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1 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. In addition, the end restraint options available for the CS-WSP bracing option include details that permit 610 mm (24 in.) long return corner walls to be used in lieu of hold-down anchorage for wall segments less than 1220 mm (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

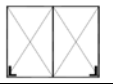
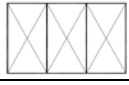

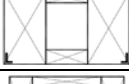

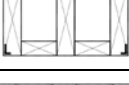

conducted the ASTM E72 wall racking tests as described in AC269.1. Both laboratories conducted tests on a full series of the CS-WSP bracing configurations described in AC269.1, including the baseline wall, corner return wall, and perforated walls. Each laboratory tested three baseline walls and either one or two walls for each additional configuration. A total of 21 walls were tested.

2 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 E72 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 configurations for an AC269.1 evaluation of CS-WSP bracing.

Wall type	Description	Wall size height x length (m)	Clear opening height, %H	Sheathed segment aspect ratio (H/L _s)	Full-height sheathed length (m)	Hold-down	Wall configuration
- ⁽¹⁾	ASTM E 72	2.4 x 2.4	-	1:0	2.4	Yes	-
1	Baseline	2.4 x 2.4	-	1:0	2.4	Yes	
2	Corner return	2.4 x 3.7	-	1:1.5	3.7	No	
3	Full-height opening	2.4 x 3.7	-	2:1	2.4	Yes	
4	Window opening	2.4 x 3.7	-	2:1	2.4	Yes	
5	Door opening	2.4 x 4.1	-	3:1	1.6	Yes	
6	Two window openings	2.4 x 4.3	-	4:1	1.8	Yes	
7	Window & door openings	2.4 x 4.6	65% window 85% door	4:1, 3:1	2.2	Yes	

(1 ft. = 0.3048 m)

⁽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

3 Specimens

The ASTM E72 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 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 38 x 90 mm (2x4 nominal) Douglas-Fir “Standard or Better” grade. Stud spacing was 406 mm (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) 38 mm thick members. Headers were single-ply 38 x 290 mm (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 9.5 mm (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 (DOC-NIST, PS2-10).
- **Sheathing nails:** Sheathing nails were 6d common (50 x 2.87 mm) (2.0 x 0.113 in.) fasteners spaced at 150 mm (6 in.) at panel edges and 300 mm (12 in.) in the field of the panel. The fasteners were installed to maintain a 9.5 mm (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 (65 x 2.87 mm) (2.5 x 0.113 in.). Top plate to header nailing consisted of 75 x 3.33 mm (3.0 x 0.131 in.) nails at 610 mm (24 in.) o.c. Window sills were end-nailed to studs using (2) 16d box (90 x 3.43 mm) (3.5 x 0.135 in.) at each end. Jack studs were nailed to king-post with 75 x 3.33 mm (3.0 x 0.131 in.) nails spaced at 610 mm (24 in.) o.c.
- **Anchor bolts:** Anchor bolts were 15.9 mm (5/8 in.) diameter with 75 x 75 x 5.82 mm (3 x 3 x 0.229 in.) square plate washers between the bottom plate and the nut. Anchor bolts were spaced at 610 mm (24 in.) o.c. An anchor bolt was located 300 mm (12 in.) from ends of each bottom plate except for walls with openings where anchor bolts were located within 300 to 600 mm (6 to 12 in.) 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 double 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 9.96 kN (2,240 lbf). The overturning force represents the wind design allowable unit shear value for Wall Type 1 times the wall height of 2.4 m (8 ft) (e.g. 4.1 kN/m x 2.4 m) (e.g. 280 plf x 8 ft). 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) 75 x 3.33 mm (3 x 0.131 in.) nails evenly-spaced to match the ASD shear wall overturning force.



Figure 1: Example Wall Type 7 specimen at Laboratory A (left) and Example Wall Type 2 – corner return specimen at Laboratory B (right)

The combination of 9.5 mm (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 linked to the 8.2 kN/m (560 plf) minimum shear strength target that AC269.1 requires for the ASTM E72 wall racking test specimens. This capacity target was based upon the 8.2 kN/m (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 610 mm (24 in.) 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 2. A three-stud corner with a 32 mm (1-1/4 in.) gap between adjacent studs was used to represent a typical framed corner in accordance with the IRC.

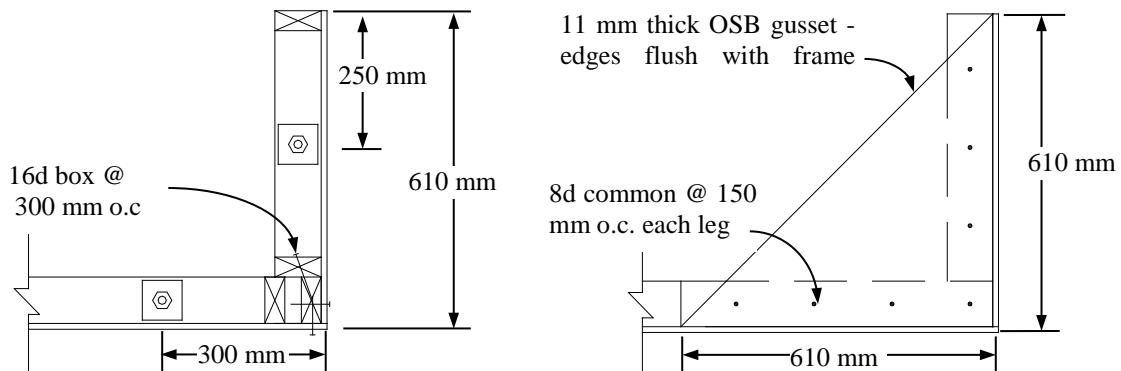


Figure 2. Wall Type 2 corner return: location of bottom plate anchor bolts (left), and attachment of triangular OSB gusset to wall top plates (right). (1 in. = 25.4 mm)

4 Test Results

Detailed test results are provided in Table A.1. Table 2 provides a summary of the strength-based criterion of interest in this study. Load deflection curves for Wall Types 1-7 from Series A data are shown in Figure 3 (left) as an example of typical load deflection behavior. Load deflection curves for the ASTM E72 walls are shown in Figure 3 (right). 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 3 (left), the x-axis represents the racking deflection at the top of wall in mm. In figure 3 (right), per ASTM E72, 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 3 provide the measured racking strength as a normalized ratio calculated in accordance with Equation 1:

$$\text{Strength ratio} = \frac{(\text{load unit shear})}{(\text{Peak load unit shear})_{\text{Baseline}}} \quad \text{Eq. 1}$$

where:

$(\text{load unit shear}) =$ load for the wall configuration divided by the total wall length, kN/m (plf)

$(\text{Peak load unit shear})_{\text{Baseline}} =$ Average peak load unit shear divided by Wall Type 1 length of 2.4 m (8 ft), kN/m (plf)

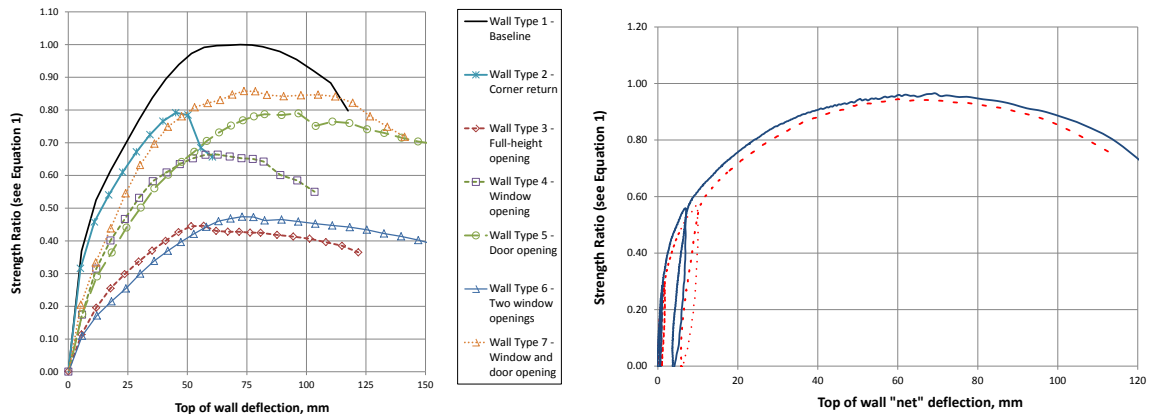


Figure 4. Series A load - deflection curves for Wall Types 1-7 (left), and Series A load - deflection curves for ASTM E72 walls (right). (1 in. = 25.4 mm)

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.

Table 2. Summary of peak load test results.

Wall type	Wall description	Data series A		Data series B		Combined data	
		Peak load kN/m	Normalized strength ratio	Peak load kN/m	Normalized strength ratio	Strength ratio	
						Average	Lower-Bound
-	ASTM E72	9.72 ⁽¹⁾	0.96	-	-	-	-
1	Baseline	9.11		9.60			
		11.06		10.58			
		10.33		8.61			
	Average:	10.17	1.00	9.60	1.00	1.00	-
	COV:	0.098		0.103			
2	Corner return	8.29	0.82	7.47 ⁽¹⁾	0.78	0.80	-
3	Full-height opening	4.61	0.45	4.85	0.51	0.48	0.43
4	Window opening	6.89	0.68	7.43	0.77	0.73	0.66
5	Door opening	3.28	0.32	3.11	0.32	0.32	0.29
6	Two window openings	4.89	0.48	4.16	0.43	0.46	0.41
7	Window & door openings	4.25	0.42	4.07	0.42	0.42	0.38

(1 plf = 0.01459 kN/m)

⁽¹⁾ Represents average of 2 tests

⁽²⁾ Lower bound is based on average minus 1 standard deviation

5 Evaluation of Measured Strength Parameters

5.1 ASTM E72 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 8.2 kN/m (560 plf) when tested in general accordance with ASTM E72 using Douglas-fir framing. In addition, the system must demonstrate racking loads of at least 2.9 kN/m (200 plf) and 5.8 kN/m (400 plf) at net deflections of 5.1 and 15.2 mm (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 9.72 kN/m (666 plf) was 19% greater than the minimum peak unit shear capacity requirement of AC269.1. The 2.9 and 5.8 kN/m (200 and 400 plf) deflections averaged 1.3 and 9.4 mm (0.05 and 0.37 in.), respectively. These findings confirm that the WSP sheathed walls met the pre-qualification requirements.

5.2 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 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 9.11 kN/m (624 plf). For series B, the minimum peak unit shear is 8.61 kN/m (590 plf). For all Wall Type 1 walls in both series A and B, average top of wall deflection did not exceed 5.1 mm (0.2 in.) at a unit shear of 2.9 kN/m (200 plf) and 15.2 mm (0.6 in.) at a unit shear of

5.8 kN/m (400 plf). To avoid a non-conservative review for alternative proprietary product, it may be appropriate for AC269.1 to impose the ASTM E72 wall strength and stiffness requirements upon Wall Type 1. WSP sheathed walls tested in this study would have supported this suggested minimum performance level.

5.3 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, it is seen that 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. This suggests 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, it is seen that even 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.

Table 3. Comparison of reference and test-based strength ratios.

[Col 1]	[Col 2]	[Col 3]	[Col 4]	[Col 5]	[Col 6]	[Col 7]
Wall type	Wall description	Reference strength ratio ¹	Test-based peak strength ratio (average)	Test-based peak strength ratio (lower-bound) ²	$\frac{\text{Col 4(Test)}}{\text{Col 3(Ref)}}$	$\frac{\text{Col 5(Test)}}{\text{Col 3(Ref)}}$
1	Baseline	1.00	1.00	-	1.00	-
2	Corner return	0.79	0.80	-	1.01	-
3	Full-height opening	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 openings	0.28	0.46	0.41	1.63	1.46
7	Window & door openings	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.

5.4 Corner return wall (Wall type 2):

Unlike the reference strength ratio for the five wall perforated 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.

5.5 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:

$$Stiffness\ ratio = \frac{\left(\frac{Reference\ load}{\Delta\ @\ Reference\ load} \right)}{\left(\frac{Reference\ load}{\Delta\ @\ Reference\ load} \right)_{Baseline}} \quad Eq. 2$$

where:

Δ = top of wall deflection, in.

Test-based stiffness ratios determined at 40% of each wall's peak unit shear capacity (kN/m or 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 (kN or 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 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: 4.1 kN/m 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 stiffnesses at the design load level.

Table 4. Comparison of reference and test-based stiffness ratios at the reference design load.

Wall type	Wall description	Reference ratio	Reference design load (kN)	Series A test-based stiffness ratio	Series B test-based stiffness ratio	Combined data from Series A and B		
						Test-based average stiffness ratio	Test Reference	
1	Baseline	1.00	9.96	1.00	1.00	1.0	1.0	
2	Corner return	0.79	11.8	1.11	0.73	0.92	1.16	
3	Full-height opening	0.40	5.98	0.33	0.71	0.52	1.29	
4	Window opening	0.51	7.56	0.50	0.56	0.53	1.05	
5	Door opening	0.21	3.45	0.21	0.21	0.21	1.00	
6	Two window openings	0.28	4.85	0.33	0.16	0.25	0.89	
7	Window & door openings	0.29	5.54	0.31	0.20	0.26	0.88	
Overall Average							1.04	

6 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 E72 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. The load levels used for the unit stiffness determinations should also be revised and clarified to account for non-linear deformation behavior and variability.

7 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.

8 References

1. APA. 2011. Voluntary Product Standard – PS2-10, Performance Standard for Wood-Based Structural-Use Panels, Form S350. APA – The Engineered Wood Association, Tacoma, WA.
2. ASTM. 2006. ASTM E564 Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings. American Society for Testing and Materials, West Conshohocken, PA.
3. ASTM. 2010. ASTM E72 Standard test Methods of Conducting Strength Tests of Panels for Building Construction, American Society for Testing and Materials, West Conshohocken, PA.

4. AWC. 2008. Special Design Provisions for Wind and Seismic (SDPWS) with Commentary. American Wood Council, Leesburg, VA.
5. 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.
6. HUD. 2003. Wood shear walls with corners. U.S. Department of Housing and Urban Development, Washington, DC. (publication available through www.huduser.org)
7. ICC. 2009. International Residential Code, International Code Council, Falls Church, VA.
8. ICC-ES. 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. International Code Council Evaluation Service, Falls Church, VA.
9. Keith, E., 2014. APA Report T2013P-18 Wood Structural Panel Shear Wall Tests in Accordance with ICC-ES AC269.1, APA-The Engineered Wood Association, Tacoma, WA.
10. Line, P, Waltz, N., Skaggs, T. 2008. "Seismic Equivalence parameter for Engineered Wood-frame Wood Structural Panel Shear Walls," Wood Design Focus (Summer), Forest Product Society. Madison, WI.
11. Waltz, N. 2012. Benchmark Monotonic and Cyclic Tests of Fully-Restrained Wood Structural Panel Braced Walls," Weyerhaeuser Engineering Laboratory Experiment No. 2379A. Weyerhaeuser, Boise, Idaho.
12. Waltz, N. 2013. ICC-ES AC269.1 Continuous Sheathing Review of Oriented Strand Board, Weyerhaeuser Engineering Laboratory Experiment No. 2407. Weyerhaeuser, Boise, Idaho.

Table A.1. Detailed data summary.

Wall type	Ref/Item	Wall Length (m)	OSB (mm)	Sheathing nails	Fastener spacing (edge/field) (mm)	Openings	Drift at 2.9 kN/m (mm)	Drift at 5.8 kN/m (mm)	40% Peak load		Peak load		Ultimate load ⁽¹⁾		Gravity load system intact	Failure ⁽²⁾
									Load (kN)	Drift (mm)	Load (kN)	Drift (mm)	Load (kN)	Drift (mm)		
- ⁽³⁾	[A]/A21	2.4	9.5	6d com	150/300	No	1.5	10.8	9.4	3.0	23.4	60.1	18.8	113.0	Yes	P, T, W
- ⁽³⁾	[A]/A22	2.4	9.5	6d com	150/300	No	1.2	7.9	9.6	2.2	23.9	69.4	19.2	116.5	Yes	P
1	[B]/1	2.4	9.5	6d com	150/300	No	6.7	10.3	8.9	8.3	22.2	80.4	17.8	139.9	Yes	W, T, P
1	[B]/2	2.4	9.5	6d com	150/300	No	2.7	12.2	10.8	6.0	27.0	74.5	21.6	124.1	Yes	P, W
1	[B]/3	2.4	9.5	6d com	150/300	No	3.6	14.3	10.1	6.4	25.2	75.5	20.2	117.2	Yes	W, T, P
1	[C]/4	2.4	9.5	6d com	150/300	No	-	9.3	9.4	-	23.4	59.3	18.7	97.2	Yes	P, W
1	[C]/5	2.4	9.5	6d com	150/300	No	1.0	5.7	10.3	2.4	25.8	70.0	20.6	98.9	Yes	P, W
1	[C]/6	2.4	9.5	6d com	150/300	No	1.7	12.7	8.4	2.6	21.0	55.9	16.8	127.7	Yes	P, T
2	[B]/7	3.6	9.5	6d com	150/300	No	-	-	12.1	5.3	30.3	48.0	24.3	62.1	Yes	T at end return bottom
2	[C]/8	3.6	9.5	6d com	150/300	No	-	-	11.0	2.8	27.5	36.2	22.0	49.7	Yes	T
2	[C]/9	3.6	9.5	6d com	150/300	No	-	-	10.8	3.3	27.1	47.8	21.7	65.5	Yes	T
3	[B]/10	3.6	9.5	6d com	150/300	Yes	-	-	6.7	10.3	16.9	59.1	13.5	124.0	Yes	T, P, W
3	[C]/11	3.6	9.5	6d com	150/300	Yes	-	-	7.1	2.4	17.7	39.4	14.2	109.5	Yes	P, W
4	[B]/12	3.6	9.5	6d com	150/300	Yes	-	-	10.1	9.2	25.2	60.6	20.1	106.6	Yes	B, P, T
4	[C]/13	3.6	9.5	6d com	150/300	Yes	-	-	10.9	5.5	27.2	56.4	21.7	80.4	Yes	P, T
5	[B]/14	4.1	9.5	6d com	150/300	Yes	-	-	5.3	14.2	13.3	94.4	10.7	165.5	Yes	P, T, B
5	[C]/15	4.1	9.5	6d com	150/300	Yes	-	-	5.0	6.0	12.6	57.4	10.1	125.9	Yes	B, P, T
6	[B]/16	4.3	9.5	6d com	150/300	Yes	-	-	8.4	14.3	20.9	75.2	16.7	158.0	Yes	B, T, P
6	[C]/17	4.3	9.5	6d com	150/300	Yes	-	-	7.1	10.9	17.7	72.6	14.2	122.8	Yes	T, B, P
7	[B]/18	4.6	9.5	6d com	150/300	Yes	-	-	7.9	12.0	19.9	77.1	15.9	146.0	Yes	B, T, P
7	[C]/19	4.6	9.5	6d com	150/300	Yes	-	-	7.6	7.8	19.0	65.3	15.2	150.6	Yes	B, T, P

Notes

(1 ft = 0.3048 m, 1 in. = 25.4 mm, 1 plf = 0.01459 kN/m, 1 lbf = 0.004448 kN)

⁽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 (2012), [B] Waltz (2013), [C] Keith (2014)