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EXPERIMENTAL TESTING OF HIGH-CAPACITY WOOD-FRAME WOOD STRUCTURAL PANEL SHEAR WALLS

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ABSTRACT: The purpose of this study was to develop a one-sided shear wall system with multiple rows of nails at edges of the panels to increase its shear capacity. Monotonic and cyclic tests were performed on 2.44 m x 2.44 m shear walls with two rows of 8d common nails (3.3 mm x 63.5 mm) and special boundary details that include double bottom plates, 2x6 nominal framing members, and multi-story boundary details but no supplemental framing hardware. Failure modes included nail withdrawal, nail head pull-through, and sheathing edge tearout that are typical ultimate failure modes for wood-frame WSP shear walls with a single row of fasteners at panel edges. The seismic equivalency parameters derived from the cyclic tests of these walls also satisfied the equivalency criteria specified in ASTM D7989. These one-sided high-capacity shear walls have a nominal unit shear capacity of 59671 N/m (4090 plf) that is 100% greater than the nominal unit shear capacities for shear walls constructed with a single row of 8d common nails and about 70% greater than the highest capacity shear wall in the current SDPWS standard (35525 N/m (2435 plf)) and can be utilized as a solution in multi-story applications.

KEYWORDS: shear wall, wood-frame construction, SDPWS, high-capacity shear wall, racking test

1 – INTRODUCTION

In the United States, the 2021 ANSI/AWC *Special Design Provisions for Wind and Seismic* (SDPWS) [1] standard is referenced in the 2024 International Building Code [2] and serves as a primary code resource for the design of wood structural panel (WSP) shear walls. At present, 35525 N/m (2435 plf)[4] is the highest SDPWS nominal in-plane unit shear capacity for a one-sided sheathed WSP shear wall. While higher capacities can be achieved with sheathing applied to both sides of the framing, sheathing both sides can be problematic in application due to construction and detailing requirements to obtain fire resistance, sound transmission ratings, and for trade sequencing when the wall cavity contains insulation and utilities. For multi-story applications that require high-capacity shear wall systems and utilize continuous rod tie-down systems, there is designer interest in developing a one-sided sheathing solution.

The shear wall configurations with the highest nominal unit shear capacity (35525 N/m (2435 plf)) in the current SDPWS standard use either 19/32 performance category WSP sheathing or 15/32 performance category Structural 1 (S1) WSP sheathing and are attached with 10d common nails at 50.8 mm (2”) on center at all panel edges. According to the 2020 edition of the *Panel Design Specification* (PDS) [3], the OSB version of these products have allowable shear-through-the-thickness capacities of 50419 N/m and 54651 N/m (3,455 lb/ft and 3,745 lb/ft), respectively. This corresponds to nominal shear-through-the-thickness capacities of 90746 N/m and 98347 N/m (6220 lb/ft and 6741 lb/ft) per National Design Specification for Wood Construction (NDS®) [4]. For the OSB WSP sheathing products in particular, this suggests that reserve panel shear capacity is available and can be utilized to allow for additional sheathing fasteners to be used to increase the nominal in-plane unit shear capacity of a one-sided sheathed shear wall to levels above 35525 N/m (2435 plf).

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[4] 1 lb = 4.448 N; 1 m = 3.28 ft

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The objective of this test program was to further explore the potential to develop a one-sided sheathed shear wall system that, similar to a high-capacity diaphragm, uses multiple rows of nails at panel edges to increase the capacity. The tested walls were then evaluated based on the ASTM D7989 [5] standard that is used to judge seismic equivalency to WSP shear walls in the SDPWS. ASTM D7989 establishes seismic equivalency parameters in terms of drift capacity, component overstrength, and ductility which are then used to evaluate the acceptability of new shear wall systems.

2 – EXPERIMENTAL PROGRAM

2.1 TEST SETUP AND CONFIGURATION

The shear wall configuration is shown in Fig.1, the test setup is shown in Fig. 2, and the test matrix is provided in Table 1. The basic test wall configuration of 2.44 m x 2.44 m (8 ft. x 8 ft.) was constructed using 2x6 nominal (1.5 in x 5.5 in. (38.1mm x 139.7 mm)) Douglas-fir framing spaced at 406.4 mm (16 in.) on center, 11.9 mm (15/32 in.) Structural 1 OSB sheathing with two rows of 8d common nails (3.3 mm x 63.5 mm (0.131 x 2.5 in.)) spaced at 50.8 mm (2 in.) on center at panel edges, and one row of 8d common nails spaced at 304.8 mm (12 in.) on center in the field of the panels. This corresponds with a nominal unit shear capacity of 59642 N/m (4090 plf) calculated based on the SDPWS assuming two times the capacity of a shear wall using 11.9 mm (15/32 in.) Structural 1 OSB attached with 8d common nails in a single sheathing nail row. This design unit shear capacity was used to verify the number of anchor bolts, size of the rod tie-downs, attachment between center post plies, etc. The number of framing plies at the end post, center post, and bottom plate were not necessarily optimized because they were governed by the need to accommodate symmetric placement of two rows of staggered 8d common nails at panel edges.

The boundary condition and the details, shown in Fig. 1, were developed to represent the performance that might be expected in the first story of a multi-story wood-frame WSP shear wall building. The concentric continuous rod tie-down system and relatively rigid load beam were chosen to be representative of this targeted application. The shear wall configuration also included double bottom plates and no supplemental framing hardware were used at the wall corners to mitigate the effects of the eccentricity created by the one-sided sheathing attachment.

The pneumatic nails used for sheathing attachment did not have an adhesive coating, since the long-term efficacy of adhesion from these variable coatings is unknown. At panel boundaries, these fasteners were evenly divided between the framing plies as depicted in Fig. 3 for the adjoining panel edge condition. Whenever a row of fasteners occurred in a single framing member, adjacent fasteners within the row were staggered.

The 4-ply studs at the center post were interconnected with Simpson Strong Tie [6] SDS screws designed to transfer the design shear force between panels.

2.2 MATERIAL

Framing

All of the wall stud and plate material used for this study was "No. 2 and better" grade Douglas fir lumber of nominal 2x6 (1.5 in x 5.5 in. (38.1mm x 139.7 mm)) size, that was high-temperature kiln-dried to a maximum moisture content of 19%. The lumber was pre-sorted for a target average specific gravity of 0.50. Materials with an estimated oven-dry specific gravity between 0.46 and 0.57 were used for framing members at panel edges and lumber with substantially higher and lower specific gravities were used for framing members that did not receive nails at panel edges.

Sheathing

The 11.9 mm (15/32 in.) S1 OSB sheathing used for this investigation was "sized for spacing," had an "Exposure 1" durability rating, a PS-2 [7] span rating of "32/16," and was stamped with APA as the third-party inspection agency.

Nails

All of the framing nails were bright nails, full-size, smooth shank, and round head nails with variable adhesive coatings.

Sheathing nails used for this study were 8d common (3.3 mm diameter x 63.5 mm long x 7.1 mm diameter (0.131 x 2.5 in. x 0.281 in.)) gun nails. These were bright, common 21º stick-collated pneumatic fasteners that did not have an adhesive coating. Physical measurements from 10 randomly selected nails suggested that they had an average diameter of 3.2 mm (0.127 in.), an average length of 62.2 mm (2.45 in.), and an average head diameter of 6.9 mm (0.272 in.).

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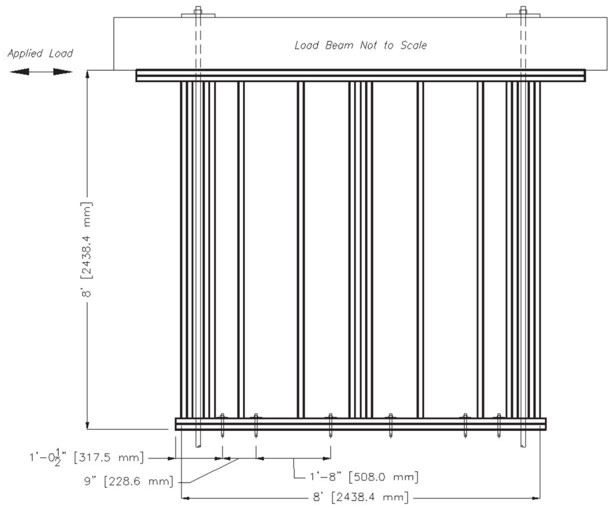

Figure 1: Basic wall configuration

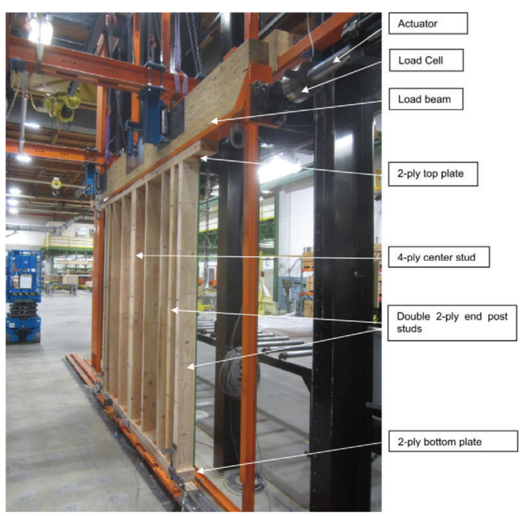

Figure 2: High-capacity shear wall test setup

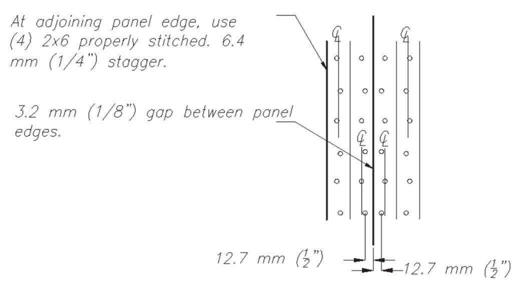

Figure 3: Sheathing to stud at adjoining panel edges

Table 1: High-capacity shear wall test matrix

ID	n	Wall Size (l x h) m (ft.)	Single Side OSB Sheath	Sheathing Common Nail (nominal size) mm x mm (in. x in.)	Type/coating	Sheath Nail Spacing (Rows-Edge/Field) mm (in.)	DF Stud	Stud Spacing mm (in.)	Concentric Rod Tie-Down Size mm (in.)	Anchors #-mm (in.)	Anchor Bolt Washer mm (in.)	Load Beam	Loading
A1	2	2.44 m x 2.44 m (8 ft. x 8 ft.)	15/32 S1	8d (3.3 mm x 63.5 mm (0.131 in. x 2.5 in.))	Gun/none	(2)-50.8 mm/304.8 mm (2 in./ 12in.)	2x6 No. 2	406.4 mm (16 in.)	31.8 mm (1 ¼ in.)	6 – 22.22 mm (7/8 in.)	76.2 x 114.3 x 5.8 mm (3 x 4.5 x 0.23)	Rigid	Monotonic[1]
A2	2												Cyclic[2]

[1] Method D of ASTM E2126, a continuous displacement ramp of 25.4 mm/min. (1 in./min.)
[2] Method C of ASTM E2126, CUREE loading protocol with Δ = 61 mm (2.4 in.) and α=0.5.

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Anchorage
All rod tie-downs were 31.8 mm (1¼ in.) diameter A307 [8] Grade A threaded rods. The 22.22 mm (7/8 in.) anchor bolts were A193 [9] Grade B7 threaded rod stock. 76.2 mm x 114.3 mm x 5.8 mm (3 in. x 4.5 in. x 0.23 in.) anchor bolt washers and 127 mm x 228.6 mm x 25.4 mm (5 in. x 9 in. x 1 in.) tie-down washers were used in the tests. All metal anchorage parts were re-used between the tests since they did not show any visible signs of damage or distortion.

Stitch Screws
The stitch screws used for the center studs were Simpson Strong-Tie [6] SDS hex head screws 6.4 mm x 152.4 mm (¼ in. x 6 in.). Due to product availability, these fasteners had a galvanized coating. They were not re-used between tests.

2.3 LOADING PROTOCOL

The shear wall racking tests were conducted in general accordance with ASTM E2126 [10]. An additional deflection measurement device was added to measure slip between the double bottom plates. The monotonic wall tests were conducted using a continuous displacement-controlled ramp of 25.4 mm/min (1.0 in./min) generated by a hydraulic controller system in accordance with Method D of ASTM E2126. The cyclic wall tests used the Method C "CUREE" loading protocol of ASTM E2126. The initial Alpha and Delta assumptions to determine the cycle magnitudes were consistently chosen as 0.5 and 61 mm (2.4 in.), respectively. All the electronic measurement devices were monitored throughout each test by a computerized data acquisition system.

2.4 TESTING

All testing in this study was conducted at the ISO 17025 [11] accredited Weyerhaeuser Engineering and Material Testing Laboratory (WEMTL) in Federal Way, WA.

Two shear walls were tested monotonically, and two shear walls were tested cyclically to determine whether this configuration would satisfy the seismic equivalency criteria of ASTM D7989.

The in-plane loads applied to the wall were measured using an electronic load cell positioned between the actuator and load head. Lateral deflections were measured at the top of the wall using a temposonic wand referenced to the wall top plate. Linear motion potentiometers (LMPs) were positioned to measure vertical displacement at the bottom of each outer end post and horizontal displacement at the horizontal bottom plate. An additional deflection measurement device, as mentioned above as an exception to the protocol, was added to measure slip between the double bottom plates.

Each wall was installed into the test fixture and anchored as shown in Fig. 1. The anchor bolts were tightened per Section 6.2.3 of ASTM E2126. The pre-tightening requirements for the concentric anchor rods are not addressed by ASTM E2126; however, these were tightened in a similar manner. Due to their length, they were tightened 1/4 turn instead of the 1/8 turn normally employed for anchors used at the base of the wall. Each wall was then allowed to sit for at least 10 minutes prior to the testing.

The test fixture used for the test provided at least 44.4 mm (1.75 in.) of rotational clearance for the sheathing at both the top and bottom of the wall. The 139.7 mm (5.5 in.) wide load head provided clearance at the top of the wall. A 44.4 mm x 139.7 mm (1.75 x 5.5 in.) Microllam® LVL spacer plate provided clearance at the bottom of the wall. Each wall specimen was loaded well past peak load for purposes of generating data at large displacements for use in seismic modeling, until an obvious failure was achieved or only a small portion of the peak capacity remained.

3 – RESULTS

3.1 FAILURE MECHANISM

All of the one-sided sheathed shear walls in this study experienced more than typical deflection at their seismic design level. Visual observations suggested that the early wall deformation for all configurations came from a combination of:

- Eccentric out-of-plane behavior caused by the heavy attachment of the single WSP sheet to the wall.
- Compression deformation from the double top and bottom plate.
- Anchor rod stretch in tension and compression in the load header.
- Sliding of the plate within the 1.6 mm (1/16 in.) oversized anchor bolt holes.
- Shear deformation of the WSP.
- Limited slip between the WSP and the framing.

As loading continued, progressively increasing amounts of slip were observed due to relative rotation between sheathing and the framing. Eventually, as illustrated in Fig. 4, this movement became a primary source of lateral deformation, and failure occurred mostly due to localized detachment of the sheathing from the framing through a of combination of nail withdrawal, nail head pull-through,

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and sheathing edge tearout. With the sheathing applied only on one side, as shown in Fig. 5, this held the framing together rigidly on one face of the wall and caused framing separations on the other side of the wall as the plates progressively twisted and cracked due to cross-grain bending. Aside from some localized plate splitting at the ends of the shear wall, the framing was intact at the end of these tests. Late in Wall Test A1_01, a vertical tear occurred in the sheathing. Otherwise, sheathing deterioration was limited to the panel edges.

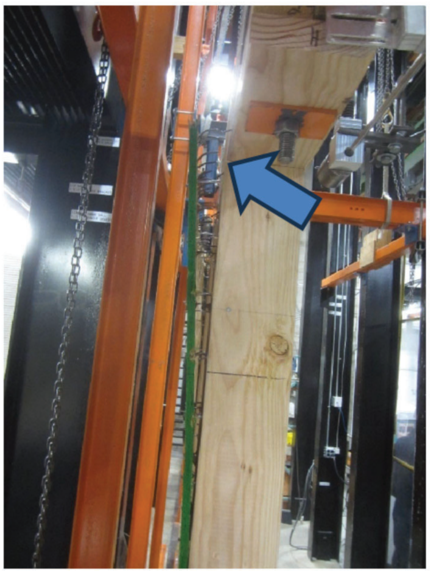

Figure 4: Typical WSP plate detachment failure mode

Figure 5: Typical cross-grain splitting caused by eccentricity between the one-sided WSP sheathing and the anchor bolts

3.2 LOAD DISPLACEMENT DATA

Table 2 and Figs. 6 through10 summarize the numerical results from the shear wall test portion of this investigation. Table 2 includes a number of calculated quantities that were derived using the terminology and protocols described in ASTM E2126 and/or ASTM D7989.

The cyclic test data was used to produce an average backbone curve using the procedures of ASTM E2126. Those average backbone curves were then used to determine the drifts of each wall at the seismic design racking load, the load and drifts at the wall peak lateral capacity, and the load and drifts at the post-peak "ultimate" capacity (load drops to 80% of peak). For these calculations, the load was assumed to be twice the SDPWS seismic design load for the corresponding configuration with a single row of panel edge fasteners. Shear walls tested under monotonic loading were analyzed in a similar manner for reference purposes. The sole exception was that the actual envelope curves were used for the analysis instead of an average cyclic backbone curve.

Cyclic backbone and monotonic load-displacement curves are presented in Fig. 10. In general, all of the one-sided sheathed shear walls in this study experienced more than typical deflection at their assumed seismic design load.

3.3 SEISMIC EQUIVALENCY PARAMETERS BASED ON ASTM D7989

ASTM D7989 evaluates equivalency to wood-frame WSP shear walls in SDPWS based on seismic equivalency parameters (SEPs) in terms of component overstrength, drift capacity, and ductility. These are provided in Table 1 of ASTM D7989 and are described as follows:

Component Overstrength:

$$2.5 \leq P_{peak,avg} / P_{ASD} \leq 5.0 \qquad (1)$$

Drift Capacity:

$$\Delta_{U,avg} \geq 0.028\ h \qquad (2)$$

Ductility:

$$\Delta_{U,avg} / \Delta_{ASD,avg} \geq 11 \qquad (3)$$

where $P_{peak,\ avg}$ = average peak load for all replicates of the wall configuration, P_{ASD} = allowable design load for the wall configuration (which is the nominal unit shear capacity divided by 2.8), $\Delta_{U,avg}$ = average ultimate displacement for all the replicates, h = heigh of the shear wall, and $\Delta_{ASD,avg}$ = average displacement corresponding to the allowable design load for all the replicates of the wall configuration.

The average backbone curves were used to calculate the SEPs which are provided in Table 2. Where provided, the equivalency parameters of ASTM D7989 for the monotonic tests should be used for comparison purposes only. A monotonic load protocol test cannot be used to judge equivalency per ASTM D7989. Looking at the results of the cyclic tests, the new developed high-capacity shear wall meets the ASTM D7989 equivalency criteria.

Table 2: Test Results[1]

ID	**0.4 Peak**		**ASD Seismic Design[2]**		**EEEP Yield**		**Peak[3]**		**Ultimate[4]**		**SG**		Primary failure mode[5]	**Normalized parameter[6]**		
	Load N (lbs.)	*Disp. mm (in.)*	*Load N (lbs.)*	*Disp. mm (in.)*	*Load N (lbs.)*	*Disp. mm (in.)*	*Load N (lbs.)*	*Disp. mm (in.)*	*Load N (lbs.)*	*Disp. mm (in.)*	*Avg.*	*Max.*		*Drift at Ult. (%)*	*Peak/ASD Load*	*Ult. Disp./ASD Disp.*
A1_01	68,419.1 (15,382)	14.4 (0.565)	51,979.3 (11,686)	9.7 (0.382)	150,885.1 (33,922)	31.6 (1.25)	171,047.8 (38,455)	107.5 (4.23)	136,838.3 (30,764)	136.8 (5.39)	0.49	0.55	W, PX, ST	5.6	3.29	14.1
A1_02	68,859.5 (15,481)	13 (0.511)	51,979.3 (11,686)	8.3 (0.325)	151,685.7 (34102)	29.1 (1.14)	172,150.9 (38,703)	104.9 (4.13)	137,719 (30,962)	146.7 (5.78)	0.48	0.57	W, P, PX	6.0	3.31	17.8
Avg.	68,641.5 (15432)	13.7 (0.538)	51,979.3 (11,686)	9 (0.354)	151,285.4 (34,012)	30.4 (1.2)	171,599.4 (38,579)	106.2 (4.18)	137,278.6 (30,863)	141.8 (5.58)	**0.48**	**0.56**		**5.8**	**3.30**	**15.9**
A2_01	62,076.3 (13956)	11.1 (0.437)	51,979.3 (11,686)	8 (0.314)	137,078.5 (30,818)	24.5 (0.97)	155,190.7 (34,890)	88.8 (3.5)	124,152.6 (27,912)	109.9 (4.33)	0.48	0.57	W, F, T, PX	4.5	2.99	13.8
A2_02	63,188.3 (14206)	13.3 (0.523)	51,979.3 (11,686)	10.2 (0.4)	139,849.6 (31,441)	29.4 (1.16)	157,975.2 (35,516)	87.4 (3.44)	126,381 (28,413)	102.2 (4.02)	0.48	0.57	W, P, PX	4.2	3.04	10.1
Avg.	62,632.3 (14081)	12.2 (0.48)	51,979.3 (11,686)	9.1 (0.357)	138,461.8 (31,129)	26.9 (1.06)	156,582.9 (35,203)	88.1 (3.47)	125,264.6 (28,162)	106 (4.18)	**0.48**	**0.57**		**4.3**	**3.01**	**11.9**

Notes:

[1] 1 lb=4.448 N, 1 in. = 25.4 mm

[2] ASD design load calculated in accordance with the 2021 Special Design Provisions for Wind and Seismic that is defined as vn/2.8, assuming 2 times the capacity of a shear wall using 11.9 mm (15/32 in.) Structural 1 OSB attached with 8d common nails in a single sheathing nail row. V_{ASD}=2 x (2045 plf/2.8) 8ft = 11686 lbs=51979.3 N;

[2] Point where maximum load is achieved in the cyclic average backbone or monotonic curve.

[3] Post peak point where the load drops to 80% of the peak load in the average backbone curve and smoothed representation of the monotonic curves.

[4] Failure codes

- T – sheathing panel edge tearout
- W – sheathing nail withdrawal
- P – head pull-through
- ST – sheathing panel tear
- PX – panel cross-grain bending

[5] Cyclic test parameters calculated in accordance with ASTM D7989. Monotonic parameters provided for reference purposes only.

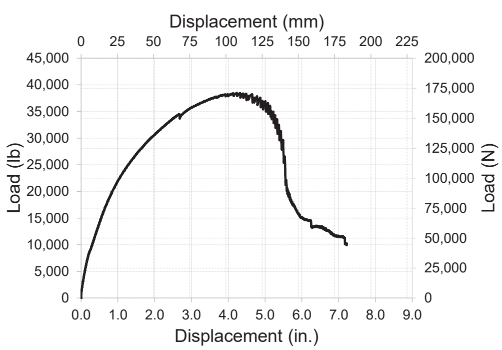

Figure 6: Test A1_01 monotonic

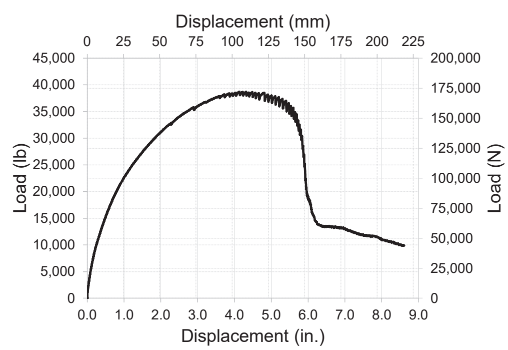

Figure 7: Test A1_02 monotonic

4 – CONCLUSION

Tests of high-capacity wood-frame WSP shear walls with two rows of 8d common nails at panel edges demonstrated failure modes associated with typical wood-frame WSP shear walls with single row of fasteners at panel edges. The seismic equivalency parameters calculated from the cyclic tests of these walls also met the equivalency criteria outlined in ASTM D7989. This can serve as a solution in multi-story applications and requires special detailing including double bottom plates, 2x6 framing members, and multi-story boundary details with continuous rod tie-down system.

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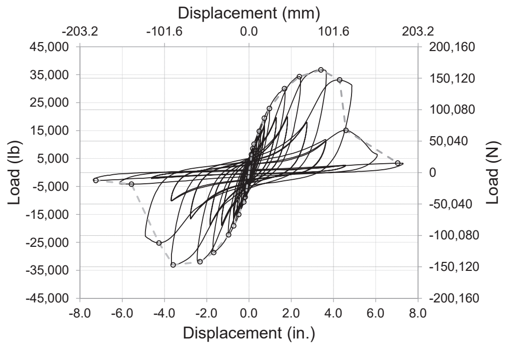

Figure 8: Test A2_01 hysteresis with backbone curve

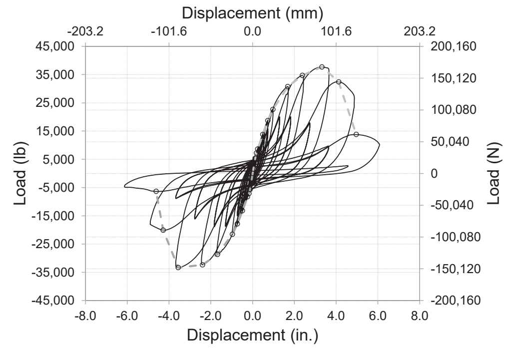

Figure 9: Test A2_02 hysteresis with backbone curve

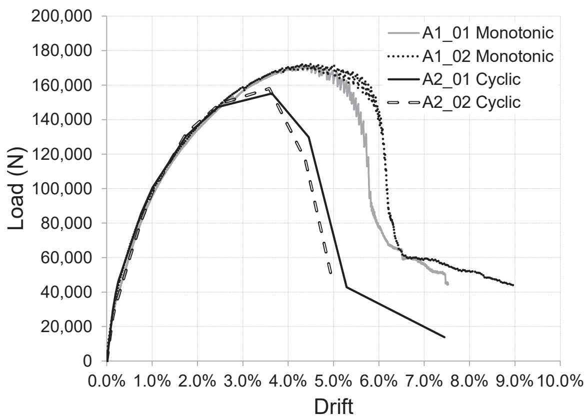

Figure 10: Backbone curves

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