DEVELOPMENT OF SEISMIC DESIGN COEFFICIENTS

FOR STEEL-CLAD WOOD-FRAMED SHEAR WALLS

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Steel-clad wood-framed shear walls are part of the main lateral load resisting system in post-frame buildings subjected to wind and earthquake lateral loadings. ANSI/ASCE [American National Standards Institute/ American Society of Civil Engineers] 7-2010 is the main code-recognized standard for determining seismic loads. ANSI/ASCE 7-2010 Table 12.2-1 lists seismic design coefficients and factors for a range of building types.

Seismic performance factors, including response modification factor (R), system overstrength factor (Ω_o), and deflection amplification factor (C_d), are used to estimate strength and deformation demands on seismic force-resisting systems that are designed using a linear method of analysis but behave nonlinearly during an earthquake. Of particular interest for post-frame buildings is Section B.23 for Building Frame Seismic Force-Resisting Systems. This section of ANSI/ASCE 7-10 is limited to cold-formed steel framing, sheathed with wood panels rated for shear resistance or with steel panels. Wood framing is not mentioned in Section B.23.

Because seismic design coefficients have not been developed for the design of SCWF shear walls, the objective of this study was to laboratory-test SCWF shear walls and develop the seismic design coefficients needed by post-frame building designers who desire to rely on SCWF shear walls as the main lateral force-resisting system for seismic forces.

BACKGROUND

The Federal Emergency Management Agency developed two methods, FEMA P-695 (2009) and FEMA P-795 (2011), for quantifying system seismic performance factors and response parameters used in seismic design. FEMA P-695 is used to evaluate the seismic design coefficients for new structural systems, or buildings. The methodology for establishing seismic performance factors requires testing under pushover and cyclic loading and extensive computer modeling of different configurations of structural systems. The configurations should represent all probable configurations of framing, construction details and material property variations. The nonlinear analysis techniques are performed after the development of configurations. Performance of structural systems is evaluated using the *collapse margin ratio*, which is the ratio between the median collapse intensity and the intensity at maximum considered earthquake-level ground motions. The whole process is iteratively performed until the collapse margin ratio is satisfied. The methodology in FEMA P-695 requires extensive physical testing and computer modeling, at a cost that easily exceeds \$500,000. Instead of using the costly and extensive FEMA P-695 approach, an alternate approach can be used to establish equivalency at the *component level* using the FEMA P-795 procedure.

The Component Equivalency Methodology (FEMA P-795) evaluates the seismic performance *equivalency of components* such as connections, structural elements, or subassemblies (e.g., shear wall segments) experiencing inelastic response that controls the collapse performance of a seismic-force-resisting system. FEMA P-795 is an adaptation of FEMA P-695, with the major difference being that FEMA P-695 evaluates the level of collapse safety on the basis of the response of the entire seismic-force-resisting system. In contrast, the Component Equivalency Methodology evaluates the seismic performance equivalency of *structural components* that are substituted for reference components in seismic-force-resisting systems. Moreover, International Code Council Acceptance Criterion 322 (2009) was developed to establish the seismic equivalency of proposed components with the specific case of nailed wood shear walls in light-frame construction. The performance parameters in Appendix A of AC 322 were developed from data sets for nailed wood shear walls with aspect ratios ranging from 2:1 to 1:1, tested using the Consortium of Universities for Research in Earthquake Engineering loading protocol. The candidate shear wall system can be used and shares the same seismic design coefficients as light-frame (wood-framed or cold-form steel-framed) wood bearing walls sheathed with wood structural panels or steel panels constructed in accordance with Appendix A of AC 322.

Post-frame buildings are typically one-story, clear-span buildings with shear walls providing resistance to lateral loading. Given this fact, we contend that the component equivalency approach is sufficient. Therefore, this research evaluated the performance of SCWF shear walls (with and without oriented-strand-board reinforcement) tested under reverse-cyclic loading in accordance with ASTM E2126-11. The seismic design coefficients were established to be equivalent to a light-framed shear wall using the FEMA P-795 methodology. The performance parameters of the post-frame shear walls were compared with nailed wood shear walls using the criteria in Appendix A of AC 322.

MATERIALS AND METHODS

A total of 18 walls of 10 configurations were tested under reverse-cyclic loading to develop design shear strength, stiffness and seismic response coefficients to resist lateral loads from seismic or wind events. There are three main SCWF shear wall constructions: unstitched, lightly stitched, and heavily stitched. For unstitched shear wall configurations, no stitch screws were used at the panel lap seam. For lightly stitched and heavily stitched (**Figure 1**) shear wall configurations, stitch screws were used at a spacing of 24 inches, and 8 inches at lap seam, respectively. Tested results show that these walls exhibited high ductility and withstood large in-plane displacements with minor load reduction, especially for the unstitched SCWF shear wall constructions. These walls also showed similar hysteresis behavior to light-frame wood shear wall constructions. The seismic response coefficients for a number of SCWF shear walls with high ductility were judged to be equivalent to light-frame wood shear walls.

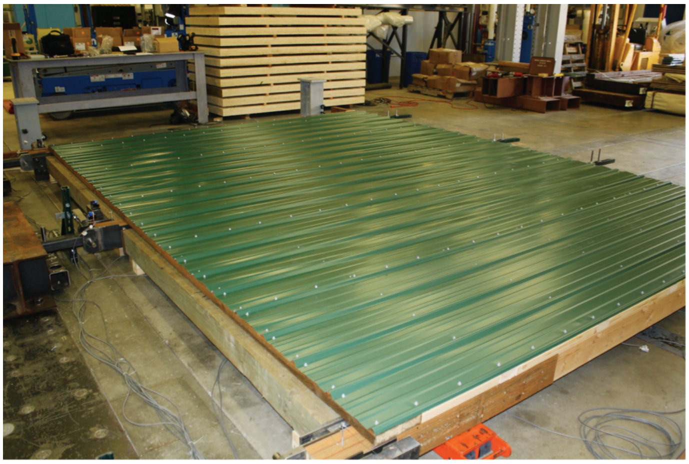

FIGURE 1. Shear wall with heavy stitching at the steel panel overlaps. The stitch screws were spaced 8 inches apart.

TABLE 1. Seismic Design Values and Ductility Ratios

SHEAR WALL ID	DESCRIPTION	AVERAGE STATISTICS		
		Allowable design unit shear, *V* (lb/ft)*	Shear modulus, *G* (kips/in)*	Ductility ratio (*D*)
1	36″ girt, #10x1″ screws in field. #10x1″ structural screws on the left side at lap joints, 9″ on center major rib panel	80	4.4	18.6
2	36″ girt, #10x1″ screws throughout, screws on both sides of major rib, 9″ oc major rib panel	85	5.8	21.7
4	24″ girt, #10x1″ screws in field, #12x1.5″ structural screw through overlap rib at girts, 9″ oc major rib panel	135	12.1	23.1
5	24″ girt, #10x1″ screws in field, #12x0.75″ stitch at 8″ oc and blocking with 8″ oc #10x1″, 9″ oc major rib panel	240	15.3	10.0
6	24″girt, #10x1″screws in field, #12x0.75″stitch through overlap rib at girts, 9″ oc major rib panel	135	13.1	14.4
7	36″ girt, #10x1″ screws in field, #12x0.75″ stitch at 18″ oc and blocking with 18″ oc #10x1″, 9″ oc major rib panel	140	12.5	9.7
10	7/16 Rated oriented strand board sheathing inset between posts on interior wall side. 1-3/4″x0.120″ coil nails spaced at 6″ on panel edges and 12″ field (panels were fully blocked)	300	11.2	5.4
11	7/16 Rated OSB sheathing on interior side, and Wall Type 4 on exterior side	455	17.1	7.3
13	Similar to wall type 7 but using 1.5″ stitch screw at girt, and 0.75″ stitch screw between girts	150	13.7	26.8
14	Similar to wall type 5, except using 1.5″ stitch at girts, and 0.75 between girts	250	22.8	13.4

*Average value was calculated from average envelope curve. Allowable unit shear rounded to nearest 5 lb/ft, and shear modulus reported at 2 significant digits (similar rounding rules as in AWC SDPWS-2015) (American Wood Council, 2015).

Materials and wall constructions

Many of the shear wall constructions reported herein mirrored those from a previous study on monotonic loading of post-frame shear walls (Bender, 2012). Walls had materials and features that would allow for conservative substitution. For example, 3-ply nail-laminated Hem-Fir 2x6 columns with splice joints were used, so a denser species grouping, such as Southern Pine, or a solid or glulam post could be conservatively substituted. Wall girts (Spruce-Pine-Fir) and skirt boards (pressure-preservative-treated, incised Hem-Fir) included splice joints at the center post. The strategy was to test as many different wall types as possible to learn the relative effects of construction details on the dynamic behavior of shear walls under reversed cyclic loading. Details of wall construction can be found in **Table 1** and the Bender (2015) report.

Methods

Shear wall tests were conducted in accordance with ASTM E2126-11. Cyclic protocols require a reference displacement to characterize the displacement history. The monotonic tests of SCWF shear walls demonstrated extremely ductile behavior, which is desirable for seismic design. One value that is needed is the wall displacement at a load level that is 80 percent of the peak load, called $0.8P_{peak}$. The SCWF shear walls proved to be so ductile that this value was not reached; hence, the reference displacement was chosen to be $2.5\%h_x = 3.6$ in (h_x is wall height). Each shear wall speci-

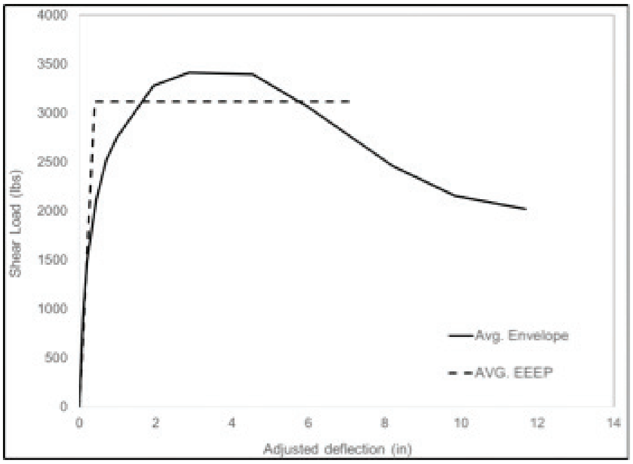

FIGURE 2. Shear strength backbone curve of unstitched shear walls, along with the equivalentenergy elastic-plastic (EEEP) overlay

TABLE 2. ICC AC 322 Criteria for Seismic Equivalency with Wood Shear Wall

SHEAR WALL ID	REPETITIONS	ICC AC 322 CRITERIA			EQUIVALENT TO LIGHT-FRAME WOOD SHEAR WALLS
		$2.5 \le V_p/V_{asd} \le 5.0$	$\Delta 0.8V_p/\Delta V_{asd} \ge 11$	$\Delta 0.8V_p \ge 2.8\%h_x$	
1	1	Pass	Pass	Pass	Yes
2	2	Pass	Pass	Pass	Yes
4	2	Pass	Pass	Pass	Yes
5	2	Pass	Pass	Pass	Yes
6	2	Pass	Pass	Fail (3.92 < 4.03)	No
7	2	Pass	Pass	Fail (2.76 < 4.03)	No
10	2	Pass	Pass	Fail (3.66 < 4.03)	No
11	3*	Pass	Pass	Pass	Yes
13	1	Pass	Pass	Pass	Yes
14	1	Pass	Pass	Fail (4.0 < 4.03)	No

*Two repetitions were checked with AC 322 criteria because one wall failed prematurely at the load strut.

V_p = peak strength capacity; V_{asd} = allowable design capacity = $V_p/2.5$;

$\Delta 0.8V_p$ = displacement at $0.8\,V_p$; ΔV_{asd} = displacement at V_{asd}; h_x = height of wall = 144 inches.

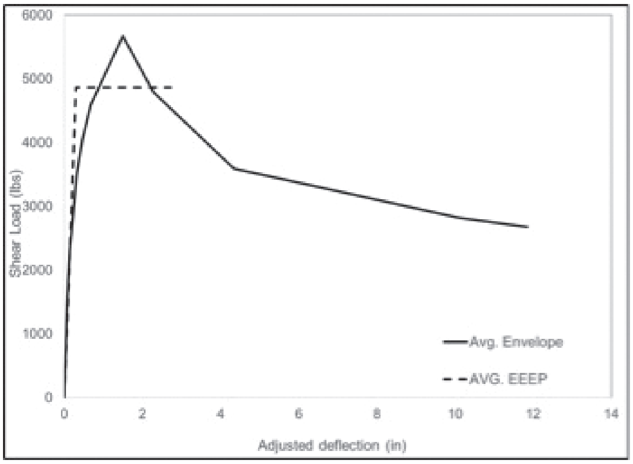

FIGURE 3. Shear strength backbone curve of lightly stitched shear walls

men was subjected to 52 load cycles with displacement amplitudes that are based on percentages of the reference displacement. A displacement rate of 0.6 in/s was chosen based on the provisions of ASTM E2126-11.

RESULTS AND DISCUSSION

The majority of the shear walls tested exhibited extreme ductile behavior with ductility ratios given in Table 1. These walls also withstood large in-plane displacements with minor load degradation, especially with the unstitched wall configurations (Wall Constructions 1 and 2), as shown in **Figure 2.** Hysteretic behavior and shear strength backbone curves of other shear wall configurations can be found in the Bender (2015) report. Moreover, the envelopes of the hysteresis loops are close to a monotonic curve (Bender, 2012) up to the point that the shear walls reach their ultimate shear strengths. However, the strength degradation of the hysteresis loops is more severe than the monotonic curves because of the cumulative damage of cyclic loading, such as screws being ejected as the holes in the steel enlarged. Design shear strength and shear stiffness are shown in Table 1. These design values are close to those of shear walls tested under monotonic load (Bender, 2012) because there is not much difference between the envelope of hysteresis loops and monotonic curves up to the ultimate strength point.

The results of the AC 322 equivalency criteria are shown in **Table 2.** The majority of the shear walls tested demonstrated ductile behaviors, easily passing the AC 322 equivalency criteria. Shear Wall Constructions 6, 7, 10 and 14 failed the AC 322 criteria regarding the displacement at 80 percent post-peak load as shown in Table 2. The primary reason was that the stitch screws that improved the initial stiffness and strength of the walls were soon ejected after reaching peak load as the holes around the stitch screws enlarged and the panels buckled during cyclic loading. As soon as the stitch screws were ejected, the shear capacity quickly diminished, as shown in **Figure 3.** With Wall Construction 10, the OSB panels were inset between the posts (on the opposite side from the steel). The posts helped restrain panel rotation, causing the post-frame OSB shear walls to have approximately 12 percent higher capacity than conventional light-frame OSB shear walls. As the ultimate capacities were reached, the OSB panels buckled, causing nail withdrawal and rapid reduction in the shear capacity. Hence, the displacement at 80 percent post-peak load did not meet the AC 322 criterion. Finally, in Wall Construction 11, steel panels were used on one side of the wall and OSB on the other. The steel added sufficient ductility to the wall to pass the AC 322 criteria, and the unit shear capacities of OSB-sheathed walls and steel-sheathed walls proved to be additive (the combined OSB/steel wall was 5 percent higher in design shear strength than the sum of Wall Constructions 6 and 10). The combined OSB/steel wall system appears to be an excellent choice when high seismic or wind forces must be resisted.

CONCLUSIONS

Tests on 18 walls of 10 configurations were conducted under reverse-cyclic loading to develop design strength, stiffness and seismic design coefficients of SCWF shear walls. The test results show that SCWF shear walls have high ductility, as well as the ability to meet design requirements of the current timber code, especially for the unstitched SCWF shear wall constructions. On the basis of our research, the recommendations for designing SCWF shear walls follow the guidelines below:

1. The seismic design coefficients for those walls that passed all the AC 322 criteria can be considered equivalent to wood light-framed shear walls (response modification coefficient R = 6.5, overstrength factor Ω_o = 3, and deflection amplification factor C_d = 4).

2. The unstitched constructions (Wall Constructions 1 and 2) had the greatest ductility values and easily passed all the three AC 322 criteria. These wall systems can be an excellent choice when light seismic or wind loads must be resisted.

3. The strength degradations of stitched wall configurations that failed the AC 322 criteria (Wall Constructions 6, 7 and 14) are greater than those of unstitched wall configurations (Wall Constructions 1 and 2) because of the ejection of stitched screws during the cyclic loading. Therefore, these wall configurations are not recommended for use in high seismic regions but would be excellent choices for high-wind regions.

4. For the combined OSB/steel wall system in which steel panels were used on one side of the wall and OSB on the other, the steel added sufficient ductility to the wall, and the capacities of OSB-sheathed walls and steel-sheathed walls proved to be additive (the combined OSB/steel wall was 5 percent higher in design shear strength than the sum of Wall Constructions 6 and 10). Therefore, this wall system appears to be an excellent choice when high seismic or wind forces must be resisted.

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