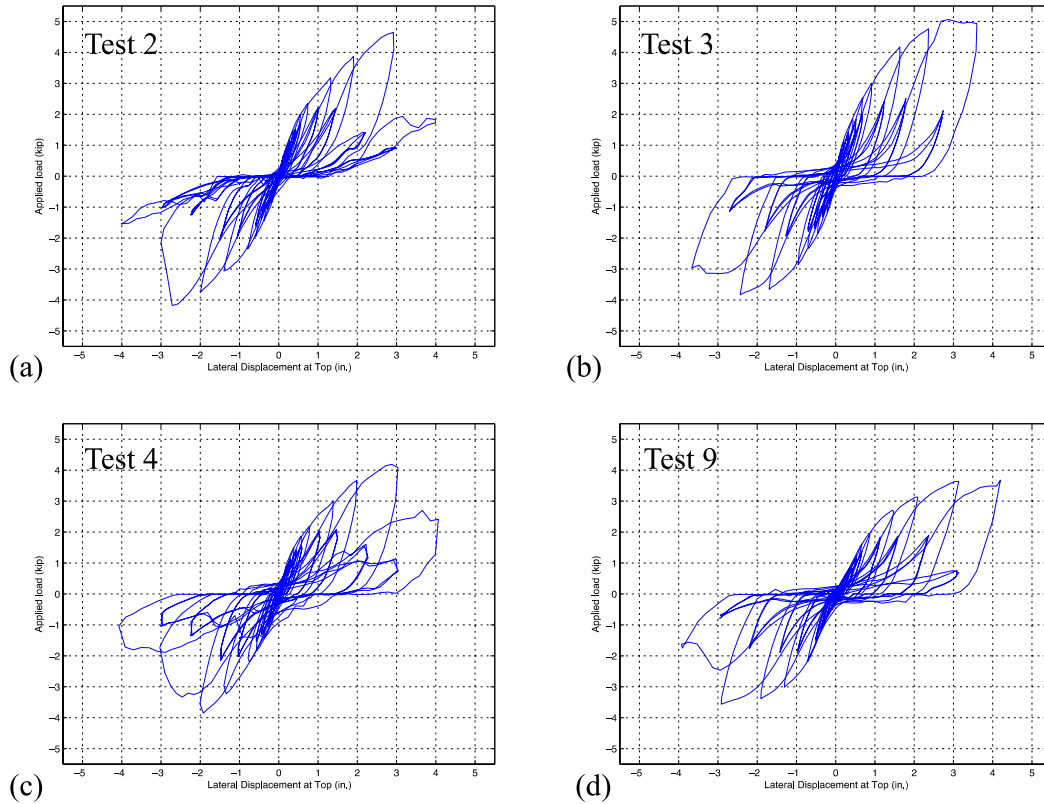


Figure 5: Hysteretic response of (a) tests 2, OSB sheathed with ledger, (b) tests 3, OSB sheathed with ledger and gypsum board, (c) tests 4, OSB sheathed without ledger and without gypsum board, and (d) tests 9, OSB sheathed without ledger and without gypsum board but additional vertical seam 2' over from side

Table 2: Results summary

Test quantity	Peak Load		Lateral Deflection at Peak		Avg. Load ¹	Avg. Disp ²	Failure Mode ³
	P+	P-	Δ +	Δ -	P_{ave}	Δ_{ave}	
unit	plf	plf	in.	in.	plf	in.	-
1c	1225	-	2.96	-	1225	2.96	PT
2	1160	1044	2.92	2.71	1102	2.82	PT
3	1265	958	2.87	2.44	1111	2.67	PT
4	2092	963	2.88	1.93	1527	2.40	PT
5	1023	950	2.83	1.96	987	2.39	PT
6	1232	830	2.78	1.69	1031	2.24	PT
7*	876	918	2.55	1.91	897	2.23	PT
8*	1036	929	3.66	3.00	982	3.33	PT
9	921	890	4.20	2.92	906	3.56	PT
10	951	950	2.91	2.98	950	2.95	PT
11c	593	-	1.00	-	593	1.00	PT+B
11r	1089	-	2.42	-	1089	2.42	PT+B
12	1256	1055	2.27	1.66	1156	1.96	PT+B
13	1327	1138	2.20	1.62	1232	1.91	PT+B
14	1056	990	2.22	1.66	1023	1.94	PT+B
15	883	839	1.62	1.67	861	1.64	PT
16	259	202	1.22	1.73	231	1.47	PT

¹Average of P+ and P-, ²Average of Δ + and Δ -, ³PT = fastener pull-through and B = fastener bearing



Figure 6: (a) Back of test 4, demonstrating fastener pull-through (PT) along the horizontal seam (b) front of test 2 depicting the resulting separation of sheathing from seam as a result of PT (c) separation of OSB from bottom wall track in test 13 (d) Fastener PT and tear-out (TO) against sheathing along field stud in test 11 (e) north chord stud in test 13, demonstrating fastener PT in the OSB and a combination of PT and bearing (B) in the gypsum.

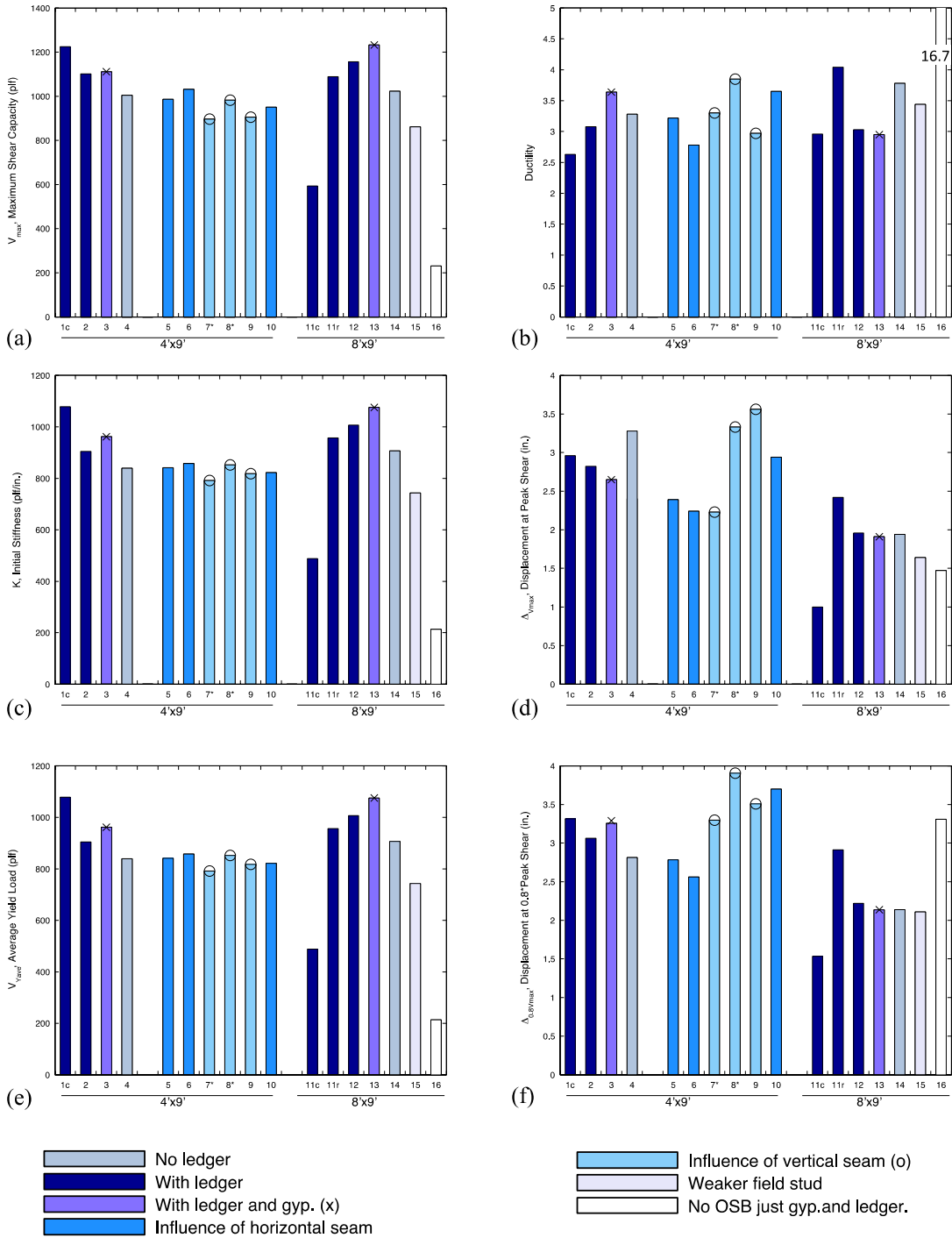


Figure 7: Test results and bar plot legend (a) maximum shear capacity (b) ductility (c) initial stiffness (d) displacement at peak shear (e) average yield load and (e) displacement at 0.8*peak shear. Note: X and O symbols on plots facilitate viewing in black and white.

The hysteretic force-deformation response of typical tests as they undergo the CUREE loading protocol (Figure 3) are provided in Figure 5. Key results from the backbone curve are recorded for each test and summarized in Table 2 and Figure 7. In addition, the observed failure mode for each test is recorded in Table 2. Note, bearing failures (B) in Table 2 also include edge tear-out failures (precipitated by bearing prior to tear out). The observed failure modes and final condition of the test specimens is further depicted in Figure 6. Fastener failures were primarily localized to the bottom track and chord studs, furthest from the load application point. The horizontal seams also experienced fastener pull-through and buckling of the steel strap. Tear-out occurred in locations, often the vertical seam, in which fasteners were closer to the sheathing edge than was typical (i.e. due to the need to accommodate two rows of fasteners in one flange).

4. Hysteretic Characterization: EEEP model

An important objective of the testing is the characterization of the nonlinear performance of the tested shear walls for use in modeling of the CFS-NEES building. Given the simplicity and popularity of Equivalent Energy Elastic Plastic (EEEP) models it was determined that this characterization would be performed first. More detailed characterization using advanced models including pinching and strength degradation will be performed subsequently. EEEP models were first proposed by Park (1989) and were adopted by Kawai et al (1997) to calculate ductility analyses for steel-framed walls. The models are based upon the idea that the energy dissipated by a shear wall under monotonic or cyclic loading is equivalent to the energy represented by a bilinear curve, as depicted in Figure 8.

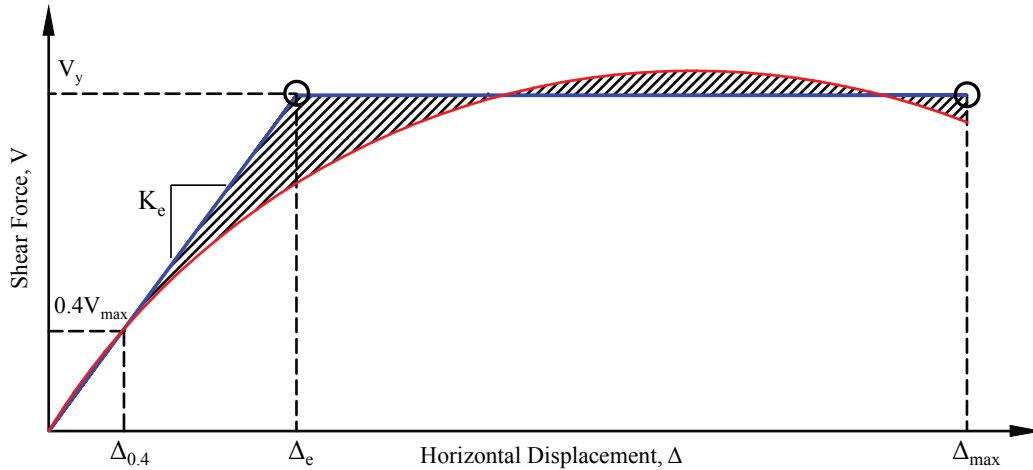


Figure 8: Equivalent Energy Elastic-Plastic (EEEP) model. Red line (curvilinear) represents load-displacement backbone curve while blue line (linear) represents EEEP model.

To construct this curve, the user must first determine 40% of the peak load, or $0.4V_{\max}$, and the corresponding displacement $\Delta_{0.4}$. These values define the elastic range of the load-displacement curve from which the elastic stiffness is calculated (Eq. 1). (Note, Eq. 1 is used for “stiffness” throughout this paper, i.e. Figure 7)

$$K_e = \frac{0.4V_{\max}}{\Delta_{0.4}} \quad (1)$$

The peak displacement Δ_{\max} is determined from the displacement at 80% post-peak strength (per ASTM E2126). Finally, the yield strength V_y is found by equating the energy under the actual backbone and the EEEP curve. Figure 9 provides an example, as constructed for test 4.

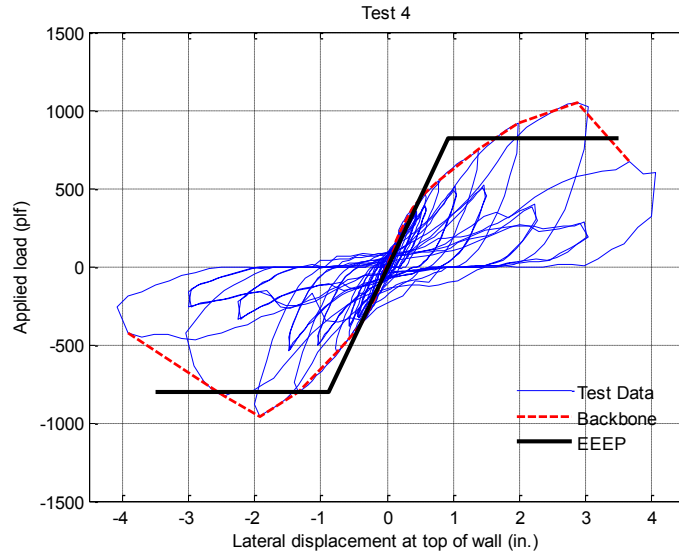


Figure 9 EEEP and Backbone curves for Test 4

From this characterization, the ductility ratio μ may be defined as the ratio of maximum displacement to maximum elastic displacement:

$$\mu = \frac{\Delta_{\max}}{\Delta_e} \quad (2)$$

Since Δ_{\max} is not the maximum deflection (but rather only at the 80% post-peak level) and Δ_e is artificially generated, the ductility ratio μ is only an approximate measure of ductility. In addition EEEP models must be used with some care if implemented in nonlinear analysis since the amount of pinching is not characterized in these models (note the small amount of energy absorbed in the 2nd and 4th quadrant of Figures 5 and 9.) EEEP results are provided in Figure 7.

5. Discussion and Analysis

5.1 Strength

Per AISI-S213-07, for shear walls with 54 mil studs utilizing 7/16 in. OSB sheathing attached at 6 in. o.c. with No. 8 fasteners on one side, the nominal shear strength is 940 plf. This compares favorably with observed strength (Figure 7a and Tables 1 and 2) with the exception of tests 7, 9 and 15. Test 7 and 9 (Table 1) include an added vertical seam as the typically 4 ft x 8 ft OSB sheet was split into two 2 ft x 8 ft. OSB sheets. Although theoretically such a seam near the center of the specimen should have little influence the strength is modestly reduced. Narrow sheets should be avoided when it is practical to do so.

Test 15 replaces the 54 mil field studs with 33 mil field studs (this is consistent with the lighter demands on field studs and gravity framing in the upper story of the CFS-NEES building). This

too results in a modest reduction in strength below 54 mil specified values. If, conservatively, 33 mil values are selected from AISI-S213-07 then the nominal values is only 700 plf, compared with the tested 861 plf which is well in excess of the nominal. Thus, as long as the designer accounts for the potential reduction due to the field studs strength is not compromised; though “overstrength” in such cases must be considered as the expected strength is between the 33 mil and 54 mil values.

5.2 Influence of Ledger track

The ledger transfers load from joists to the shear wall and is a critical component of the lateral force resisting system of a CFS building. To determine the contribution of the ledger to the frame, stiffness tests were conducted on unsheathed frames prior to monotonic testing. For both wall sizes, the ledger almost doubled the bare wall stiffness: in 4 ft. wide walls, stiffness increased from 51 lbs/in to 106 lbs/in and in 8 ft. walls, from 94 lbs/in to 167 lbs/in. After this initial characterization tests were performed with and without ledger track.

Full response is provided for a 4 ft x 9 ft wall in test 4 without a ledger (Figure 5c) and in test 2 with the ledger in place (Figure 5a). The hysteretic loops appear slightly greater (fatter, dissipating more energy) for test 4, without the ledger, but this is somewhat deceiving as in reality the only significant difference was the ability to incur one more stable cycle in test 4 than test 2. In observing the test in real time (or video) the primary difference one notices is that the ledger completely blocks off the deformations of the upper 1 ft of the wall, essentially turning the 4 ft x 9 ft wall into a 4 ft x 8 ft wall. In general, as shown in Figure 7, the addition of the ledger modestly increases strength, and stiffness, and decreases peak deflection and deflection at 80% post-peak shear strength. The simple EEEP ductility measure, μ , is ~ 3 with the ledger in place and ~ 3.3 without the ledger. No failure states in the ledger were observed in testing. In actual buildings the continuous nature of the ledger and its interaction with the floor systems in the transverse direction and the gravity framing in the lateral direction would lead to more complicated behavior.

5.3 Gypsum board

Gypsum board is typically used for fire resistance in interior walls, and is significantly less stiff and more brittle than OSB. The shear walls as designed in the CFS-NEES building are sheathed with exterior OSB and interior gypsum. Full hysteretic response of a specimen with gypsum board is provided in Figure 5b for test 3. Compared with the case without gypsum board (Figure 5a, test 2) the gypsum board provides some relief from the severe pinching and modest energy dissipation in the degraded cycles. Overall, Figure 7, the addition of the gypsum board modestly increases strength and initial stiffness, but post-peak performance is driven by the fastener failure modes in the stronger OSB sheathing and the end result in terms of EEEP ductility is quite similar to the case without gypsum board. The case of gypsum board overlaying the OSB on the exterior is not studied here. Also note, for comparison, test 16 provides results for gypsum only sheathing (no OSB).

5.4 Panel Seams

Since OSB and gypsum board are typically available only in 4 ft x 8 ft panels, multiple panels or parts of panels are necessary to accommodate walls with other dimensions. In real construction (such as the CFS-NEES building) panel seams are common; however, in the testing utilized for

the nominal strength tables in AISI-S213-07 panel seams are not considered as tests have been on either 2 ft, 4 ft, or 8 ft wide walls. For horizontal seams the standard detail, as depicted in Figure 4a is to attach a taugt strap of the same thickness as the studs across the studs and fasten the sheathing to the strap. This detail is intended to allow the necessary shear force transfer. For vertical seams, if the panel seam does not fall at a stud line, then an additional field stud is added to accommodate the seam. To investigate the influence of these seams various vertical and horizontal seams, as summarized in Table 1, were introduced into the standard width walls.

Overall the presence of additional panel seams in a wall has only a modest impact on the initial stiffness, peak strength, and even the EEEP ductility. Indicating typical details are generally adequate; however, vertical panel seams, at least with narrow panels, provide the greatest difference from baseline. For example, tests 8 and 9 exhibit significant increases from baseline in the deflection at peak strength and the deflection at the 80% post-peak strength, as shown in Figure 7b and d. Both of these 4 ft x 9 ft walls have a vertical seam at 2 ft over – thus utilizing two 2 ft x 8 ft panels side by side and one 4 ft x 1 ft panel at the top. It is known that the perimeter fasteners dissipate energy and are by far the most important, but near peak and in the post-peak regime the two panels appear to both be contributing to the flexibility and rotating about their own panel centers, instead of the wall center. However, as discussed in Section 4.1 the presence of the vertical seam also results in modestly reduced strength indicating narrow vertical panels should be avoided when it is practical to do so.

The study on horizontal seams is tied to the presence (or lack) of the ledger. When the ledger and horizontal panel seam are aligned (tests 2 and 4) the strength is greatest with the ledger in place, but when the ledger and panel seams are at different heights (ledger at 8 ft panel seam at 7 ft as in test 5 or only a panel seam at 7 ft as in test 6) the strength is greater without the ledger. The stiff ledger is a detriment to the performance of the nearby more flexible seam, when they are not aligned, and the result is a modestly reduced capacity; 10% in this case. Initial stiffness, peak deflections, and EEEP ductility all exhibit similar small variations, but the response can be altered by the panel seam-ledger interaction. For example, Test 2 with the panel seam and ledger aligned failed first along the seam line, while test 5 with the panel seam at 7 ft and ledger at 8 ft failed at the bottom. However, both specimens failed via fastener pull-through. If the horizontal seam is at mid-height (test 10) the reduction is similar to the horizontal seam at 7 ft. The most beneficial location for the horizontal seam is when it is aligned with the ledger at 8 ft; essentially creating a fully blocked condition at that seam height.

5.5 Chord stud and field stud

As discussed in Section 4.1, test 14 and 15 demonstrate that that the grade and thickness of field studs have an impact on shear strength, ductility, and stiffness. The shear strength of test 14 (54mil steel, 50ksi field studs) is 16% higher than that of test 15 (33mil steel, 33ksi field studs) even though the back-to-back chord studs, track, sheathing, and ledger are identical. Thus, all of the perimeter fasteners are under identical conditions; nonetheless, the contribution from the fasteners in the field is significant enough to reduce the observed strength.

5.5 Stability

Stability limit states for the chord studs were not observed (e.g., see Figure 6e). In the initial conceptualization of this effort consideration was given to using undersized chords to

specifically explore local, distortional, and global buckling in concert with the typical fastener limit states. It was decided to instead align the testing with more practical building details and to explore cyclic behavior of the member limit states in a separate companion project. This companion project is now underway and being lead by Prof. Moen, details are available at www.moen.cee.vt.edu/research/research_energydiss.html for this work.

5.6 Future work

A complete test report detailing all of the work performed in this project is under preparation and will be available in Spring 2012 as part of the CFS-NEES project at www.ce.jhu.edu/cfsnees. In addition, all source data and drawings for this project are part of the curated data available at www.nees.org under the CFS-NEES project: NSF-CMMI-1041578: NEESR-CR: Enabling Performance-Based Seismic Design of Multi-Story Cold-Formed Steel Structures. Work is currently underway to characterize the tested data with a nonlinear hysteretic model that includes pinching and strength degradation and then implement this model in a nonlinear time history analysis of the CFS-NEES building.

6. Conclusions

A series of cyclic (CUREE protocol) tests were conducted on cold-formed steel shear walls with the specific objective of understanding the influence of details that are employed in practical construction that deviate from conventionally tested walls. The baseline wall consisted of back-to-back 54 mil cold-formed steel chord studs with 7/16 in. OSB sheathing. Practical details examined included studies of the influence of: ledger (rim track), interior gypsum board, horizontal and vertical panel seams within the wall, and the use of lower thickness and grade studs in the field of the wall. Tested capacities are sensitive to practical details: peak strength varies by 30% across the tested details. If one rejects the use of lower thickness field studs and does not account for gypsum board then the spread reduces to 22%, with the ledger track (beneficial) and vertical seams (detrimental) providing the most important source for the remaining spread. Tested capacities exceed or are within expected scatter (5%) of the AISI-S213-07 specified shear strength. Integration of the results into the larger CFS-NEES project focused on seismic performance-based design of cold-formed steel structures is underway.

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