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Cyclic Performance of Steel Sheet Connections for CFS framed Steel Sheet Sheathed Shear Walls

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Abstract

The main objective of this research is to study fastener-level force-deformation response appropriate for standard cold-formed steel (CFS) framed steel sheet sheathed shear walls under cyclic loads. Recently completed CFS-framed shear wall tests employing thin steel sheets screw-fastened to thicker CFS-framing have recorded higher capacity and ductility for the CFS-framed steel sheet sheathed shear walls. For the seismic performance of these shear walls, the cyclic nonlinear response of the fastener connection is especially important and should incorporate the impact of shear buckling of the steel sheet on the strength and ductility of the connection. Minimal cyclic fastener-level shear test data exists, especially for combinations of screw fastened thin steel sheet and thick framing steel. To address this, a unique lap shear test following AISI S905 was designed to elucidate and characterize the cyclic fastener behavior. The specimens were loaded with an asymmetric cyclic loading protocol which intentionally buckles the sheet in the compression direction, and progressively increases in the tension direction. A total of 93 tests demonstrating a wide range of framing thickness, sheet thickness, fastener size, and loading types were conducted. Key experimental statistics, including the characterization with a multi-linear backbone curve, are provided. Fastener connection strength is sensitive to whether the thin steel sheet ply is buckling away from or towards the fastener head in some test series. AISI S100-16 screw shear strength provisions performance is evaluated. The work is aimed at providing critical missing information for CFS-framed steel sheet sheathed shear walls for use in both simulation and design.

1. Introduction

The need for low cost, multi-hazard resilient, sustainable, lightweight building structures can be potentially fulfilled by cold-formed steel (CFS) framed mid-rise structures. One of the primary structural components providing lateral resistance in CFS-framed structures are CFS-framed steel sheet sheathed shear walls [1]. A CFS-framed steel sheet sheathed shear wall consists of single- or double-sided steel sheet sheathing, CFS studs, CFS tracks, blocking members, hold-downs or tie rods, and fasteners connecting the framing and steel sheet sheathing. For example, a wall line consisting of two steel sheet sheathed shear walls and an interior gravity wall was tested on shake tables as shown in Figure 1 [2][3]. A series of shear buckling waves in the steel sheet sheathing as shown in Figure 1a and b was easily observed as the dominant feature. But, peak strength and post-peak behavior are controlled largely by fastener failures as shown in Figure 1c. The cyclic nonlinear response of the fastener connection is particularly significant for the overall shear wall response, and the impact of the steel sheet shear buckling on the connection behavior needs to be considered.

(a) Test set up (b) Sheet buckling (c) Fastener failure

Figure 1: CFS-framed steel sheet sheathed shear wall line test by Singh et. al. [2][3]

A recently compiled shear wall database [4] summarizes the available CFS-framed steel sheet sheathed shear wall tests [5-20]. As load bearing CFS framing has seen increased use

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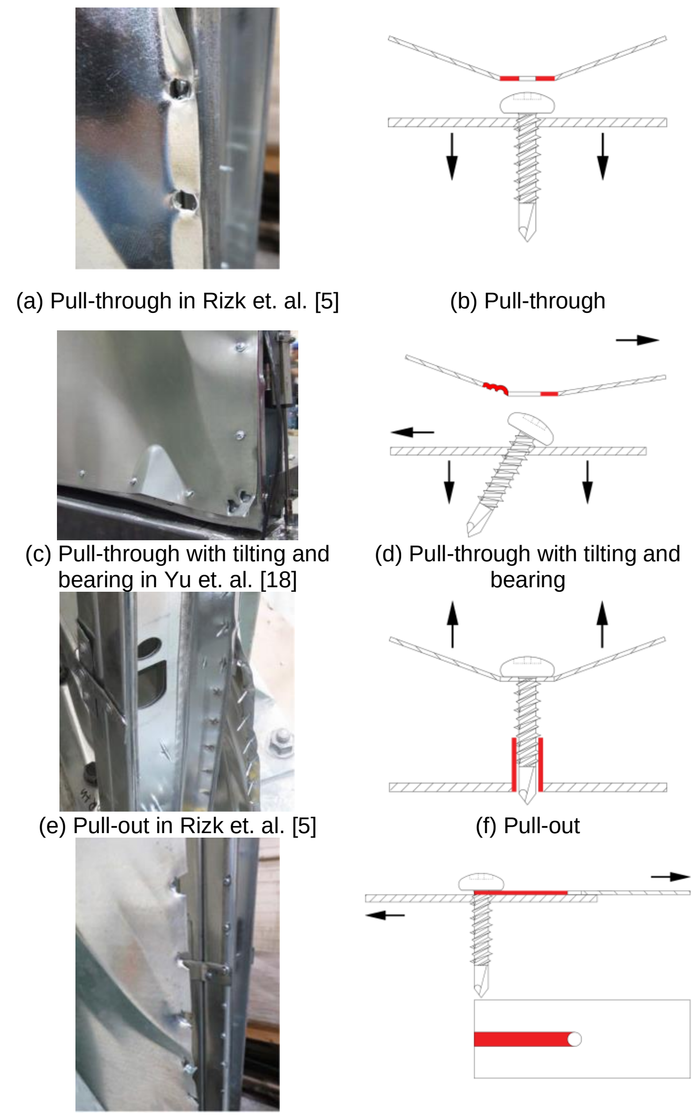
in multi-story buildings, the demands on these shear walls both in terms of gravity load as well as overturning and overstrength requirements have led to the adoption of thicker framing members (e.g., increasing from 1.37 mm to 2.46 mm). This has motivated experimental efforts towards these members, which demonstrate higher shear wall capacity and ductility and are thus desirable for additional reasons. However, the available thickness combinations of existing cyclic fastener lap shear (or similar) connection tests [21-24] are not consistent with steel sheet-to-framing connections, especially thin steel sheet attached to thick framing. Study of the cyclic performance of thin steel sheet attached to thick steel framing is needed.

The strength of a limited number of CFS-framed steel sheet sheathed shear wall configurations are provided through tabulated response in AISI S400-15 [25]. Precise knowledge of the connection-level behavior provided in AISI S100-16 [26] is necessary for determining the shear wall strength of unique cases using the “effective strip method” or the principles of mechanics and supplemental data in AISI S400-15 [25].

The objectives of this experimental research on steel-to-steel cyclic connection response in shear is to (i) provide results appropriate for screw fastened steel sheet shear walls incorporating the impact of steel sheet shear buckling on the connection, (ii) establish baseline behavior and characterize the connection performance, and (iii) investigate the applicability of current code provisions for this configuration. A unique cyclic lap shear testing configuration, following AISI S905 [27], demonstrating one thin steel sheet ply and one thick framing ply connected by one single fastener with proper sensors is designed and built. The cyclic loading protocol investigated is asymmetric with a small displacement applied in the direction which buckles the thin steel sheet followed by progressively larger displacements in the opposite direction. The test data are characterized to support the design and simulation for CFS-framed steel sheet sheathed shear walls. This research provides the technical details and processing of 93 conducted tests on steel sheet connections for CFS-framed screw-fastened steel sheet shear wall configurations.

2. Failure modes

Connection failure mode is as important as connection strength in design. The primary failure mode is bearing in the steel sheet and for some thickness ranges tilting as the leading demand on the fasteners is shear. However, disengagement of the stud and sheet is the ultimate failure mode, which can be divided into four distinct modes: pull-through, pull-through with tilting and bearing, and pull-out, as illustrated in Figure 2, (which are primarily associated with tensile demands on the connection) shear rupture (or edge tear out, which stems from high shear demand). Pull-through, as shown in Figure 2a and 2b, is not specifically defined in AISI S100-16 [26], but is recognized in the related technical literature [28] and is close in behavior to pull-over. Pull-through develops when the stud or track flange deforms and pulls the fastener with it resulting in the fastener head tearing through the sheet. If the stud or track deformation involves significant twisting then the failure mode is pull-though with tilting and bearing as shown in Figure 2c and d. Finally, if instead of the sheet tearing, the threads pull-out of the stud or track, then pull-out failure mode occurs, as shown in Figure 2e and f. Shear rupture failures occur due to minimal edge distance limiting the bearing capacity, as shown in Figure 2g and h.



(a) Pull-through in Rizk et. al. [5]
(b) Pull-through
(c) Pull-through with tilting and bearing in Yu et. al. [18]
(d) Pull-through with tilting and bearing
(e) Pull-out in Rizk et. al. [5]
(f) Pull-out
(g) Shear Rupture in Rizk et. al. [5]
(h) Shear Rupture

Figure 2: Fastener failure modes observed in recent shear wall tests and idealized failure mechanism

3. Experimental program

3.1 Test matrix

The fastener and sheet ply thicknesses selected in this testing program are summarized in Table 1. The test specimen consists of one thick steel framing ply and one thin steel sheet ply (in contact with the fastener head) fastened by a single screw. This test matrix is designed considering both the existing fastener and shear wall test data and covering a wide range of steel sheet thickness, fastener size, and various loading types. Existing data is highlighted in Table 1 with shading. Only one relevant configuration has both connection and steel sheet shear wall data.

In a standard test series, there are seven tests including one monotonic test, three asymmetric cyclic tests with thin sheet buckling away from the fastener head, and three asymmetric cyclic tests with thin sheet buckling towards the fastener head. The necessity to force the thin sheet buckling direction was demonstrated in shakedown tests [29] and is discussed further in the results. Each test series was given a unique name according to an established nomenclature: e.g. the “54-8-30” series, stands for a 1.37 mm (54 mil) thick framing steel ply fastened with a single #8 self-drilling pan head screw (#10 and #12 screws are also adopted) to an 0.76 mm (30 mil) thin steel sheet ply. In the “97-10-30” series an additional 3 tension only cyclic tests were completed – for a total of 10 tests. To verify results additional repetitions were conducted in some cases resulting in a total of 93 tests completed vs. 80 tests originally proposed in the test matrix. Each test conducted was assigned a unique test number and all individual results are available in a comprehensive test report [29].

Table 1: Proposed test matrix and existing related data

Framing Steel (mm / mil)	Steel Sheet (mm / mil)								
	0.33 / 13			0.48 / 19			0.76 / 30		
	#8	#10	#12	#8	#10	#12	#8	#10	#12
1.37 / 54	7	7		7	7		7	7	
1.73 / 68									
2.46 / 97		7			7		7	10	7

Range of existing fastener testing [21-24]
Range of existing steel sheet shear wall testing references [5-20]
Range of existing fastener and matching shear wall testing

3.2 Test specimens

The steel sheet buckling behavior and resulting pull-through fastener failure mode is not currently captured by the standard lap-joint shear test specimen configuration per AISI S905 [27]; therefore, the test specimen must be specially designed based on the failure modes observed in shear wall tests as shown in Figure 2 and previously detailed. Due to the shear buckling of the steel sheet the perimeter fasteners not only resist shear demand but also must resists out-of-plane forces that work on the fastener head due to extensive buckling of the thin steel sheet under cyclic loading. The force caused by the sheet buckling itself is not a large demand, but can lead to premature pull-through behavior as opposed to pure bearing in a connection. This “shear-tension” interaction of interest in this testing program is identical for the fastener behavior and the overall shear wall response under seismic events.

Dimensions and loading protocol of a standard lap shear joint test are modified in this testing program. As presented in Figure 3, the upper and bottom shaded parts with 50.8 mm (2 in.) length are clamping areas for the grips, 50.8 mm x 50.8 mm (2 in. x 2 in.) spacers are placed inside the grips to avoid eccentric loading. The thin steel sheet ply length is set equal to the half-wave length of steel sheet sheathing close to the shear wall framing boundary. After reviewing typical shear wall tests (W2 test in [5], W21 test in [6], M11 test in [18]), a simple estimate for the shear buckling half-wave length in the perimeter is approximately 102 mm (4 in.). This distance then corresponds to the length between the top grip and fastener head of the specimen. The edge distance for the thick framing ply is chosen to be 20.6 mm (0.81 in.) which corresponds to half of the flange width of a typical chord stud section (362S162-97), and the edge distance for the thin steel sheet ply is set to 19.0 mm (¾ in.) which meets the 1.5d minimum edge distance requirement (J4.2 in AISI S100-16 [26]) for all tests. The length between the fastener and bottom grip is minimized to 25.4 mm (1 in.) to minimize tilting of the steel ply in a standard lap-joint shear test per AISI S905 [27].

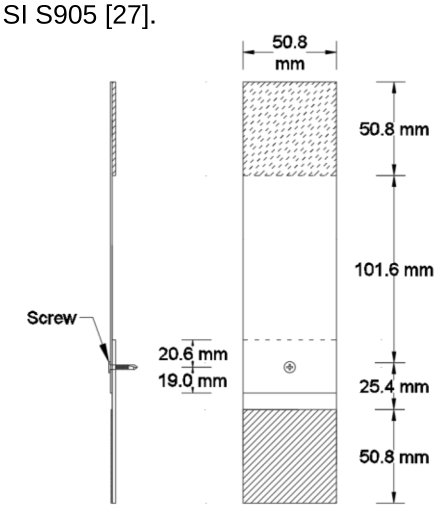
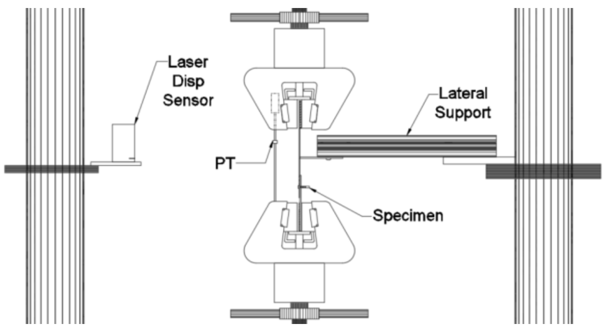


Figure 3: Typical test specimen

3.3 Test setup

The test rig is shown in Figure 4a and 4b. All the tests were conducted in an MTS servohydraulic test system. A position transducer (PT) and a load cell are adopted to acquire deformation and force data. In addition, a laser displacement sensor is utilized to monitor the out-of-plane thin steel sheet buckling deformation. A mechanical lateral support was installed at either left or right side to guide the thin sheet to buckle away or towards the fastener head. Figure 4c depicts a typical test specimen. One-sided cyclic lap shear testing is adequate for capturing the shear behavior since previous cyclic testing demonstrates that the response is symmetric [21][22][23]. Further, the buckling of the thin steel sheet creates a shear-tension interaction on the connection consistent with fasteners in steel sheet shear walls, and maximizes the opportunity that the fastener tilts and slips through the thin steel sheet.



(a) Test setup

(b) Test rig

(c) Specimen in the test rig

Figure 4: Test rig and specimen

3.4 Loading protocol

The FEMA 461 Quasi-Static loading protocol [30] is implemented to be consistent with other recently completed CFS fastener-level cyclic shear tests [21][22][23]. The loading protocol was modified to incorporate a small magnitude of compression displacement: 2.54 mm (0.1 in.) is estimated using a sine wave approximation for the buckling wave [29] with out-of-plane buckling deformation equal to 10.2 mm (0.4 in.) based on the shell finite element simulation of steel sheet shear walls in ABAQUS [31]. As depicted in Figure 5, the modified FEMA 461 loading protocol [30] demonstrates two repeated symmetric cycles increasing in magnitude by a factor of 1.4 until the compression displacement exceeds 0.1 in. and subsequent two repeated asymmetric cycles with only tension side increase by a factor of 1.4. The loading rate is 0.028 mm/sec (0.0011 in./sec) in the initial six cycles while later cycles employ 0.084 mm/sec (0.0033 in./sec) loading rate. For the monotonic tests that are conducted, the tests follow the AISI S905 [27] which uses a loading rate of 0.021 mm/sec (0.00083 in./sec).

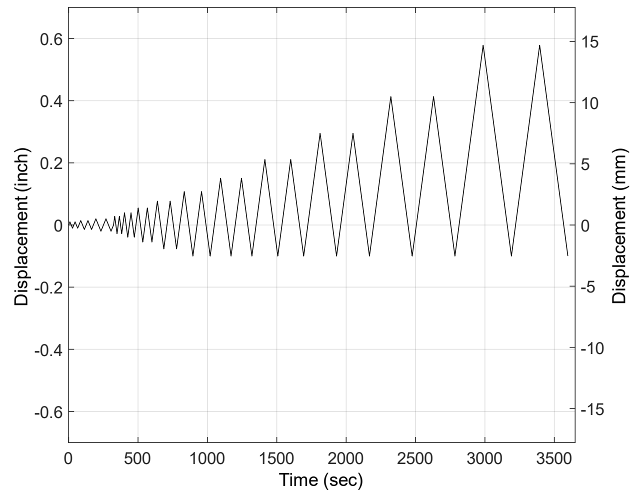


Figure 5: Asymmetric cyclic loading protocol

4. Test results

4.1 Result summary

Averaged summary statistics for each test series are provided in Table 2. Stiffness and strength are largely governed by sheet thickness with fastener size and framing thickness playing secondary roles. The initial stiffness k_0 is estimated based on the response at 40% of the peak strength (P_{max}). Deformation corresponding to the peak strength is donated as D_{max} while deformation corresponding to the 80% post peak force level is denoted as $D_{80\%}$.

Table 2: Average test results for each test series

Test series	Screw size	Sheet (mm)	Framing (mm)	*	k_0 (kN/mm)	D_{max} (mm)	P_{max} (kN)	$D_{80\%}$ (mm)
54-8-13	8	0.33	1.37	7	8.11	1.04	1.14	3.73
54-10-13	10	0.33	1.37	7	6.55	1.07	1.34	3.87
97-10-13	10	0.33	2.46	7	14.68	0.10	1.51	2.77
54-8-19	8	0.48	1.37	7	9.75	3.39	1.67	5.19
54-10-19	10	0.48	1.37	7	10.13	3.36	1.88	5.18
97-10-19	10	0.48	2.46	7	12.80	2.88	1.75	5.49
54-8-30	8	0.76	1.37	8	5.33	12.55	4.14	14.13
54-10-30	10	0.76	1.37	11	11.65	10.23	4.08	12.14
97-8-30	8	0.76	2.46	9	21.75	5.32	3.56	7.18
97-10-30	10	0.76	2.46	14	16.80	5.70	3.41	7.52
97-12-30	12	0.76	2.46	9	27.51	5.45	3.51	7.36

* Number of conducted tests in each test series

4.2 Typical behavior and failure modes

Predominant failure modes observed in the testing are bearing, tilting and bearing, pull-through with tilting/bearing, and shear rupture. Bearing, or bearing and tilting, is always developed prior to disengagement by either pull-through or shear rupture. The pull-through with tilting/bearing failure mode, as depicted in Figure 2d, occurs only after bearing, or tilting and bearing failure modes have been initiated and is accompanied by tearing of the thin steel sheet ply area in contact with the fastener head and subsequently described as “pull-through” herein. A plastic hinge always forms in the middle of the thin steel sheet after a small number of compression cycles.

Observed different behaviors between test series lie in the difference in thickness between the thick (framing) ply and the thin (sheet) ply. For fastener configurations with the framing and sheet thickness far apart, such as the “54-8-13” test series, bearing dominates [29] as shown in Figure 6. For the case where the thin steel sheet ply is constrained to buckle towards the fastener head the pull-through failure mode is incorporated into the bearing behavior after the peak force level, and is ultimately accompanied by the edge tear out and disengagement of the fastener from the thin steel sheet ply. In general, little difference is observed between forcing the thin ply buckling away from or towards the fastener head. Similar overall observations can be found in most tests with a 0.33 mm (13 mil) or 0.48 mm (19 mil) thin steel sheet ply [29].

(a) (b) (c) (d)

Figure 6: Deformation and failure of a test in the “54-8-13” test series: (a) Peak strength level front view; (b) Peak strength level side view; (c) 80% post peak strength level; (d) After test

For the “54-8-30” and “54-10-30” test series where framing and sheet are relatively close in thickness, the results are sensitive to whether the thin steel sheet ply is buckling away from or towards the fastener head. In the “54-8-30” test series where the thin steel sheet ply is constrained to buckle away from the fastener head primarily pull-through, with tilting and bearing, is the observed failure mode [29], as shown in Figure 7a, b, c, and d. Pull-through ultimately triggers disengagement of the fastener from thin steel sheet ply (it is not obvious in Figure 7d, but when the specimen is in compression the disengagement is readily observed). The demand on the fastener in the test is primarily shear with a small amount of tension in the pre-peak load regime and shear-tension interaction (demand) at and after the peak load. The pull-through limit state is also accompanied by the fastener head tearing the thin steel sheet ply area in contact with the fastener head past peak load.

In the “54-8-30” test series where the thin steel sheet ply is constrained to buckle towards the fastener head bearing, fastener tilting, and shear rupture are observed [29], as shown in Figure 8a, b, c, and d. Bearing, fastener tilting, and shear rupture limit states are all observed in the thinner 0.76 mm (30 mil) sheet ply throughout the test and demand for the fastener is predominately shear. The tearing deformation (shear rupture) of the thin steel sheet ply demonstrating longitudinal shearing of the thin steel sheet along two approximately parallel planes is initialized prior to peak load and develops as the test progresses until disengagement.

(a) (b) (c) (d)

Figure 7: Deformation and failure of a “54-8-30” test with thin sheet buckling away from the fastener head: (a) Peak strength level front view; (b) Peak strength level side view; (c) 80% post peak strength level; (d) After test

(a) (b) (c) (d)

Figure 8: Deformation and failure of a “54-8-30” test with thin sheet buckling towards the fastener head: (a) Peak strength level front view; (b) Peak strength level side view; (c) 80% post peak strength level; (d) After test

The observed sensitivity in strength and failure mode to the buckling direction of the thin steel sheet ply is not universal. When the framing-to-sheet thickness ratio are between the aforementioned two cases, including “97-8-30”, “97-10-30”, and “97-12-30” test series, the buckling direction (away or towards) influences the observed behavior in only a few cases in the same test series. No fastener tilting is observed in the tests with this configuration since the 2.46 mm (97 mil) framing steel is quite stiff. Bearing and pull-through is the dominant failure modes for most asymmetric cyclic tests with these fastener connection configurations.

A representative test in the “97-12-30” test series with the thin steel sheet constrained to buckle away from the fastener head [29] is shown in Figure 9a, b, c, and d. The bearing limit state is observed in the thinner sheet ply throughout the test and the pull-through limit state gradually develops after peak load. Another test in the same “97-12-30” test series with the thin steel sheet constrained to buckle towards the fastener head [29] is depicted in Figure 10a, b, c, and d. Different from the former test, bearing and shear rupture is the dominant failure modes for this test and the demand on the fastener is predominately shear.

(a) (b) (c) (d)

Figure 9: Deformation and failure of a “97-12-30” test with thin sheet buckling away from the fastener head: (a) Peak strength level front view; (b) Peak strength level side view; (c) 80% post peak strength level; (d) After test

(a) (b) (c) (d)

Figure 10: Deformation and failure of a “97-12-30” test with thin sheet buckling towards the fastener head: (a) Peak strength level front view; (b) Peak strength level side view; (c) 80% post peak strength level; (d) After test

4.3 Force-Displacement response

Force-displacement response is highly nonlinear, but overall trends based on the relative difference in thickness between the two steel plies are still readily observed. Response in the representative test series when the thickness of the framing ply and sheet ply are relatively similar are provided in Figure 11, and when the thickness of the two plies are far apart in Figure 12. The “54-8-30” test series responses as provided in Figure 11 indicates the response is sensitive to whether the thin steel sheet ply (in contact with the fastener head) is buckling away from (denoted with an A) or towards the fastener head (denoted with a T) in the compression cycles, the monotonic test is denoted with a M. The buckling away cases create additional tension demand on the fastener connection which triggers the pull-through limit state and degrades the strength and post-peak shear behavior. In the buckling towards cases, the thin steel sheet ply tends to flatly align with the shear load path and the fastener head does not create additional tension demand on the connection, resulting in only bearing and no pull-through limit state. The dominant limit states are bearing and shear rupture, which results in higher strength. Moreover, buckling towards cases demonstrating higher test strength than the buckling away cases can also be observed in other configurations including the “54-10-30”, “97-8-30”, “97-10-30”, and “97-12-30” [29].

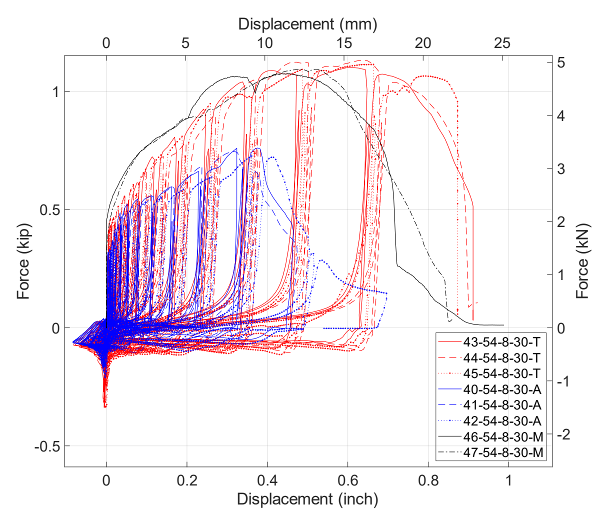


Figure 11: Force-displacement curves of “54-8-30” test series

The test strength is not sensitive to whether the thin steel sheet ply is buckling away from or towards the fastener head when the framing thickness and steel sheet thickness are far apart, as shown in Figure 12. As provided for the “54-8-13” test series in Figure 12, great consistency in strength and post-peak behavior for the monotonic tests and cyclic tests with thin steel sheet buckling away or towards the fastener head is observed. The thin 0.33 mm (13 mil) steel sheet ply is thin and flexible under compression demand and does not significantly influence the connector behavior. The pre peak load response is dominated by bearing, but the combined shear-tension interaction demand on the fastener connection triggers the pull-through limit state and ultimately disengagement of the parts. The same insensitivity can also be detected in the “54-10-13”, “54-8-19”, “54-10-19”, “97-10-13”, and “97-10-19” [29].

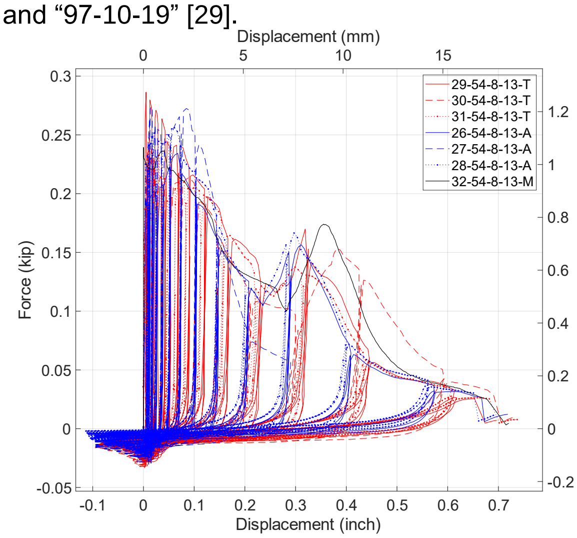


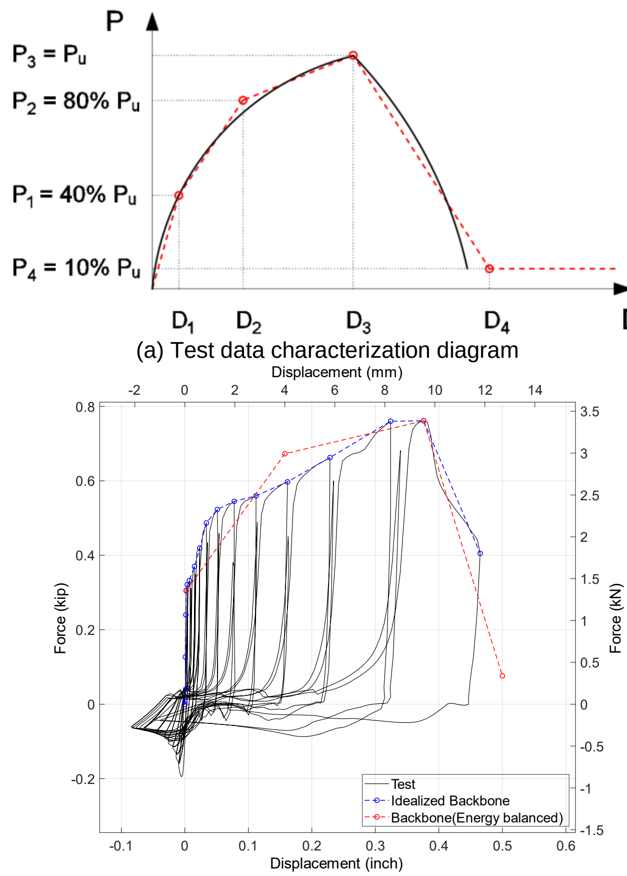
Figure 12: Force-displacement curves of “54-8-13” test series

4.4 Connector behavior characterization

4.4.1 Multi-segment linear backbone curve - procedure

A procedure is developed for idealizing the test results with a multi-segment linear backbone phenomenological model to provide a convenient means to implement the tested connections in models. A four segment model, consistent with the Pinching4 material model in OpenSees, is selected for the backbone. The model is fit by balancing energy between the linear segment model and the nonlinear test results. Only the tension side test result is adopted for the data characterization herein. As illustrated in Figure 13a, the developed modeling parameters (D_1,P_1; D_2,P_2; D_3,P_3; D_4,P_4) are intended to support numerical models which need to simulate the nonlinear (hysteretic) fastener response, e.g. in a shear wall simulation. Test data characterization detailed results are tabularized and provided in the test report [29].

A cyclic test in the “54-8-30” test series [29] is adopted herein to detail the characterization procedure. As shown in Figure 13b, the procedure first generates an “idealized backbone” based on the tension side cyclic test data, composed of the peak load point of each loading step before the last loading cycle and the peak displacement point of the last loading cycle. Then a multi-segment linear backbone model is developed based on energy dissipation balance (i.e., the accumulative product of force and displacement) between the “idealized backbone” and the multi-segment linear backbone model. The multi-segment linear backbone model consists of four points, as shown in Figure 13a, the third point is the peak strength point in the test curve while the strength value of the first, second and fourth point are set as 40%, 80%, and 10% (post-peak) of the peak strength respectively. The first point displacement is determined based on the force level and the initial stiffness of the test curve, and the second point displacement is used for adjusting the linear backbone model’s energy dissipation (area underneath the backbone curve) to be the same with the “idealized backbone” from the test results before the peak strength. Similarly, the fourth point displacement is adopted to balance the energy dissipation after the peak strength. For the monotonic test curves, the test curve itself is an “idealized backbone” and the same multi-segment linear backbone phenomenological model is applied.



(b) Characterization of test #40 in the “54-8-30” test series

Figure 13: Backbone data characterization based on equivalent cumulative energy dissipation

4.4.2 Multi-segment linear backbone curve - result

Individual, fitted, multi-segment linear backbone results for each test are provided in in the test report [29]. It is expected for modeling in steel sheet shear walls average backbone curves of the connections will be used, therefore averaged results, even with all their simplifications, are provided here. For the test series with sensitivity to buckling direction the averaging is the most approximate, as illustrated for the "54-8-30" test series in Figure 14 where averages of the monotonic tests, and cyclic tests (including depending on direction of buckling for the thin sheet) are all provided. The average cyclic response is recommended for modeling connections in steel sheet shear walls, as summarized in Table 3.

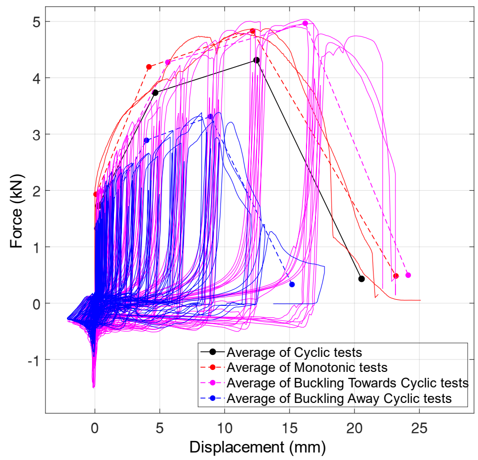


Figure 14: Test data characterization average values for "54-8-30"

Table 3: Average four point backbone values for all test series

Test series	D_1 (mm)	D_2 (mm)	D_3 (mm)	D_4 (mm)	P_1 (kN)	P_2 (kN)	P_3 (kN)	P_4 (kN)
54-8-13	0.06	0.11	1.04	13.13	0.45	0.97	1.14	0.11
54-10-13	0.08	0.13	1.07	13.65	0.54	1.13	1.34	0.13
97-10-13	0.04	0.07	0.10	12.11	0.60	1.20	1.51	0.15
54-8-19	0.07	0.53	3.39	11.50	0.67	1.44	1.67	0.17
54-10-19	0.07	0.13	3.36	11.53	0.75	1.57	1.88	0.19
97-10-19	0.05	0.10	2.88	14.61	0.70	1.47	1.75	0.18
54-8-30	0.31	4.80	12.55	19.66	1.66	3.58	4.14	0.41
54-10-30	0.14	2.98	10.23	18.85	1.63	3.52	4.08	0.41
97-8-30	0.07	1.58	5.32	13.68	1.42	3.01	3.56	0.36
97-10-30	0.08	1.66	5.70	13.88	1.36	2.93	3.41	0.34
97-12-30	0.05	0.75	5.45	14.02	1.40	2.88	3.51	0.35

5. Code strength predictions

The failure modes observed in this testing program include bearing, tilting and bearing, pull-through, and shear rupture. The bearing, or tilting and bearing strength limit states develop before the pull-through or shear rupture. The fastener connection test strength can be predicted by the screw shear strength limited by tilting and bearing provisions in J4.3.1 in the AISI S100-16 [26], as shown in Eq 1.

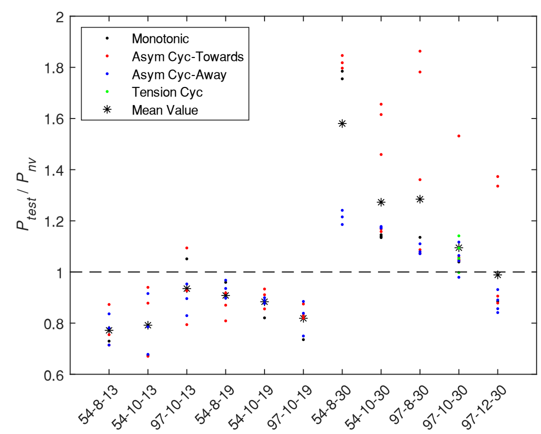
$$P_{nv}=4.2(t_2{}^3 d)^{1/2}F_{u2} \quad (1.1)$$
$$P_{nv}=2.7t_1 dF_{u1} \quad (1.2)$$
$$P_{nv}=2.7t_2 dF_{u2} \quad (1.3)$$

Where t_1 and F_{u1} are the thickness and ultimate strength of the steel sheet in contact with the screw head (always the thinner sheet ply in the tests here), t_2 and F_{u2} are the thickness and ultimate strength of the steel sheet not in contact with the screw head (the framing ply in the tests here), and d is the screw diameter. For $t_2 / t_1 \leq 1.0$, P_{nv} shall be taken as the smallest of Eq 1.1, Eq 1.2, and Eq 1.3. For $t_2 / t_1 \geq 2.5$, P_{nv} shall be taken as the smaller of Eq 1.2 and Eq 1.3. Interpolation is needed if $2.5 > t_2 / t_1 > 1.0$. In the specimens studied here t_2 / t_1 is always larger than 2.5 except the 54-(8 or 10)-30 series, and the shear strength for the fastener connections tested herein are limited by thin steel sheet bearing, which can be predicted with provision Eq 1.2. Comparison of the test to code strength is provided as ratios in Figure 15a.

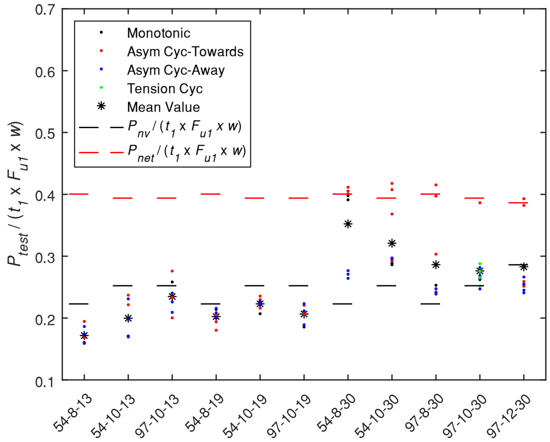
Shear rupture is commonly observed in the tests at final failure, for comparison purposes we also evaluated the connection strength limited by shear rupture provision from J6.1-1 in the AISI S100-16 [26], as presented in Eq. 2.

$$P_{net}=0.6F_{u1}2t_1 e_{net} \quad (2)$$

Where e_{net} is the clear distance between end of material and edge of fastener hole. The test and prediction strength values are normalized with t_1 x F_{u1} x w, where w implies specimen width taken as 2 in. herein, as shown in Figure 15b.



(a) Test-to-code predicted strength ratio



(b) Normalized test and code predicted strength

Figure 15: Test strength and code strength prediction

As presented in Figure 15a, tests with 0.33 mm (13 mil) and 0.48 mm (19 mil) thin steel sheet ply demonstrate test-to-predicted ratios lower than 1, i.e. unconservative predictions. It can further be observed from Figure 15a that when the sheet plies are in the same configuration the test-to-predicted ratio typically decreases with the increase of the screw diameter. By normalizing to the ideal strength, as given in Figure 15b, one can observe that the cases with 0.76 mm (30) mill sheet ply, in which the buckling of the thin sheet ply is towards the fastener head, have strength consistent with shear rupture. While all other tests, including the same configurations but with buckling of the thin ply in the opposite direction, have strength closer to the bearing capacity.

6. Conclusions

Nonlinear cyclic response of the connection between the framing and steel sheet sheathing significantly influences the seismic performance of CFS-framed screw fastened steel sheet sheathed shear walls. Further, the impact of the steel sheet shear buckling on the strength and ductility of this critical connection should be considered. Little existing data exists for the behavior of these connections in shear, especially cyclic data at the relevant combinations of thin steel sheet and thick steel framing. A cyclic lap shear testing protocol was developed with small compression displacements to buckle the thin sheet ply in the lap. The setup is found to be appropriate for investigating the impact of sheet buckling and the resulting "shear-tension" interaction demand on the connection response. For configurations where the framing and sheet thickness are relatively close (e.g. 1.37 mm (54 mil) framing and 0.76 mm (30 mil) sheet) the test strength is sensitive to the direction in which the thin steel sheet ply is buckled. When the sheet ply buckles away from the fastener head it can create additional tension demand on the connection which can trigger a pull-through limit state that degrades the strength and post-peak shear behavior of the connection. For configurations where the framing and sheet thickness are far apart (e.g., 1.37 mm (54 mil) or 2.46 mm (97 mil) framing with 0.33 (13 mil) or 0.48 mm (19 mil) sheet) the test strength is not sensitive to the buckling direction of the thin steel sheet ply. Nonetheless, the buckling of the thin ply still influences the results as the additional shear-tension interaction demand on the connection degrades the strength modestly and influences the post-peak response. Screw shear strength limited by the tilting and bearing provision in J4.3.1 of AISI S100-16 [26] provides reasonable shear strength predictions for connections with 0.76 mm (30 mil) steel sheet, but an adjustment for thinner 0.33 mm (13 mil) or 0.48 mm (19 mil) steel sheet connections is needed. The cyclic fastener testing data and the multi-linear backbone curve generated by characterization of the testing provide critical missing information for the design and simulation of cold-formed steel framed steel sheet sheathed shear walls.

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