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Editorial

This Wood Design Focus summarizes three recent wood industry research programs that investigated the performance of wood frame shear walls and their in-plane shear performance, i.e. racking performance. All three of these programs consist of structural sheathing nailed to repetitive wood framing.

In the first paper, the cyclic in-plane shear performance of unblocked shear walls were experimentally evaluated and shown to be similar to that of blocked shear walls with respect to commonly recognized seismic performance attributes. Prior testing of unblocked shear walls used different cyclic loading protocols than those used to evaluate seismic compatibility with blocked wood structural panel shear walls. The study also offers suggestions for refinement of design provisions to achieve greater compatibility in performance to blocked shear walls construction that may appear in future design provisions.

The second paper summarizes testing of wood frame wood structural panel shear walls with openings as well as corner returns associated with the continuous sheathed wood structural panel (CS-WSP) bracing method in the International Residential Code (IRC). This testing documents the performance of wood structural panel shear walls using established procedures for evaluating the adequacy of alternative sheathing products. Increases in the strength performance targets of AC269.1 for CS-WSP bracing may be warranted.

In the third paper, in-plane racking strength tests of structural fiberboard perforated shear walls with openings are summarized. The design of structural fiberboard perforated shear walls is recognized in the Wood Frame Construction Manual's (WFCM's) prescriptive provisions; however, the applicability of the perforated shear wall design method provisions in AWC's Special Design Provisions for Wind and Seismic (SDPWS) has not yet been extended to walls sheathed with structural fiberboard. Results of testing showed that the empirical perforated shear wall equation establishes a conservative prediction of strength for the configurations tested.

On behalf of the many authors that contributed to this issue, I hope you find this information useful. Your comments and questions are welcome.

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Cyclic Testing of Unblocked Wood-Frame Wood Structural Panel Shear Walls

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Abstract

Design provisions for unblocked wood-frame wood structural panel shear walls were first introduced into the Special Design Provisions for Wind and Seismic (SDPWS) in the 2008 edition. In the work summarized by this paper, cyclic data from unblocked shear wall tests is compared against reference seismic equivalency parameters derived from tests of blocked wood structural panel shear walls. Data from 18 wall tests covering the extremes of permissible SDPWS configurations for unblocked wood structural panel shear walls are evaluated. While most of the tests were conducted on shear walls with 8d common sheathing nails, walls with 10d common sheathing nails were also tested. With a proposed 10% reduction in the design capacity for walls that employ 10d common sheathing nails, the seismic equivalency parameters from unblocked walls collected as part of this study are consistent with the range of parameters from reference blocked wood structural panel shear walls.

Introduction

Design provisions for unblocked wood-frame wood structural panel shear walls were introduced into the Special Design Provisions for Wind and Seismic (SDPWS) in the 2008 edition based on review of unblocked wall data from various sources (Ni. and Karacabeyli 2002, Mi et al. 2006, APA 1999). The data included various sizes and configurations ranging from 8 ft x 8 ft to 16 ft by 16 ft, tested at different times, and using different test methods with both cyclic and monotonic loading protocols. While extensive, the unblocked shear wall data was not directly comparable to seismic equivalency parameters for blocked wood structural panel shear walls which were developed based on the ASTM E 2126 Method C ("CUREE") cyclic load protocol (Line et al. 2008).

A total of 18 unblocked shear walls were tested in this study to establish monotonic and cyclic performance data. Cyclic loading was acquired using the CUREE load protocol. Test specimens consisted of unblocked wall configurations 9 ft in height and 8 ft long. The sheathing nail and stud spacing covered the full range of permitted unblocked configurations in accordance with SDPWS. The majority of testing was conducted in one laboratory; however, supplemental tests were conducted at a second laboratory. Testing summarized herein was under taken as part of a collaborative effort between American Wood Council, Weyerhaeuser, and APA-The Engineered Wood Association. The primary purpose of the testing was to develop data for unblocked wall configurations using CUREE cyclic loading to enable direct comparison to the existing blocked wall database.

Test Method

For cyclic loading, in-plane racking tests were conducted in accordance with the Method C ("CUREE") load protocol of ASTM E2126 Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings (ASTM 2011). For monotonic loading, in-plane racking tests were conducted in general accordance with ASTM E564 Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings (ASTM 2006). The primary exceptions were that a continuous, monotonic in-plane shear load was applied and a single replicate was tested for each configuration. These deviations were judged appropriate given the exploratory nature of this investigation and the large number of accompanying cyclic tests. Specific test specimen configurations are described in Table 1.

Table 1. Unblocked Shear Wall Test Specimen Configuration

Test Group	*n*	Lab	Test	Sheathing Nails: Common nail size	Sheathing Nails: Spacing (Edge/ Field), inch	Stud Spacing, inch	ASD Design Unit Shear, plf	Hold-down: No. Screws	Hold-down: ASD Capacity, plf	Wall Configuration
A1	2	A	Cyclic	10d	6 / 6	12	340	12	3360	Figure 1a
A2	2	A	Cyclic	10d	6 / 6	12	340	12	3360	Figure 1b
B	2	A	Cyclic	8d	6 / 6	12	280	12	3360	Figure 1a
B	2	A	Monotonic	8d	6 / 6	12	280	12	3360	Figure 1a
C	2	A	Cyclic	8d	6 / 12	12	224	9	2520	Figure 1a
C	1	A	Monotonic	8d	6 / 12	12	224	9	2520	Figure 1a
D	3	A	Cyclic	8d	6 / 12	24	112	4	1120	Figure 1c
D	2	B	Cyclic	8d	6 / 12	24	112	4	1120	Figure 1c
D	1	A	Monotonic	8d	6 / 12	24	112	4	1120	Figure 1c
E	2	A	Cyclic	8d	6 / 6	24	140	5	1400	Figure 1c

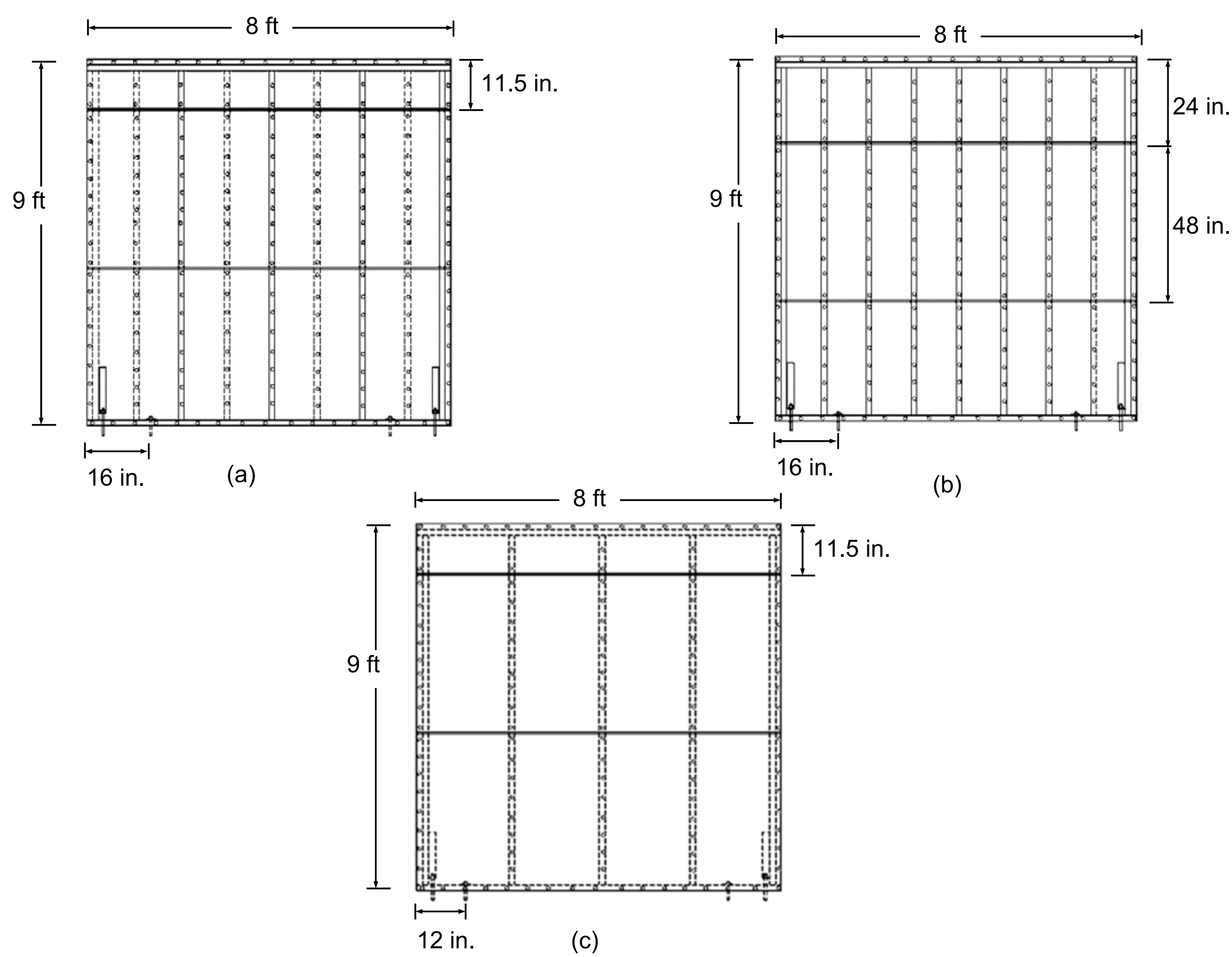

Figure 1. Unblocked Shear Walls, a) 12 in. stud spacing and minimum panel width of 11.5 in., b) 12 in. stud spacing and minimum panel width of 24 in., and c) 24 in. stud spacing and minimum panel width of 11.5 in.

In addition to the basic wall configuration details, Table 1 also provides the “ASD design unit shear” loads for each test wall. These allowable in-plane unit shear loads represent the SDPWS reference seismic design value multiplied by the appropriate unblocked shear wall adjustment factor provided in SDPWS.

Specimens

Figure 1 depicts wall size, sheathing layout, and anchor bolt locations for the unblocked wall specimens. Detailed drawings were developed for each wall configuration and used by each laboratory to fabricate test walls.

All framing was “standard” grade 2x4 nominal Douglas-fir. All end studs were built-up (2) 2x members joined using self-drilling ¼ in. diameter x 3 in. screws. A single commercial “low deformation” hold-down was used for all testing. The number of screws in the hold-down and built-up end studs varied such that the ASD design capacity of the hold-down-to-post connection was matched as closely as possible to the calculated ASD shear wall overturning forces.

All of the wall sheathing was 15/32 in. thick “Structural 1” oriented strand board (OSB) produced in accordance with Performance Standard for Wood-Based Structural Use Panels (PS 2) (DOC-NIST 2004). This relatively high grade of sheathing was chosen to maximize the targeted shear loads. Panel edge distance for the sheathing nails was a 3/8 in. minimum for all configurations.

Anchor bolts were 5/8 in. diameter with 3 x 3 x 0.229 in. plate washers provided between the bottom plate and the nut. Two anchor bolts were used in each wall test and were located 12 in. from wall ends for specimens with 24 in. o.c. stud spacing and 16 in. from wall ends for specimens with 12 in. o.c. stud spacing.

As illustrated in Figure 1a and 1c, most of the walls were tested using two full horizontal sheathing panels and a single 1ft. wide horizontal strip (exact dimension of panel strip is 11.5 in. as shown in Figure 1a and 1c). This configuration, when paired with the 9 ft. wall height, provided: two horizontal joints per specimen, one sheathing panel that was completely unblocked, and relied upon a relatively narrow 1 ft. wide horizontal sheathing strip. Each of these items were judged to be conservative and consistent with the SPDWS requirement that full sheets of sheathing are to be used except at “boundaries and changes in framing.” Group A2 was tested using a 2 ft. wide sheathing strip at the top of the wall, as shown in Figure 1b, to explore if this variable proved to be significant.

The test specimens were detailed to provide wall designs in accordance with 2008 SDPWS. The anchorage details (hold-downs and anchor bolts) were matched to allowable in-plane design capacity for a given unblocked wall configuration. Care was taken in detailing so that the bottom plate shear anchorage and overturning connections were not significantly over-designed.

Test Results

Table 2 summarizes the test results. The cyclic test data was analyzed in accordance with ASTM E2126 and in a manner consistent with methods used to analyze the reference wall database for blocked wood structural panel shear walls (Waltz et al. 2008). The summary data derived from the average backbone curve as described in ASTM E2126 is shown in Table 2. An example load displacement hysteresis curve with the average backbone curve is shown in Figure 2. Load displacement curves for each wall type are shown in Figure 3. The y-axis in these figures represents the ratio of load to ASD seismic design load of the wall configuration and the x-axis represents top of wall displacement in inches.

The reported drift capacity, component overstrength, and ductility contained within Table 2 are defined as follows:

- Drift Capacity: The “ultimate” displacement in accordance with ASTM E2126. This parameter is expressed as a percentage of the wall height.
- Component Overstrength: The peak load capacity of the wall divided by the allowable stress design load. This parameter is unitless.
- Ductility: The “ultimate” displacement divided by the displacement at the allowable stress design load. This parameter is unitless.

The primary failure modes observed were nail withdrawal from the framing and sheathing edge tear-out. Observed failures generally involved a combination of modes that led to loss of shear capacity in the test specimens. In 4 out of 18 tests where 10d sheathing nails were used, localized splitting of the studs was observed at the sheathing fasteners along the unblocked sheathing joints in combination with other failure modes. For test specimens with smaller diameter fasteners, similar localized stud splitting damage was not present.

Table 2. Unblocked Shear Wall Test Results

Test Group	n	Load Type	ASD		Peak		Ultimate		Drift Capacity, %ht	Component Over-Strength	Ductility	Primary Failure Mode[1]
			Load, lbf	Disp., in	Load, lbf	Disp., in.	Load, lbf	Disp., in.				
A1	2	Cyclic	2,720	0.312	5,738	1.840	4,590	3.093	2.9%	2.1	10.1	T, W, S
A2	2	Cyclic	2,720	0.332	6,050	2.185	4,848	4.414	4.1%	2.2	13.3	T, W, S
B	2	Cyclic	2,240	0.240	5,788	2.594	4,630	3.899	3.6%	2.6	16.2	T, W
B	1	Mono	2,240	0.278	6,440	3.106	5,152	4.618	4.3%	2.9	16.6	T, W
C	2	Cyclic	1,792	0.163	4,455	1.817	3,564	2.993	2.8%	2.5	18.5	T, W
C	1	Mono	1,792	0.232	4,622	2.687	3,698	3.847	3.6%	2.6	16.6	T, W
D	3	Cyclic	896	0.114	2,777	1.803	2,222	2.845	2.6%	3.1	25.1	T, W
D[2]	2	Cyclic	896	0.118	2,364	1.625	1,892	2.798	2.6%	2.6	23.9	—
D	1	Mono	896	0.148	2,973	2.148	2,378	3.212	3.0%	3.3	21.7	T, W
E	2	Cyclic	1,120	0.144	3,367	2.540	2,693	3.485	3.2%	3.0	24.2	T, W

[1] Failure Description: W—sheathing nail withdrawal, T—sheathing nail edge tearout of panel, S—localized stud splitting.

[2] Tests performed by separate laboratory.

Comparison of Unblocked Shear Wall Data to Reference Data

Summary statistics for the unblocked walls in this study are shown in Table 3 and compared to the seismic equivalence parameters for blocked wood structural panel shear walls determined from an 80 wall data base (Line et al. 2008) identified as "Reference data".

Taken as a whole, Table 3 suggests a significant overlap for each of the three quantitative parameter distributions between the reported unblocked and reference data. The mean and minimum values for each parameter are reasonably aligned between the unblocked wall test data from this study and the blocked wall reference database.

While the overall statistics for unblocked walls generally suggest similar performance, there are some differences between populations. For example, the average overstrength measured for this sampling of unblocked walls was about 13% lower than the reference data. Test Groups A1 and A2 contributed disproportionately to this difference. Localized stud splitting associated with the close-proximity of 10d common nails immediately across the unblocked sheathing joints contributed to reduced over-strength values relative to tests utilizing the smaller diameter 8d common nails.

Seismic performance equivalency criteria for new structural systems have been established which require new systems to perform at a level that meets or exceeds the "mean minus 1 standard deviation" level defined by the reference wood structural panel database (Line et al. 2008). If the average performance of each tested wall configuration meets or exceeds this level for each of the three quantitative cyclic test parameters and the system failure under lateral load does not significantly degrade the vertical load carrying capability of the wall, then it is assumed that the new system can be assigned the seismic performance design parameters normally associated with wood structural panel sheathing.

Table 3. Comparison of Unblocked Cyclic Data and Reference Cyclic Data

Statistic	Unblocked Wall Data			Reference Data			Ratio of Unblocked Data to Reference Data		
	Drift Capacity (% h)	Component Over-Strength	Ductility	Drift Capacity (% h)	Component Over-Strength	Ductility	Drift Capacity (%h)	Component Over-Strength	Ductility
n		15			80			15/80	
Maximum	4.6	3.22	27.04	5.5	5.4	43.4	0.84	0.60	0.62
Minimum	2.3	2.1	8.5	2.3	2.1	6.4	1.00	1.00	1.33
Average	3.1	2.6	19.2	3.4	3.0	20.1	0.91	0.87	0.96
COV	0.191	0.141	0.305	0.196	0.226	0.429	0.97	0.62	0.71
Avg. + 1 STD	3.7	3.0	25.0	4.1	3.7	28.8	0.90	0.81	0.87
Avg.—1 STD	2.5	2.3	13.3	2.8	2.3	11.5	0.89	1.00	1.16

As suggested by comparison between the cyclic test parameters of Table 2 and the reference blocked wall database summarized in Table 3, there were two wall configurations that would not have entirely satisfied this criterion. In the first case, the average overstrength of the 10d common A1 walls was calculated as 2.1, which did not exceed the 2.3 value associated with the reference database. Likewise, Group A1's ductility parameter average value was calculated as 10.1, falling below the 11.5 level suggested by the reference database. Altering the sheathing layout for Group A2 provided some improvement in performance; however the average overstrength factor of 2.2 still fell below the value of 2.3 representing the mean minus 1 standard deviation from the reference database.

To address these issues, it is recommended that the unblocked shear wall design adjustment factors in the SPDWS be revised to reflect a 10% reduction in design capacity for unblocked shear walls using 10d common nails for sheathing attachment. With this alteration in design capacity as shown in Table 4, the average measured component overstrengths for Groups A1 and A2 are 2.3 and 2.5 respectively, and meet the mean minus standard deviation overstrength value from the reference blocked wall database. Also, the ductility parameter from Groups A1 and A2 are 12.5 and 15.8 respectively, and meet the mean minus standard deviation ductility value from the reference blocked wall database.

The drift capacity of the Group D walls also appeared to slightly underperform the reference targets. At both laboratories, the average drift capacity of this dataset was 2.6%. The minimum target level associated with the mean minus one standard deviation performance of the reference database is 2.8%. This 0.2% difference for only one out of the five unblocked shear wall configurations in this study is judged to be insignificant in the context of an assembled wood frame structure, and considering that no Group D replicate at either laboratory demonstrated a drift capacity below the minimum value of 2.3% associated with the benchmark database. Taken together, these findings suggest that the drift capacity of the Group D walls reasonably aligned with the performance expectations typically associated with blocked wood structural panel shear walls.

Table 4. Unblocked Shear Wall Test Results—Group A1 and A2 with 10% Design Load Reduction to Account for the Use of 10d Common Nails

Test Group	n	Load Type	ASD		Peak		Ultimate		Drift Capacity (%h)	Component Over-Strength	Ductility	Primary Failure Mode
			Load, lbf	Disp., in.	Load, lbf	Disp., in.	Load, lbf	Disp., in.				
A1	2	Cyclic	2,448	0.253	5,738	1.840	4,590	3.093	2.9%	2.3	12.5	T, W, S
A2	2	Cyclic	2,448	0.257	6,050	2.185	4,848	4.414	4.1%	2.5	15.8	T, W, S

Sheathing Placement

As described above, the primary difference between Groups A1 and A2 was the width of the sheathing strip installed at the top of the wall. Group A1, and all of the remaining wall tests, were undertaken using a 1 ft. strip of sheathing along the top of the wall. In almost all of these tests, most of the wall distortion and progressive deterioration seemed to radiate from the top horizontal sheathing joint.

Group A2 was tested to see if a wider strip of sheathing at the top of the wall would improve shear transfer into the studs and improve performance. This change created a more balanced wall failure mode with more of the progressive damage occurring lower in the wall. As indicated in Table 2, the quantitative performance was also somewhat improved. The average drift capacity increased from 2.9 to 4.1% and the average ductility improved from 12.5 to 15.8. This also provides some further assurance that the configuration tested for Group D was likely conservative with respect the potential for drift capacity relative to other possible unblocked wall configurations. It also suggests that improved wall performance can be achieved if small strips of sheathing are avoided whenever possible.

Recommendations for Future Testing

Localized stud splitting was consistently observed at unblocked adjoining panel edges where 10d nails were used. The splitting is attributed to several factors: i) the close proximity of adjacent nails across the unblocked joint, ii) higher shears associated with 10d nailed construction, and iii) tension perpendicular to grain stresses induced in the stud at the unblocked joint. Unblocked specimens were fabricated with a 3/8 in. minimum distance to the sheathing edge resulting in spacing of nails across the joint of approximately 7/8 in.. Greater spacing of adjacent 10d nails across the unblocked joints on the order of 10-15 diameters (1.5-2 in.) may reduce the occurrence of localized splitting and improve over-strength performance due to both avoidance of splitting and improved sheathing tear out strength.

Summary

Unblocked shear walls constructed with 8d common nails exhibit cyclic performance including drift capacity, component overstrength, and ductility similar to the reference dataset of 80 blocked wood structural panel shear walls and can reasonably be assigned similar seismic performance factors. Unblocked shear walls constructed with 10d common nails failed to meet the target performance levels based on current design requirements. However, a recommended allowable design load reduction of 10% applied to walls constructed with 10d common nails, would enable all of the unblocked wall configurations tested in this study to meet the cyclic target performance levels associated with blocked wood structural panel shearwalls. Wall specimens sheathed with minimum 2 ft. panel width exhibited improved drift capacity, overstrength, and ductility performance relative to wall specimens sheathed with a minimum 1 foot panel width.

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In-Plane Racking Strength Tests of Wood-Frame Continuous Sheathed Wood Structural Panel Wall Bracing In Accordance With ICC-ES Acceptance Criteria AC269.1

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Abstract

This paper summarizes results from a series of in-plane racking tests of wood structural panel sheathed walls conducted in accordance with the continuous sheathed wood structural panel (CS-WSP) braced wall provisions of ICC Evaluation Service's Acceptance Criteria AC269.1: Acceptance Criteria for Proprietary Sheathing Attached to Wood Light-Frame Wall Construction Used and Braced Wall Panels Under the IRC. The objective of the testing was to evaluate the appropriateness of the criteria's performance targets for determining whether a proprietary sheathing performs in a manner consistent with CS-WSP bracing. A total of 21 racking tests were conducted in two separate laboratories. Testing included racking tests on ASTM E 72 walls and the CS-WSP "baseline," "corner return", and perforated wall configurations. Results show that test-based strengths for the wood structural panel walls in the perforated wall configurations exceeded the criteria's strength performance targets by 20% to 63%. While the stiffness of the perforated wall configurations largely agreed with current design expectations, these results did not always satisfy AC269.1's current minimum stiffness criteria and varied significantly between laboratories. Recommendations for revision of the acceptance criteria have been developed.

Introduction

The "continuous sheathed wood structural panel" (CS-WSP) wall bracing method in the International Residential Code (IRC) (ICC 2009) is favored by designers due to its high strength and stiffness that result in reduced lengths for individual bracing segments and smaller total lengths of required bracing relative to other bracing methods. Also, end restraint options available for the CS-WSP bracing option include details that permit 2 ft. long return corner walls to be used in lieu of hold-down anchorage for wall segments less than 48 in.

Due in part to the attractive attributes of the IRC's CS-WSP bracing provisions, ICC-Evaluation Service has developed acceptance criteria for proprietary sheathing products to gain recognition for use in CS-WSP braced wall applications. AC269.1, Acceptance Criteria for Proprietary Sheathing Attached to Wood Light-Frame Wall Construction Used and Braced Wall Panels Under the IRC (AC269.1) (ICC-ES 2013) provides criteria intended to provide alternative proprietary sheathing panel manufacturer with a means to evaluate whether their product performs in a manner consistent with CS-WSP bracing. The criteria include a series of in-plane wall racking tests that address a range of different boundary conditions and wall configurations and require the proprietary product to meet or exceed a series of performance targets to gain recognition as a CS-WSP bracing substitute.

The IRC's CS-WSP bracing provisions are not based on tests of the specific CS-WSP bracing configurations described in AC269.1. In the absence of a test basis for the specific perforated wall configurations, performance targets for perforated wall configurations in AC269.1 are based on the perforated shear wall calculation method summarized in the American Wood Council's Special Design Provisions for Wind and Seismic (SDPWS 2008). However, the perforated shear wall calculation method is intended to provide conservative design strengths and has generally been assumed to under-predict actual test-based strengths. As a result, using the calculation method may result in establishment of non-conservatively low strength performance targets for the recognition of alternative sheathing products.

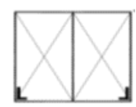
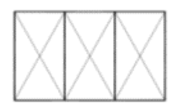
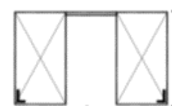
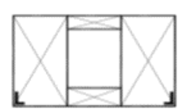
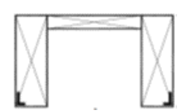
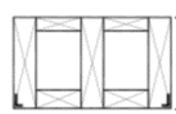
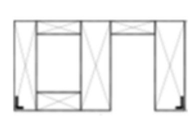
Testing summarized herein was undertaken as part of a collaborative effort between American Wood Council, Weyerhaeuser, and APA-The Engineered Wood Association for the purposes of documenting the in-plane racking performance of WSP sheathed walls in the specific CS-WSP bracing configurations described in AC269.1. In-plane racking tests were conducted on WSP sheathed walls in two separate laboratories. One laboratory conducted the ASTM E 72 wall racking tests as described in AC269.1. Both laboratories conducted tests on a full series of the CS-WSP bracing configurations described in AC269.1, including the baseline wall, corner return wall, and perforated walls. Each laboratory tested three baseline walls and either one or two walls for each additional configuration. A total of 21 walls were tested.

Test Method

Testing was conducted in general conformance with requirements of AC269.1 for the wall configurations depicted in Table 1. For Wall Types 1-7, racking tests were in accordance with AC269.1's modifications to ASTM E 564 Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings (ASTM, 2006). The applied shear loading at both laboratories was in compression and directed from left to right based on orientation of walls depicted in Table 1.

In addition to testing of Wall Types 1 - 7, ASTM E 72 wall racking tests were also conducted in accordance with AC269.1's modifications to ASTM E72 *Standard Test Methods of Conducting Strength Tests of Panels for Building Construction* (ASTM 2010).

Table 1. Test Wall Configuration for an AC269.1 Evaluation of CS-WSP Bracing

Wall Type	Description	Wall Size Height x Length	Clear Opening Height, %H	Sheathed Segment Aspect Ratio (H/L_s)	Full-Height Sheathed Length	Hold-down?	Wall Configuration
--[1]	ASTM E 72	8 ft x 8 ft	-	1:0	8 ft.	Yes	-
1	Baseline	8 ft x 8 ft	-	1:0	8 ft.	Yes	
2	Corner Return	8 ft x 12 ft	-	1:1.5	12 ft	No	
3	Full-Height Opening	8 ft x 12 ft	100%	2:1	8 ft	Yes	
4	Window Opening	8 ft x 12 ft	85%	2:1	8 ft	Yes	
5	Door Opening	8 ft x 13.3 ft	85%	3:1	5 ft-4 in	Yes	
6	Two-window Opening	8 ft x 14 ft	65%	4:1	6 ft	Yes	
7	Window & Door Opening	8 ft x 15.3 ft	65% Window 85% Door	4:1, 3:1	7 ft-4 in	Yes	

[1] Wall racking tests conducted in accordance with AC269.1 modifications to ASTM E72 to address the capacity of the sheathing and sheathing-to-framing attachment.

Specimens

The ASTM E 72 walls were fabricated in accordance with AC269.1's modifications. Wall Types 1-7 were fabricated in general conformance with the available requirements and details outlined in AC269.1. To minimize differences in interpretation of wall construction details for specific wall assemblies of AC 269.1, detailed drawings were developed for each wall configuration and used by each laboratory to fabricate test walls. The detailed drawings removed potentially different judgments between laboratories for wall construction details that could influence the measured performance such as minimum anchor capacity, exact anchor bolt placement, corner stud attachment, framing nail type and placement. Figure 1 depicts framing placement and anchor bolt locations for Wall Type 1. Figure 2 illustrates a typical test wall in the test frame used by each test laboratory. Other relevant materials and details of construction used in fabrication of the Wall Type 1-7 test specimens were as follows:

- ***Framing:*** Studs and plates were 2x4 nominal Douglas-Fir "Standard or Better" grade. Stud spacing was 16 in. o.c. except where wall configurations required smaller stud spacing adjacent to openings. All end studs where hold-downs were used were built-up (2) 2x members. Headers were single-ply 2x12 nominal Douglas-Fir No. 2 Grade. Headers were supported at each end by one jack stud. All of the stud and plate framing that received perimeter WSP nailing was pre-screened to ensure that the average oven-dry specific gravity was 0.50 ± 0.03.

- ***Sheathing:*** All WSP sheathing used by both laboratories was 3/8 in. oriented strand board (OSB) obtained from the same single bundle that was purchased on the open market and produced in accordance with Performance Standard for Wood-Based Structural Use Panels PS-2 (DOC-NIST 2010).

- ***Sheathing nails:*** Sheathing nails were 6d common (2.0 x 0.113 in.) fasteners spaced at 6 in. o.c. at panel edges and 12 in. o.c in the field of the panel. The fasteners were installed to maintain a 3/8 in. minimum edge distance at all OSB panel perimeters. The nails used by both laboratories were manufactured by the same nail manufacturer.

- ***Framing nails:*** Framing nails were installed in accordance with prescribed minimum nailing from the IRC unless otherwise noted. Headers were toe-nailed to full-height studs at each end using (4) 8d box nails (2.5 x 0.113 in.). Top plate to header nailing consisted of 3 x 0.131 in. nails at 24 in. o.c. Window sills were end-nailed to studs using (2) 16d box (3.5 x 0.135 in.) at each end. Jack studs were nailed to king-post with 3 x 0.131 in. nails spaced at 24 in. o.c.

- ***Anchor bolts:*** Anchor bolts were 5/8 in. diameter with 3 x 3 x 0.229 in. square plate washers between the bottom plate and the nut. Anchor bolts were spaced at 24 in. o.c. An anchor bolt was located 12 inches from ends of each bottom plate except for walls with openings where anchor bolts were located within 6 to 12 inch from each end of each bottom plate.

- ***Hold-downs:*** "HDQ8-SDS3" hold-downs were used for Wall Types 1, 3, 4, 5, 6 and 7. In all cases, eight screws attached each hold-down to (2) 2x end studs. The number of screws used for overturning anchorage attachment was determined such that the hold-down-to-end stud connection was only slightly greater in strength than the wall's allowable stress wind design overturning force of 2,240 lbf. The overturning force represents the wind design allowable unit shear value for Wall Type 1 times the wall height of 8 ft (e.g. 280 plf x 8 ft = 2,240 lbf). Hold-downs in Wall Types 3, 4 5, 6, and 7 are sized for this same unit shear force to enable the tension side end panels to develop the same unit shears as associated with Wall Type 1. Nailing between the two-ply end studs consisted of (15) 3 x 0.131 in. nails evenly-spaced to match the ASD shear wall overturning force.

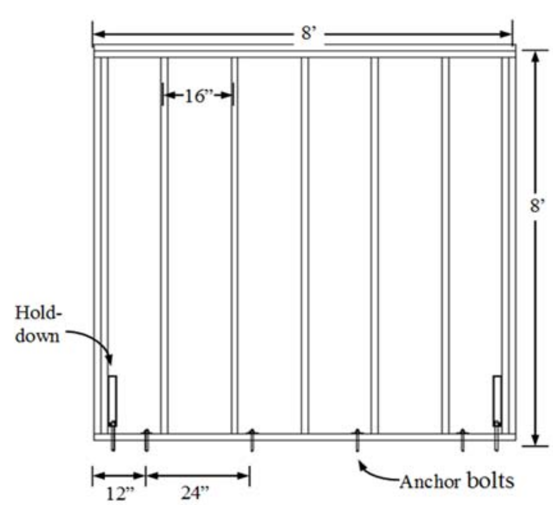


Figure 1. Example Drawing for AC269.1 Wall Type 1 Specimen

(a) **(b)**

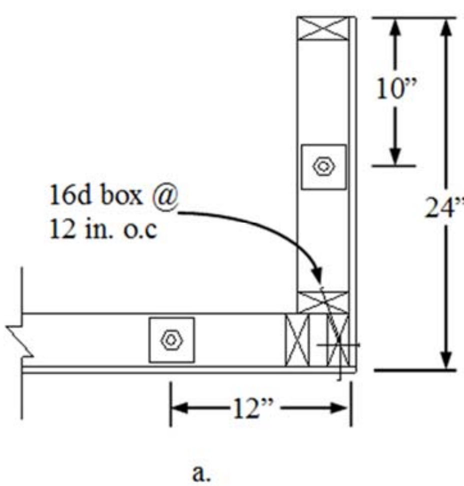
Figure 2: a) Wall Type 7 Door and Window Specimen, b) Example Wall Type 2 Corner Return Specimen

The combination of 3/8 in. thick WSP sheathing and 6d common sheathing nails used for all of the test specimens in this study is associated with the minimum requirements for CS-WSP bracing described in the IRC. This same sheathing and attachment are also associated with the 560 plf minimum shear strength target that AC269.1 requires for the ASTM E 72 wall racking test specimens. This capacity target was based upon the 560 plf shear wall nominal unit shear for wind design that the SDPWS tabulates for the same combination of sheathing and sheathing nailing with framing that has a specific gravity of at least 0.50.

As illustrated in Table 1, all walls utilized a hold-down at ends except for Wall Type 2. Wall Type 2 was framed with 24 in. long sheathed corner returns which were used to provide alternative end restraint. Details of construction of the corner return walls used in this study are depicted in Figure 3. A three-stud corner with a 1-1/4" gap between adjacent studs was used to represent a typical framed corner in accordance with the IRC.

Test Results

Detailed test results are provided in Table A.1. Table 2 provides a summary of the strength-based criterion of



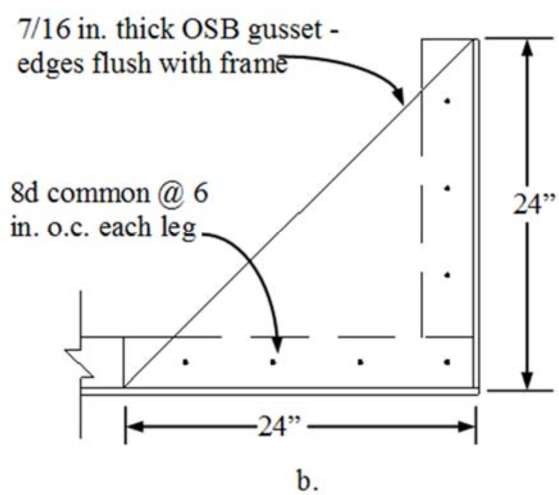


Figure 3. Wall Type 2 Corner Return, a) Location of Bottom Plate Anchor Bolts, and b) Attachment of Triangular OSB Gusset to Wall Top Plates.

interest in this study. Load deflection curves for Wall Types 1-7 from Series A data are shown in Figure 4 as an example of typical load deflection behavior. Load deflection curves for the ASTM E 72 walls are shown in Figure 5. In each summary table and figure, the data has been divided into "Series," with each Series representing the test data from one of the two laboratories involved in the test program. It should be noted that in Figure 4, the x-axis represents the racking deflection at the top of wall in inches. In figure 5, per ASTM E 72, the x-axis represents the "net" racking deflection at the top of the wall with the rigid body rotation and translation components of deflection removed. Table 2 and the y-axes of Figure 4 and Figure 5 provide the measured racking strength as a normalized ratio calculated in accordance with Equation 1:

$$\textit{Strength Ratio} = \frac{(\textit{load unit shear})}{(\textit{Peak load unit shear})_{Baseline}}$$

where:

(load unit shear) = Load for the wall configuration of interest divided by the total wall length, plf

(Peak load unit shear)$_{Baseline}$ = Average peak load unit shear for Wall Type 1 (i.e. Average peak load of Wall Type 1 divided by length of 8 ft.), plf

Failure modes included a combination of nail withdrawal from the framing, nail heads pulling through the thickness of the sheathing (commonly referred to as "nail head pull through"), and sheathing edge tear-out. Bearing failures at panel edges were observed at the corners of walls with openings. Panel buckling and panel shear failures were not observed. All studs were judged to be intact and capable of supporting gravity loads at the conclusion of the test.

Evaluation of Measured Strength Parameters

ASTM E 72 Pre-Qualification

AC 269.1's pre-qualification requirements for CS-WSP bracing recognition require the sheathing and sheathing attachment to achieve a peak shear capacity of at least 560 plf when tested in general accordance with ASTM E 72 using Douglas-fir framing. In addition, the system must demonstrate racking loads of at least 200 plf and 400 plf at net deflections of 0.2 and 0.6 in., respectively. Review of Table 2 and Table A.1 shows that WSP sheathed walls in this study satisfied these targets. The average peak shear capacity of 666 plf was 19% greater than the minimum peak unit shear capacity requirement of AC269.1. The 200 and 400 plf deflections averaged 0.05 and 0.37 in., respectively. These findings confirm that the WSP sheathed walls met the pre-qualification requirements.

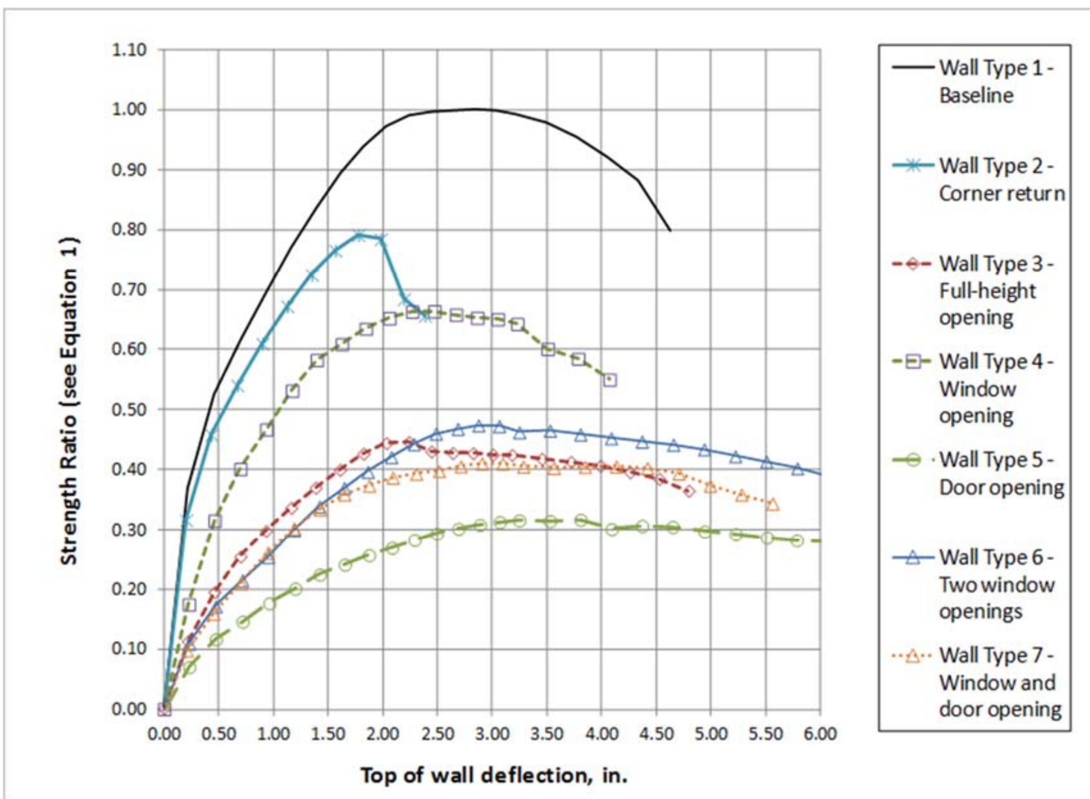


Figure 4. Series A Load-Deflection Curves for Wall Types 1-7.

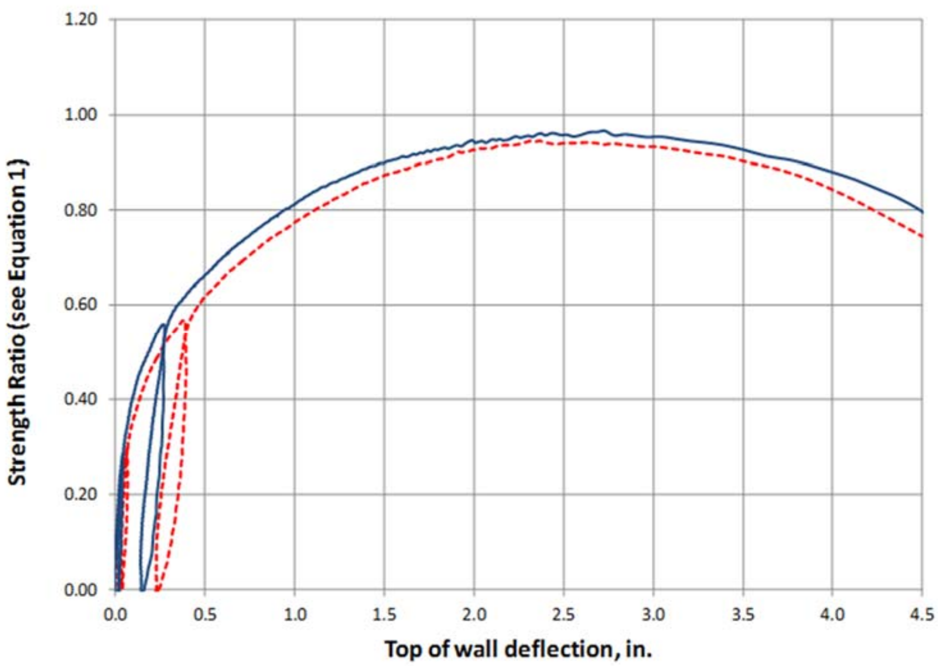


Figure 5. Series A Load-Deflection Curves for ASTM E72 Walls

Continuous Sheathed Baseline (Wall Type 1):

Once the pre-qualification requirements have been satisfied, additional CS-WSP bracing criteria of AC269.1 are applicable and include testing of Wall Type 1. Wall Type 1 serves as a baseline used for relative evaluation of the other six specific wall types with end returns and openings. Acceptability is based upon how well those walls perform compared to the performance of Wall Type 1.

While the performance of Wall Type 1 becomes critical for the review, AC269.1 does not currently impose minimum strength or stiffness targets. If Wall Type 1 were to be tested with weak

Table 2. Summary of Peak Load Test Results.

Wall Type	Wall Description	Data Series A		Data Series B		Combined Data From Series A and B	
						Strength Ratio	
		Peak Load (lbf/plf)	Normalized Strength Ratio	Peak Load (lbf/plf)	Normalized Strength Ratio	Average	Avg. –1 STD (Lower Bound)
—	ASTM E72	5327/ (666)[1]	0.96	-	-	-	-
1	Baseline	4988/ (624)		5267/ (658)			
		6066 / (758)		5796/ (725)			
		5662/ (708)		4717/ (590)			
	Average:	**5572/ (697)**	**1.00**	**5260/ (658)**	**1.00**	**1.00**	**-**
	COV:	**0.098**		**0.103**			
2	Corner Return	6820/ (568)	0.82	6138/ (512)	0.78	0.80	-
3	Full-Height Opening	3793/ (316)	0.45	3985/ (332)	0.51	0.48	0.43
4	Window Opening	5659/ (472)	0.68	6111/ (509)	0.77	0.73	0.66
5	Door Opening	2994/ (225)	0.32	2836/ (213)	0.32	0.32	0.29
6	Two Window Opening	4694/ (335)	0.48	3985/ (285)	0.43	0.46	0.41
7	Window & Door Openings	4465/ (291)	0.42	4276/ (279)	0.42	0.42	0.38

[1] Represents Average of 2 Tests.

Table 3. Comparison of Reference and Test-Based Strength Ratios

Wall Type	Description	Reference Strength Ratio	Test-Based Strength Ratio (Average)	Test-Based Peak Strength Ratio (Lower Bound)	Average / Reference	Lower Bound / Reference
1	Baseline	1.00	1.00	-	1.00	-
2	Corner Return	0.79	0.80	-	1.01	-
3	Full-Height Open-ing	0.40	0.48	0.43	1.20	1.08
4	Window Opening	0.51	0.73	0.65	1.42	1.27
5	Door Opening	0.21	0.32	0.29	1.54	1.38
6	Two-window Opening	0.28	0.46	0.41	1.63	1.46
7	Window & Door Opening	0.29	0.42	0.38	1.45	1.31

[1] Reference strength ratios for walls with openings (Wall types 3, 4, 5, 6, and 7) are calculated in accordance with the perforated shear wall strength ratio equation: F=r/(3-2r) where r = sheathing area ratio (see Commentary to AWC, 2008). Reference strength ratio is called the "reduction factor" in AC269.1 and used in both strength and stiffness evaluations of CS-WSP bracing.

[2] Lower bound values based on average minus one standard deviation estimates from test data where COV of baseline wall data was 0.10.

anchorage or other detailing, it is possible that a non-conservative review for the remaining configurations may result. For Series A, the minimum peak unit shear is 624 plf. For series B, the minimum peak unit shear is 590 plf. For all Wall Type 1 walls in both series A and B, average top of wall deflection did not exceed 0.200 inch at a unit shear of 200 plf and 0.600 inch at a unit shear of 400 plf. To avoid a non-conservative review for alternative proprietary product, it may be appropriate for AC269.1 to impose the ASTM E 72 wall strength and stiffness requirements upon Wall Type 1. WSP sheathed walls tested in this study would have supported this suggested minimum performance level.

Continuous Sheathed Wall Comparisons:

A comparison of test-based peak strength ratios in accordance with Equation 1 and the current minimum "reference" strength ratios required by AC269.1 is provided in Table 3. Column 3 provides minimum required reference strength ratios assigned by AC269.1 to each wall configuration. Column 4 provides average test-based strength ratios at peak load. For comparison purposes, Column 6 provides the ratio between Columns 4 and 5. Values greater than 1.0 indicate that test-based strengths exceed the reference strength targets. From Column 6, the average test-based strengths in this study exceeded the reference calculation-based strengths for the perforated configurations by varying margins ranging from 1.20 to 1.63. These strength values suggest that the average WSP bracing performance measured in this study for the walls with perforations was 20-63% greater than the current minimum targets in AC269.1. This finding was not unexpected given that the AC269.1 targets were generated using the SPDWS perforated wall calculation method believed to be conservative for design purposes.

Column 5 provides newly proposed lower-bound – average minus one standard deviation – strength ratios for perforated wall configurations that might be considered for incorporation as potential new minimum targets for AC269.1. The standard deviation estimate used for all wall configurations was based upon an assumed 10% coefficient of variation (COV). The COV for the Wall Type 1 configuration obtained by combining the six replicates from Series A and B was 9.5%. Using a reasonable lower bound target has precedent in other ICC-ES acceptance criteria and provides some flexibility to account for the limited number of samples required as part of an AC269.1 evaluation. The average minus one standard deviation approach helps to account for the likelihood that any given small sampling may fall above or below a population average. From Column 7, the proposed lower bound test-based strengths would exceed the existing reference calculation-based strength ratios for walls with openings by varying margins ranging from 1.08 to 1.46.

Corner return wall (Wall type 2)

Unlike the reference strength ratio for the five perforated wall configurations, the required minimum reference strength ratio of 0.79 for the corner return wall used by AC269.1 was not based on the perforated shear wall calculation method. While it is based on prior testing; the specific factor of 0.79 is not directly prescribed in corner return wall test reports (Dolan, 1997 or HUD, 2001). It is also worth noting that AC269.1 does not currently specify the corner return framing configurations and that corner return details specified in the IRC have evolved with time. For corner return walls constructed in accordance with details reported herein, the average strength ratio is 0.80 which is considered to be in support of the continued use of the current reference strength ratio of 0.79 based on prior testing.

Stiffness/deflection evaluation

In addition to the minimum required relative strength, the CS-WSP bracing criteria of AC269.1 also contain stiffness targets that are numerically equivalent to the minimum strength ratios. Test-based stiffness ratios are determined in accordance with Equation 2:

$$\textit{Stiffness Ratio} = \frac{\left(\dfrac{\textit{Reference Load}}{\Delta \text{ @ } \textit{Reference Load}}\right)}{\left(\dfrac{\textit{Reference Load}}{\Delta \text{ @ } \textit{Reference Load}}\right)_{\textit{Baseline}}}$$

where:

Δ = top of wall deflection, in.

Test-based stiffness ratios determined at 40% of each wall's peak unit shear capacity (plf), as required by the current criteria, showed high variability and resulted in both large increases and decreases in stiffness targets relative to current AC269.1 values. Under the current AC269.1 approach, unit shear is calculated as load divided by the overall wall length. Test-based stiffness ratios evaluated at 40% of each wall's peak capacity (lbf) were also highly variable but met or exceeded current stiffness targets when data from series A and B were averaged. It is observed that measured deflections are both small in magnitude and can vary significantly between laboratories on a percentage basis. Also, while 40% of peak load is intended to approximate a region of generally elastic response, varying levels of non-linearity are present such that

Table 4. Comparison of Reference and Test Based Stiffness Ratios at the Reference Design Load

Wall Type	Description	Reference Ratio	Reference Design Load (lbs)	Data Series A Test-Based Stiffness Ratio	Data Series B Test-Based Stiffness Ratio	Combined Data from Series A and B	
						Test-Based Average Stiffness Ratio	Test / Reference
1	Baseline	1.00	2240	1.00	1.00	1.0	1.0
2	Corner Return	0.79	2655	1.11	0.73	0.92	1.16
3	Full-Height Opening	0.40	1345	0.33	0.71	0.52	1.29
4	Window Opening	0.51	1700	0.50	0.56	0.53	1.05
5	Door Opening	0.21	775	0.21	0.21	0.21	1.00
6	Two-window Opening	0.28	1090	0.33	0.16	0.25	0.89
7	Window & Door Opening	0.29	1245	0.31	0.20	0.26	0.88
						Overall Average	1.04

slight increases in load can lead to relatively large increases in deflection.

An alternative stiffness ratio analysis is provided in Table 4 where deflection is taken at a load more representative of an allowable "reference" design load and calculated as: 280 plf x wall length x the reference ratio. The alternative stiffness ratio analysis was investigated to evaluate stiffness ratio at load levels associated with a load level approximating an allowable stress design of the shear walls with openings in accordance with the perforated shear wall method. Results of the analysis are shown in Table 4. Such load levels are intended to represent the elastic response region of the load deflection curve, For Wall Types 3-7, the test-based stiffness ratios are observed to vary significantly between laboratories. When data series A and data series B are averaged, test-based ratios are observed to range from 0.88 to 1.29 times the reference ratios with an overall average ratio of 1.04. Based on averaged results for each configuration, the alternative analysis shows that the stiffness ratio from testing is greater than the reference ratio for Wall Types 3, 4 and 5 but less than the reference ratio for Wall Types 6 and 7. Where tested stiffness ratio was less than the reference ratio, the maximum difference was 12% for Wall Type 7. However, in general, the overall average test/reference ratio of 1.04 for the test program suggests reasonable agreement between the tested and predicted perforated wall stiffness at the design load level.

Findings and Recommendations

In addition to documenting results from testing wood structural panel sheathed walls in specific CS-WSP bracing configurations in AC269.1, information reported herein is intended to assist in further refinement of procedures for evaluation of equivalence to CS-WSP bracing. Findings and recommendations from this testing program include the following:

a) This test program confirmed that wood structural panel sheathing satisfies the racking pre-qualification requirements for strength and stiffness based upon an ASTM E72 racking test. The pre-qualification requirement provides for a standard evaluation of the sheathing and sheathing-to-framing attachment.

b) The strength and stiffness of Wall Type 1 is essential for establishing strength and stiffness ratios for CS-WSP bracing configurations in AC269.1. While testing indicates relative low strength variability for the baseline wall tests, additional clarification of requirements for baseline tests is recommended. This includes clarification of fabrication details, such as the minimum overturning anchorage capacity, and compliance of the baseline walls with minimum strength and stiffness requirements similar to that of the ASTM E 72 racking test provisions.

c) Results of testing specific wall configurations in AC269.1 confirm the conservatism of the perforated shear wall calculation method in SDPWS for estimating design strength. This conservatism for design; however, translated into minimum strength performance targets in AC269.1 that are non-conservative relative to the tested performance of wood structural panel sheathed walls.

d) Revised minimum strength performance targets for perforated shear wall configurations in AC269.1 are proposed using an "average minus one standard deviation" basis. These test-based strength performance targets for the perforated configurations ranged from 1.08 to 1.46 times the current strength performance targets. For corner return walls constructed in accordance with details reported herein, the average strength ratio is 0.80 which supports the continued use of the current reference strength ratio of 0.79 based on prior testing.

e) Additional construction details for fabrication of walls are recommended to improve consistency in results. Details that have the potential to impact the results include: location and installation of anchor bolts, measurement of corner return wall lengths, header framing size and support methods, and framing nailing guidance. Revised guidance for a minimum hold-down size that aligns the hold-down strength with the expected wall racking strength is also recommended.

f) Methods used for calculation of strength and stiffness ratios are currently dependent on footnoted information in AC269.1. Further clarification or re-organization of the calculation method used for evaluation of strength and stiffness is recommended.

g) In recognition of observed stiffness variability in Wall Types 2-7 and that only a single test of each of those configurations is required by AC269.1, it is proposed to remove stiffness performance requirements for Wall Types 2-7 provided that minimum strength and stiffness requirements are met for Wall Type 1. Alternatively, the stiffness targets for specific CS-WSP bracing configurations should be revised to reflect the results of this study based on the construction details in this study.

Summary

Testing of wood structural panel sheathed walls in specific CS-WSP bracing configurations described in AC269.1 was undertaken in two separate laboratories. Test-based strength performance of the CS-WSP bracing configurations with perforations are 20% to 63% greater than the existing calculation-based reference minimum strength performance targets used by AC269.1 for evaluating proprietary sheathing materials. While test data confirmed the expected conservatism of the perforated shear wall calculation method used to establish the existing targets, it also shows that AC269.1's strength performance targets based on calculations underestimate actual tested strengths of specific wall configurations. Alternative strength ratio performance targets ranging from 1.08 to 1.46 times current calculation-based levels for perforated wall configurations are suggested. Stiffness ratios based on deflection at 40% of peak load observed in this study did not mirror the strength ratios as assumed by AC269.1. The approach used by AC269.1 for the unit stiffness evaluation of Wall Types 2-7 should be replaced with alternative criteria or revised to reflect the results of this study.

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Table A.1. Detailed Data Summary

Wall Type	Ref / Item	Wall Size H by L (ft)	OSB (in)	Sheathing nails	Fastener Spacing (Edge / Field) (in.)	Openings?	Drift at 200 plf (in.)	Drift at 400 plf (in.)	40% Peak Load		Peak Load		Ultimate Load[1]		Gravity Load System Intact?	Failure[2]
									Load (lbf)	Drift (in.)	Load (lbf)	Drift (in.)	Load (lbf)	Drift (in.)		
-[3]	[A]/A21	8 by 8	3/8	6d com	6/12	No	0.059	0.425	2108	0.117	5270	2.368	4216	4.448	Yes	P, T, W
-[3]	[A]/A22	8 by 8	3/8	6d com	6/12	No	0.046	0.311	2153	0.088	5383	2.732	4306	4.587	Yes	P
1	[B]/1	8 by 8	3/8	6d com	6/12	No	0.262	0.404	1995	0.328	4988	3.165	3991	5.509	Yes	W, T, P
1	[B]/2	8 by 8	3/8	6d com	6/12	No	0.106	0.479	2426	0.235	6066	2.933	4853	4.884	Yes	P, W
1	[B]/3	8 by 8	3/8	6d com	6/12	No	0.143	0.563	2265	0.252	5662	2.973	4530	4.613	Yes	W, T, P
1	[C]/4	8 by 8	3/8	6d com	6/12	No	-	0.366	2107	-	5267	2.334	4214	3.825	Yes	P, W
1	[C]/5	8 by 8	3/8	6d com	6/12	No	0.041	0.224	2318	0.095	5796	2.755	4637	3.894	Yes	P, W
1	[C]/6	8 by 8	3/8	6d com	6/12	No	0.065	0.501	1887	0.102	4717	2.199	3773	5.027	Yes	P, T
2	[B]/7	8 by 12	3/8	6d com	6/12	No	-	-	2728	0.207	6820	1.888	5456	2.445	Yes	T at end return bottom
2	[C]/8	8 by 12	3/8	6d com	6/12	No	-	-	2472	0.112	6179	1.424	4943	1.956	Yes	T
2	[C]/9	8 by 12	3/8	6d com	6/12	No	-	-	2438	0.129	6096	1.880	4877	2.579	Yes	T
3	[B]/10	8 by 12	3/8	6d com	6/12	Yes	-	-	1517	0.405	3793	2.328	3035	4.881	Yes	T, P, W
3	[C]/11	8 by 12	3/8	6d com	6/12	Yes	-	-	1594	0.095	3985	1.552	3188	4.311	Yes	P, W
4	[B]/12	8 by 12	3/8	6d com	6/12	Yes	-	-	2263	0.364	5659	2.384	4527	4.196	Yes	B, P, T
4	[C]/13	8 by 12	3/8	6d com	6/12	Yes	-	-	2444	0.215	6111	2.219	4889	3.166	Yes	P, T
5	[B]/14	8 by 13.3	3/8	6d com	6/12	Yes	-	-	1197	0.561	2994	3.716	2395	6.514	Yes	P, T, B
5	[C]/15	8 by 13.3	3/8	6d com	6/12	Yes	-	-	1134	0.236	2836	2.260	2269	4.955	Yes	B, P, T
6	[B]/16	8 by 14	3/8	6d com	6/12	Yes	-	-	1878	0.564	4694	2.959	3756	6.222	Yes	B, T P
6	[C]/17	8 by 14	3/8	6d com	6/12	Yes	-	-	1594	0.428	3985	2.858	3188	4.833	Yes	T, B, P
7	[B]/19	8 by 15.3	3/8	6d com	6/12	Yes	-	-	1786	0.472	4465	3.034	3572	5.747	Yes	B, T P
7	[C]/19	8 by 15.3	3/8	6d com	6/12	Yes	-	-	1710	0.307	4276	2.570	3421	5.930	Yes	B, T P

[1]Point where the wall capacity is 80% of the peak.

[2]Failure description: W-sheathing nail withdrawal from framing, P – sheathing nail head pull-through the panel, T-sheathing nail edge tear-out of panel, B – bearing failure of sheathing panel at edges.

[3]These walls were tested using matched materials in accordance with the racking test method of ASTM E72. The reported drifts represent the net lateral deflection after rigid body rotation and translation have been analytically removed. The drift reported for the remaining walls represents the total lateral movement from all sources.

Appendix Table References:

[A] Waltz, Ned. “Benchmark Monotonic and Cyclic Tests of Fully-Restrained Wood Structural Panel Braced Walls,” Weyerhaeuser Engineering Laboratory Experiment No. 2379A. Weyerhaeuser, Boise, Idaho. 2012

[B] Waltz, Ned. “ICC-ES AC269.1 “Continuous Sheathing” Review of Oriented Strand Board,” Weyerhaeuser Engineering Laboratory Experiment No. 2407. Weyerhaeuser, Boise, Idaho. 2013.

[C] Keith, E., “APA Report T2013P-18 Wood Structural Panel Shear Wall Tests in Accordance with ICC-ES AC269.1,” APA-The Engineered Wood Association, Tacoma, WA. 2013.

In-Plane Racking Strength Tests of Wood-Frame Structural Fiberboard Perforated Shear Walls

Louis Wagner, Philip Line, PE, Deepak Shrestha, PE

Abstract

This paper summarizes results from a series of in-plane racking tests of structural fiberboard perforated shear walls. The objective of the testing was to evaluate the applicability of the perforated shear wall design provisions of Special Design Provisions for Wind and Seismic (SDPWS) for estimation of the design strength of perforated shear walls sheathed with structural fiberboard sheathing. A total of 9 racking tests were conducted. Testing included racking tests on fully-sheathed "baseline," "corner return", and perforated wall configurations. Results show that test-based strengths for the structural fiberboard perforated shear walls exceeded the perforated shear wall method strength predictions established by the empirical equation F=r/(3-2r) as described in SDPWS Commentary (AWC, 2008).

Introduction

The perforated shear wall design provisions in SDPWS are currently limited to design of perforated walls sheathed with wood structural panels. This limitation is due to the empirical basis of the perforated shear wall strength reduction factor which is largely based on tests of wood structural panel shear walls. To extend applicability of the design method to perforated shear walls sheathed with structural fiberboard, testing was undertaken to verify that the perforated shear wall strength reduction factors can be safely extended to structural fiberboard perforated shear walls. Perforated wall configurations used in this study were identical to those used in ICC-ES AC269.1. In addition, as part of this study, the corner return configuration of AC269.1 was tested. Testing summarized herein was undertaken as part of a collaborative effort between American Wood Council and the Structural Fiberboard Association.

Test Method

Racking tests were in accordance with AC269.1's modifications to ASTM E 564 Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings (ASTM, 2006). The applied shear loading was in compression and directed from left to right based on orientation of walls specimens depicted in Table 1.

Specimens

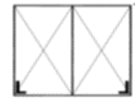
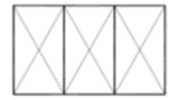
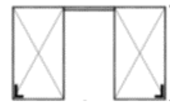
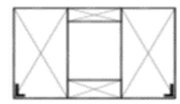
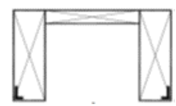
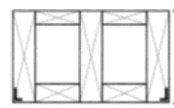
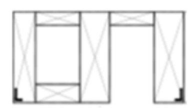
Wall Types 1-7 were fabricated in general conformance with the available requirements and details outlined in AC269.1. Figure 1 depicts basic dimensions for Wall Type 1. Figure 2 illustrates a typical test wall in the test frame. Other relevant materials and details of construction used in fabrication of the test specimens were as follows:

Framing: Studs and plates were 2x4 nominal Douglas-Fir "No. 1 or Better" grade. Stud spacing was 16 in. o.c. except where wall configurations required smaller stud spacing adjacent to openings. All end studs where hold-downs were used were built-up (2) 2x members. Headers were single-ply 2x12 nominal Douglas-Fir. Headers were supported at each end by one jack stud.

Sheathing: All structural fiberboard sheathing was 1/2 in. in accordance with ASTM C208 Standard Specification for Cellulosic Fiber Insulating Board.

Sheathing nails: Sheathing nails were nominal 1/8 in. diameter (i.e. 0.120 in. diameter) x 1 3/4 in. long x 3/8 in. head diameter galvanized roof nail spaced at 3 in. o.c. at panel edges and 6 in. o.c in the field of the panel. The fasteners were installed to maintain a 3/8 in. minimum edge distance at adjoining panel edges and at

Table 1. Test Wall Configuration for Structural Fiberboard Sheathed Walls

Wall Type	Description	Wall Size Height x Length	Clear Opening Height, %H	Sheathed Segment Aspect Ratio (H/L_s)	Full-Height Sheathed Length	Hold-down?	Wall Configuration
1	Baseline	8 ft x 8 ft	-	1:0	8 ft.	Yes	
2	Corner Re-turn	8 ft x 12 ft	-	1:1.5	12 ft	No	
3	Full-Height Opening	8 ft x 12 ft	100%	2:1	8 ft	Yes	
4	Window Opening	8 ft x 12 ft	85%	2:1	8 ft	Yes	
5	Door Open-ing	8 ft x 13.3 ft	85%	3:1	5 ft-4 in	Yes	
6	Two-window Opening	8 ft x 14 ft	65%	4:1	6 ft	Yes	
7	Window & Door Open-ing	8 ft x 15.3 ft	65% Win-dow 85% Door	4:1, 3:1	7 ft-4 in	Yes	

¾ in. edge distance at all other structural fiberboard panel perimeters.

Framing nails: Plates were nailed to studs using (2) 3.5 in. x 0.131 in. nails at each end. Headers were toe-nailed to full-height studs at each end using (4) 2.5 in. x 0.120 in. nails). Top plate to header nailing and top plate to top plate nailing consisted of 3 in. x 0.120 in. nails at 24 in. o.c. Jack studs were nailed to adjacent full-height studs with 3 in. x 0.120 in. nails spaced at 24 in. o.c.

Anchor bolts: Anchor bolts were 5/8 in. diameter with 3 in. x 3 in. x 1/4 in. square plate washers between the bottom plate and the nut. Anchor bolts were spaced at 24 in. o.c.

Hold-downs: "HDQ8-SDS3" hold-downs were used for Wall Types 1, 3, 4, 5, 6 and 7. In all cases, nine screws attached each hold-down to (2) 2x end studs. The number of screws used for overturning anchorage attachment was determined such that the hold-down-to-end stud connection was only slightly greater in strength than the wall's allowable stress wind design overturning force of 2,580 lbf. The overturning force represents the wind design allowable unit shear value for Wall Type 1 times the wall height of 8 ft (e.g. 323 plf x 8 ft = 2,580 lbf). Hold-downs in Wall Types 3, 4 5, 6, and 7 are sized for this same unit shear force to enable the tension side end panels to develop the same unit shears as associated with Wall Type 1. Nailing between the two-ply end studs consisted of (17) 3 in. x 0.120 in. nails evenly-spaced.

The combination of 1/2 in. thick SFB sheathing and nominal 1/8 in. diameter galvanized roofing nails used for all of the test specimens in this study is associated with the 645 plf shear wall nominal unit shear for wind design that the SDPWS tabulates for framing that has a specific gravity of at least 0.50.

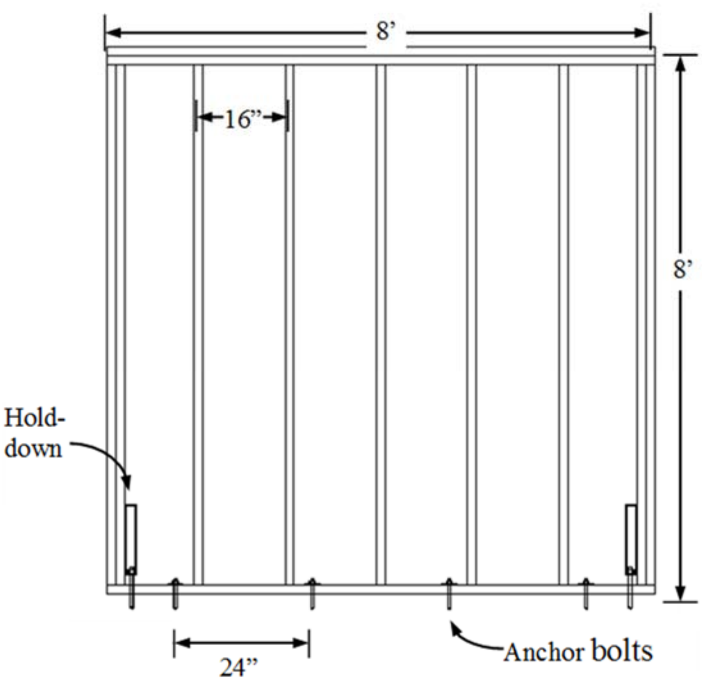


Figure 1. Wall Type 1 Specimen.

As illustrated in Table 1, all walls utilized a hold-down at ends except for Wall Type 2. Wall Type 2 was framed with 24 in. long sheathed corner returns. Details of construction of the corner return used in this study are depicted in Figure 3. Figure 3a depicts nailing of adjacent corner studs at 12 in. o.c. as well as installation of an additional row of sheathing fasteners, spaced 3 in. o.c., to the inside corner stud. This row of nailing is considered to be more than the minimum typical edge nailing consisting of a single row of fasteners at each panel edge. A three-stud corner with a 1-1/4 in. gap between adjacent studs was used to represent a typical framed corner in accordance with the IRC.

Test Results

Table 2 provides a summary of the peak load for each wall test. Load deflection curves for Wall Types 1-7 are shown in Figure 4. It should be noted that in Figure 4, the x-axis represents the racking deflection at the top of wall in inches. Table 2 and the y-axes of Figure 4 provide the measured racking strength as a strength ratio calculated in accordance with Equation 1:

$$\textit{Strength Ratio} = \frac{(\textit{load unit shear})}{(\textit{Peak load unit shear})_{\textit{Baseline}}}$$

where:

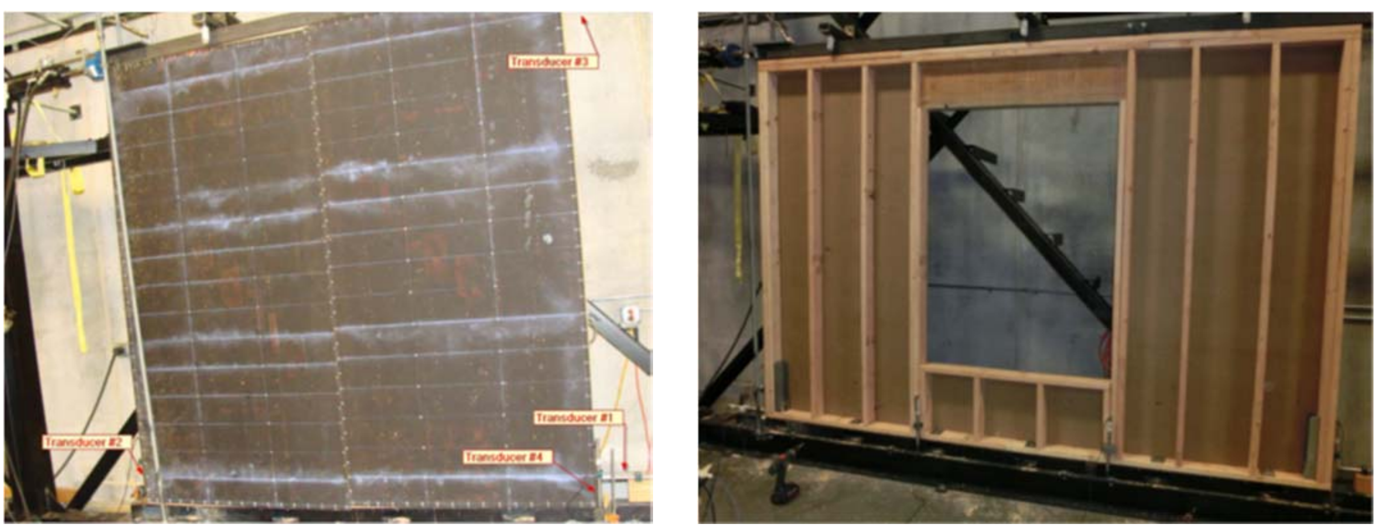
(load unit shear) = load for the wall configuration of interest divided by the total wall length, plf

(Peak load unit shear)$_{Baseline}$ = Average peak load unit shear for Wall Type 1 (e.g. Average peak load of Wall Type 1 divided by length of 8 ft), plf

Failure modes included a combination of nail heads pulling through the thickness of the sheathing (commonly referred to as "nail head pull through"), and sheathing edge tear-out (see Figure 5).

Fully Sheathed Baseline (Wall Type 1):

Wall Type 1 serves as a baseline used for relative evaluation of the other six wall types.



(a) **(b)**

Figure 2. (a) Outside Face of Wall Type 1, (b) Inside Face of Wall Type 4.

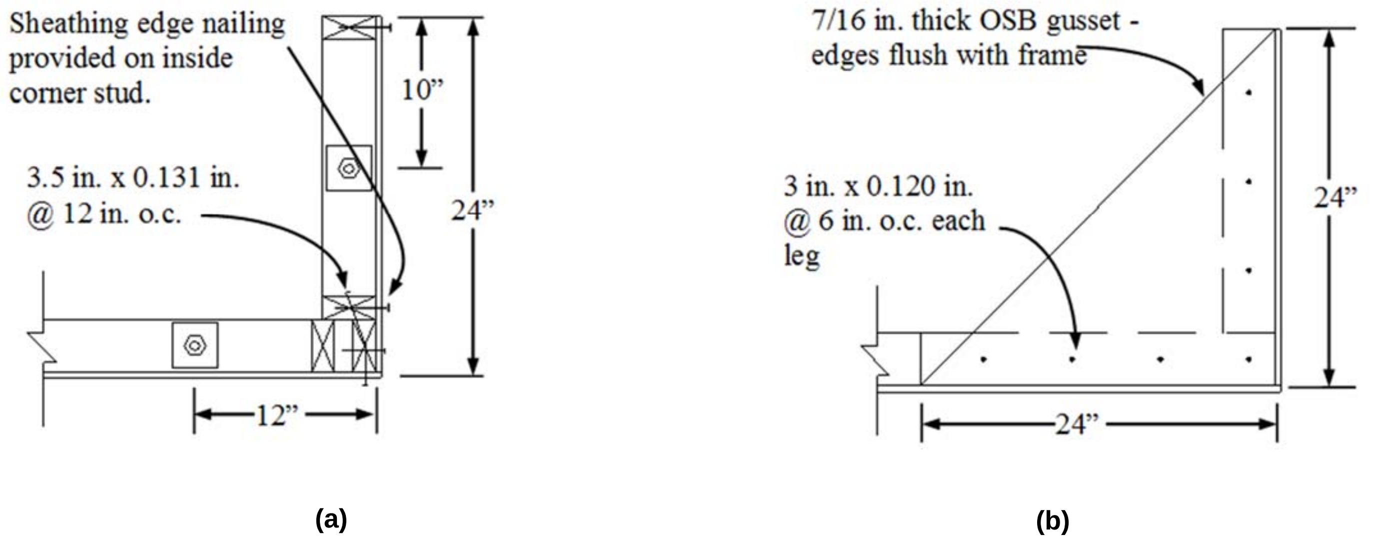


Figure 3. Wall Type 2 Corner Return, (a) Location of Bottom Plate Anchor Bolts, (b) Attachment of Triangular OSB Gusset to Wall Top Plates.

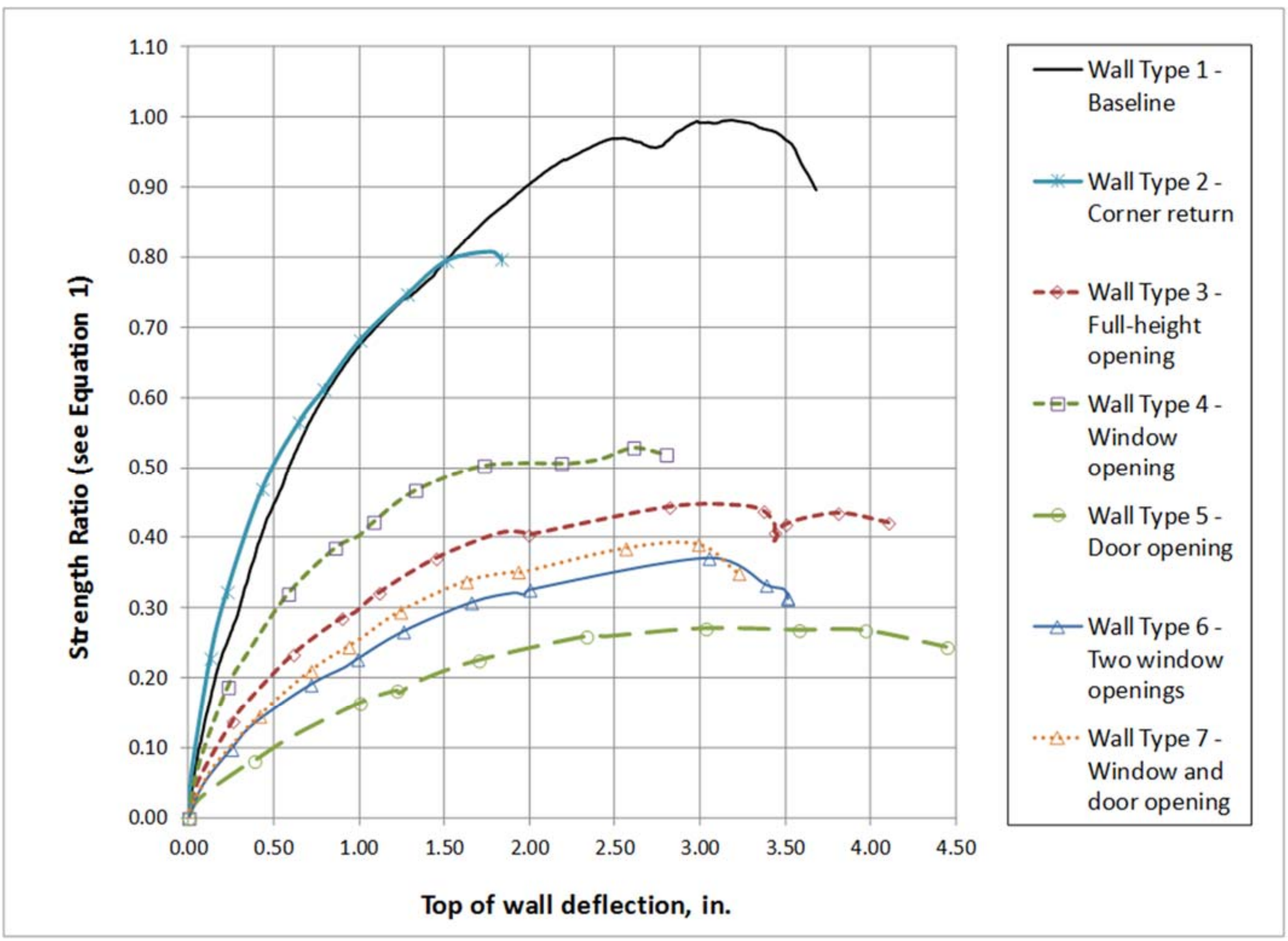


Figure 4. Load-Deflection Curves for Wall Types 1-7.

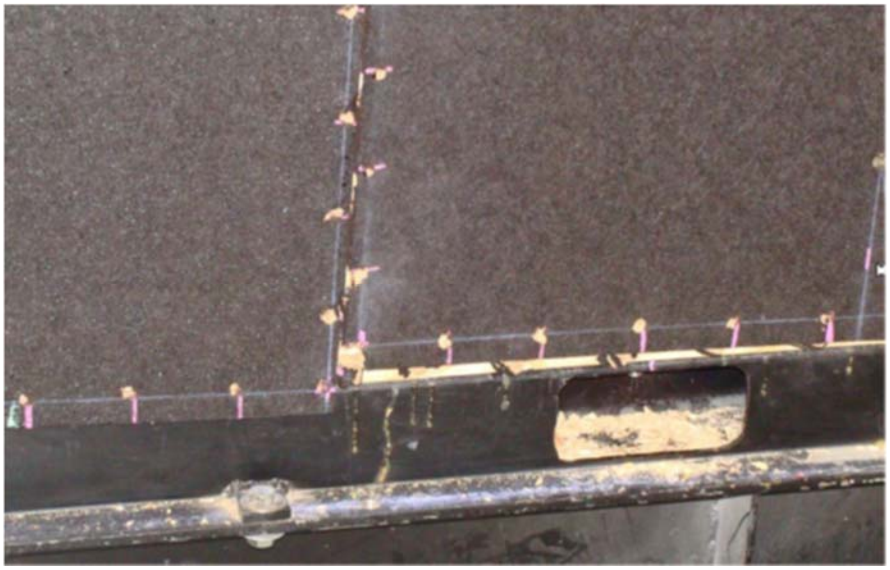


Figure 5. Typical Failure Modes Including Nail Head Pull Through and Sheathing Tear Out.

Perforated Shear Wall Comparisons (Wall Type 3, 4, 5, 6 and 7):

A comparison of tested peak strength ratios in accordance with Equation 1 and "reference" strength ratios in accordance with the perforated shear wall design method is provided in Table 3. Column 3 provides reference perforated shear wall strength ratios. Column 4 provides tested strength ratios at peak load. Column 5 provides the ratio between Columns 3 and 4. From Column 5, it is seen that tested strengths in this study exceeded the reference calculation-based strengths for the perforated configurations by varying margins ranging from 1.04 to 1.34. The structural fiberboard perforated shear wall strength performance is 4-34% greater than the strength ratio resulting from the underlying equation for the perforated shear wall design method in SDPWS.

Corner return wall (Wall Type 2):

The corner return wall is not part of the perforated shear wall calculation method and therefore results are tabulated separately. For SFB corner return walls, the average strength ratio is 0.81 which is greater than the 0.79 factor specified in AC269.1. The 0.79 factor is based on prior testing of WSP walls (Dolan, 1997 or HUD, 2001).

Summary

This test program confirmed the applicability of the perforated shear wall design strength reduction factors for the design of structural fiberboard perforated shear walls. Results show that tested strength ratios exceeded design strength ratios for all perforated wall configurations tested. Testing of corner return wall configurations of AC 269.1 showed that SFB corners in accordance with that method meet requirements of AC269.1 for corner returns.

References

International Code Council Evaluation Service (ICC-ES). February 2013. ICC ES AC269.1 Acceptance Criteria for Proprietary Sheathing Attached to Wood Light-Frame Wall Construction Used and Braced Wall Panels Under the IRC. Falls Church, VA.

Table 2. Summary of Test Results[1]

Wall Type	Wall Description	Peak Load (lbf/plf)[2]	Strength Ratio[3]
1	Baseline	5455/(682)	
		5496/(687)	
	Average:	5476/(685)	1.00
2	Corner Return	6658/(555)	0.81
3	Full-Height Opening	3698/(308)	0.45
4	Window Opening	4351/(363)	0.53
5	Door Opening	2471/(186)	0.27
6	Two Window Opening	3554/(254)	0.37
7	Window & Door Opening	4093/(268)	0.39

[1]From PFS Test Report #13-064 ASTM E564 Racking Load Tests for North American Fiberboard Association by PFS Corporation, Cottage, Grove, Wisconsin.
[2]Unit shear values, plf, calculated by dividing peak load by total wall length to be consistent with the total wall length basis of the perforated shear wall strength ratio equation: F=r/(3-2r) (see Commentary to AWC, 2008).
[3]Strength ratio calculated in accordance with Equation 1.

Table 3. Comparison of Reference and Tested Strength Ratios

Wall Type	Description	Reference Strength Ratio	Test Strength Ratio	$\left(\frac{\text{Test Strength Ratio}}{\text{Reference Strength Ratio}}\right)$
1	Baseline	1.00	1.00	1.00
3	Full-Height Opening	0.40	0.45	1.03
4	Window Opening	0.51	0.53	1.04
5	Door Opening	0.21	0.27	1.29
6	Two Window Opening	0.28	0.37	1.32
7	Window & Door Opening	0.29	0.39	1.34

American Wood Council (AWC). 2008. Special Design Provisions for Wind and Seismic (SDPWS) with Commentary. Leesburg, VA.

Dolan, J.D. and C. Heine. 1997. Sequential Phased Displacement Tests of Wood-framed Shear Walls with Corners. Report No. TE-1997-003. Brooks Forest Products Research Center, VPI&SU, Blacksburg, VA.

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