CENTER FOR LIGHT FRAME STRUCTURAL RESEARCH

Department of Civil Engineering
Santa Clara University

COMBINED ADHESIVE-STEEL PIN APPLICATIONS FOR CFS FRAME SHEAR WALLS

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"Improving Structural Design Through Innovative Applied Research"

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INTRODUCTION

Over the past several years, a number of innovative proprietary lateral force resisting elements have been developed for implementation in the light frame residential and commercial construction. These elements are typically used where demands are relatively high and wall space is limited. In designs with low to moderate lateral demands, conventional sheathed walls that utilize mechanical fasteners are often sufficient. To explore the potential benefits, both structural and economical, of structural adhesives in combination with mechanical fasteners in light frame shear wall construction, a joint research effort was initiated in 2002 between the Center for Light Frame Structural Research and the Henkel Loctite Corporation.

The use of adhesives to bond sheathing to light framing is not an entirely new concept. In fact, the 2003 IBC, Section 2305.3.9 (wood frame construction) recognizes the additive strength of adhesives but limits the benefits to wind design and structures in Seismic Design Categories (SDC) A, B and C: *"Adhesive attachment of shear wall sheathing is not permitted as a substitute for mechanical fasteners, and shall not be used in shear wall strength calculations alone, or in combination with mechanical fasteners in Seismic Design Category D, E or F."* Section 2305.3.9 imposes no specific requirements on the properties of the adhesive. In the application presented in this report, the role of the adhesive is primary. The adhesive was developed by chemists at the Henkel Loctite Corporation to provide a dependable structural bond between wood structural panels or sheet steel and cold-formed steel framing members with the expectation that the contribution (number and type) from mechanical fasteners may be reduced. Specifically, this report documents the reversed cyclic performance of 27 mil sheet steel and 7/16-in. OSB rated sheathing (24/16 exposure 1) attached to cold-formed steel (CFS) framing with a structural adhesive and pneumatically driven steel pins produced by Aerosmith Inc..

In the following sections, details of the project scope, test procedures and test results are presented, interpreted and discussed.

SCOPE OF WORK

A series of eight single-sided (sheathing on one side only) cold-formed steel frame shear wall tests were conducted on 2 ft. x 8 ft. and 4 ft. x 8 ft. (out-to-out dimensions) walls. The eight tests comprised four different shear wall configurations that utilized either a single 27 mil (33 ksi) sheet steel or a single 7/16-in. OSB rated sheathing 24/16 exposure 1 wood structural panels.

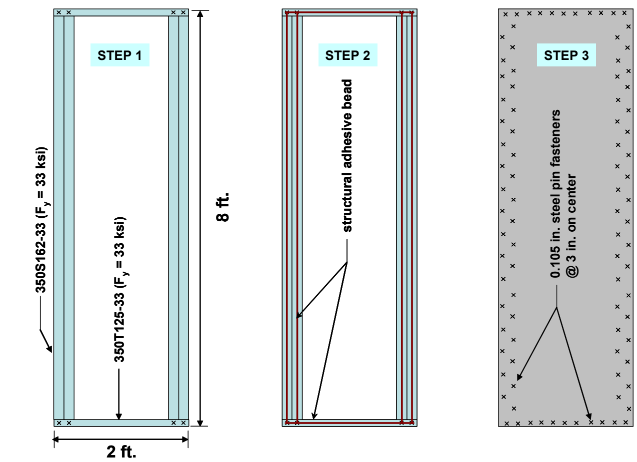
Four of the eight walls were constructed using 27 mil sheet steel. These steel sheathed walls were identical except for their overall dimensions—two walls were 2 ft. x 8 ft. and the other two were 4 ft. x 8 ft. Framing for each wall consisted of 350S162-33 studs at 24 in. on center and 350T125-33 mil top and bottom tracks. The chord studs were back-to-back studs connected with two No. 10 fasteners (transverse to the stud height) at 12 inches on center through the web of the studs. The 27 mil sheet steel was attached to the CFS frame with a bead of structural adhesive on each "contact flange" and Aerosmith 0.105 in. knurled steel pins at 3 in. on center at sheet edges and 12 in. on center in the field. At the chords, the 3 in. on center spacing was achieved with two lines of pins—one line per stud flange—in a staggered configuration. Additional details regarding the configuration of the sheet steel shear walls are given in Table 1 and the sequence of wall construction is illustrated in Figure 1.

The 7/16-in. OSB shear walls were identical (4 ft. x 8 ft.) except for the spacing of mechanical fastener at the panel edges. These walls were framed with 350S162-54 studs at 24 in. on center and 350T125-43 top and bottom tracks. The OSB was attached to the framing with beads of an acrylic structural adhesive on each "contact flange" and Aerosmith 0.105 in. knurled steel pins at either 6 in. or 12 in. on center at the panel edges and at 12 in. on center in the field. The chord studs were back-to-back studs connected with the same structural adhesive used for the sheathing and steel pin at 12 in. on center through the webs. Additional details of the OSB shear walls are given in Table 1 and the sequence of wall construction is illustrated in Figure 1.

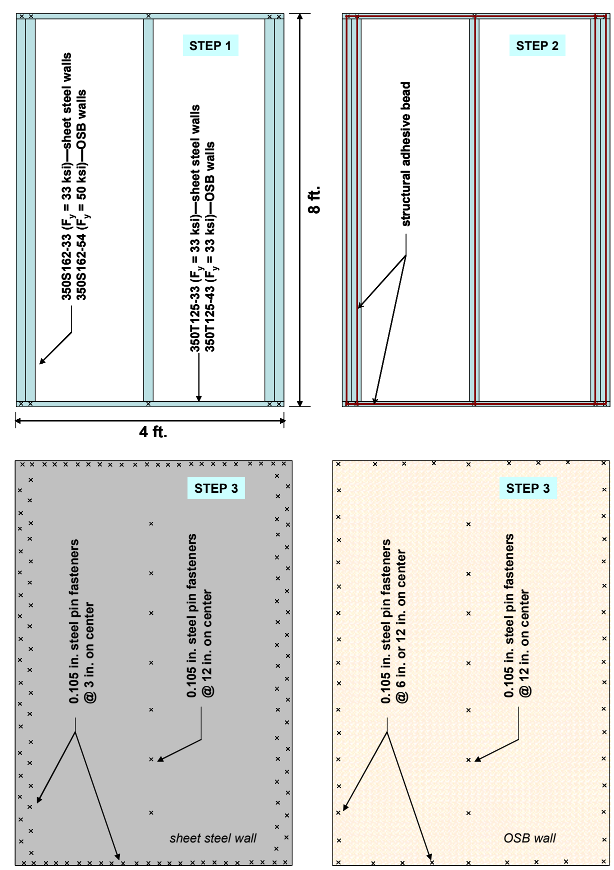
Table 1. Shear wall specimens

Specimen [1, 2]	Shear Element	Attachment of Shear Element [3]	Anchorage
2by8-TA 2by8-TB	2 ft. x 8 ft. 27 mil sheet steel (nominal F_y = 33 ksi)	0.105 in. pins at 3 in. on center at the sheet edges and structural adhesive on the contact flange of each framing member	S/HD15 at the chords (back-to-back 350S162-33 studs connected with two No. 10 fasteners at 12 in. on center)
4by8-TA 4by8-TB	4 ft. x 8 ft. 27 mil sheet steel (nominal F_y = 33 ksi)	0.105 in. pins at 3 in. on center at the sheet edges and 12 in. on center in the field; structural adhesive on the attached flange of each framing member	S/HD10 at the chords (back-to-back 350S162-33 studs connected with two No. 10 fasteners at 12 in. on center) and 3/4 in. bolts 12 in. in from each holdown
OSB6-TA OSB6-TB	4 ft. x 8 ft. 7/16-in. (24/16 span rating) OSB rated sheathing	0.105 in. pins at 6 in. on center at the sheet edges and 12 in. on center in the field; structural adhesive on the attached flange of each framing member	S/HD15 at the chords (back-to-back 350S162-54 studs connected with two longitudinal adhesive beads and one steel pins at 12 in. on center) and 3/4 in. bolts 12 in. in from each holdown
OSB12-TA OSB12-TB	Same as above	0.105 in. pins at 12 in. on center at the sheet edges and 12 in. on center in the field; structural adhesive on the attached flange of each framing member	S/HD10 at the chords (back-to-back 350S162-54 studs connected with two longitudinal adhesive beads and one steel pins at 12 in. on center) and 3/4 in. bolts 12 in. in from each holdown

[1] Studs at 24 in. on center
[2] All specimens were 4 ft. x 8 ft. (out-to-out) except 2by8-TA and 2by8-TB which were 2 ft. x 8 ft. (out-to-out)
[3] Nominal adhesive bead width was 0.1875 in.



(a) 2 ft. x 8 ft. walls



(b) 4 ft. x 8 ft. walls

Figure 1. Wall construction sequence

TEST SETUP/PROCEDURE

Each wall was tested in a horizontal position. The bottom track of the wall was attached directly to a reaction beam with holdowns on each end of the wall and 3/4-in. high strength shear bolts 12 in. in from the holdowns (for the 4 ft. x 8 ft. walls only). No shear bolts were used in the 2 ft. x 8 ft. wall tests. The holdown schedule is given in Table 1. With the bottom of the wall anchored in place, the top of the wall was attached to the load distribution member, through the wall top track, with four 3/4-in. high strength bolts. All attachments of the wall to the test frame were accomplished using a pneumatic wrench.

After a wall was installed in the test frame, displacement transducers were attached to monitor and record the wall performance. The transducers measured and recorded overturning uplift at the bottom of the wall (at each holdown), slip at the bottom of the wall, lateral displacement at the top of the wall and reaction beam displacement (see Figure 2). The resisting load was measured directly by a load cell in line with the load distribution member and the hydraulic ram.

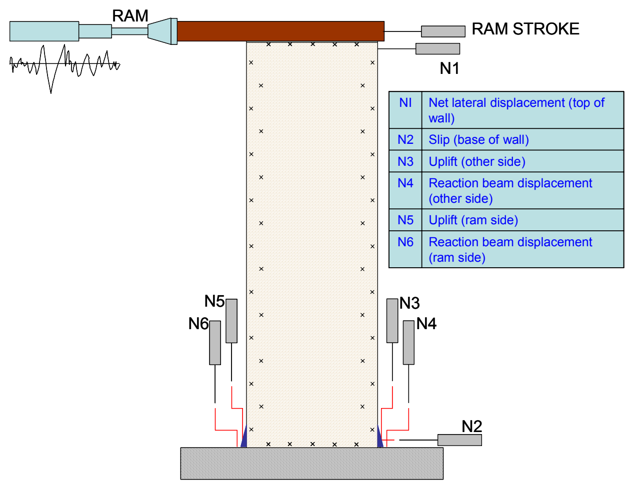


Figure 2. Instrumentation and test setup

The reversed cyclic test procedure used in this program required cycling a wall through a series of specified increasing top wall displacements/drifts (target displacements) up to 2.8 in.. Target displacements and the corresponding number of cycles at each displacement are given in Table 2. Under the current model codes (IBC, UBC and NFPA), the maximum/allowable inelastic drift for an 8 ft. wall height is limited to 2.4 in.. Thus, per Table 2, the incremental displacement from one target displacement to the next was approximately 8 percent of the model codes inelastic drift limit. During a test, the cycling frequency was held constant at 0.2 Hz (or 5 seconds per cycle), and data was sampled and recorded at a rate of 50 samples per seconds (i.e. one sample every 0.02 seconds).

Table 2. Reversed cyclic test procedure

Target Displacement, in.	No. of Cycles	Target Displacement, in.	No. of Cycles
0.2	3	1.8	3
0.4	3	2.0	3
0.6	3	2.2	3
0.8	3	2.4	3
1.0	3	2.6	3
1.2	3	2.8	3
1.4	3		
1.6	3		

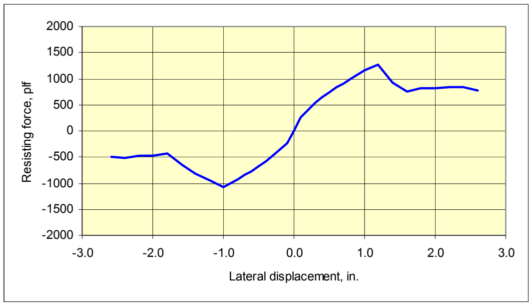
TEST RESULTS

Table 3 summarizes the failure modes, maximum resistances and corresponding lateral displacements/drifts (resistance and displacement are given as the average of the positive (pull) and negative (push) values from the peak response envelope or backbone curves) for the eight wall tests. Figures 3 and 4 show the envelope (backbone) curves derived from the hysteretic response of the sheet steel and OSB walls, respectively. The complete hysteresis response curves are given in Appendix A.

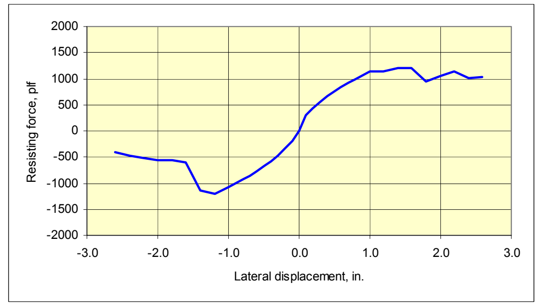
Table 3. Test results

Test No.	General Wall Description [1]	Measured Resistance: Maximum Load [2, 3], plf	Measured Resistance: Total Drift @ Maximum load, in.	Mode of Failure
2by8-TA 2by8-TB	2 ft. x 8 ft. wall with 27-mil sheet steel; pins at 3” and adhesive	1165 1207	1.094 1.296	Buckling in the chord (boundary) studs at the web punchout.
4by8-TA 4by8-TB	4 ft. x 8 ft. wall with 27-mil sheet steel; pins at 3”/12” and adhesive	1376 1121	1.092 1.099	Loss of bond between the sheet steel and the adhesive; fastener pullout from the framing.
OSB6-TA OSB6-TB	4 ft. x 8 ft. wall with 7/16-in. OSB; pins at 6”/12” and adhesive	1419 1656	0.699 0.899	In-plane (rolling) shear failure in the OSB; A combination of fastener pullout from the framing, fastener fracture and panel pullover.
OSB12-TA OSB12-TB	4 ft. x 8 ft. wall with 7/16-in. OSB; pins at 12”/12” and adhesive	1200 1532	0.699 0.895	In-plane (rolling) shear failure in the OSB; A combination of fastener pullout from the framing, fastener fracture and panel pullover.

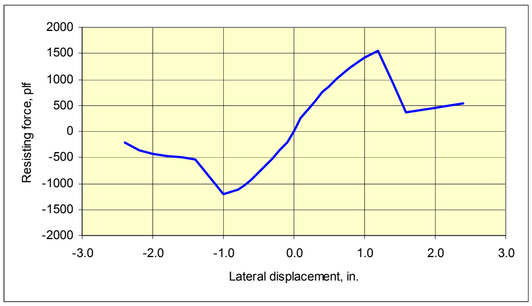
[1] Adhesive applied per Figure 1
[2] Measured resistance in lb. divided by the wall dimension parallel to the applied load
[3] Average of “push” and “pull” resistances



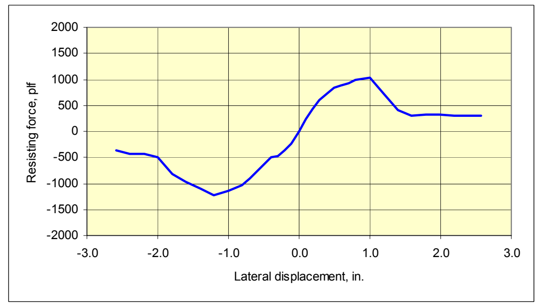
2by8-TA



2by8-TB

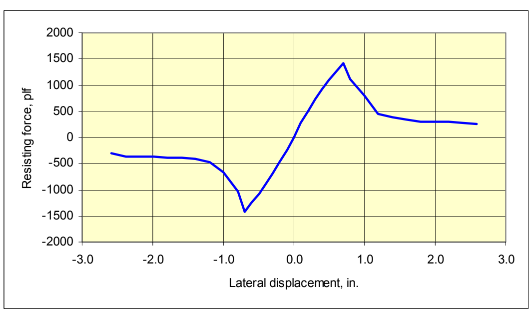


4by8-TA

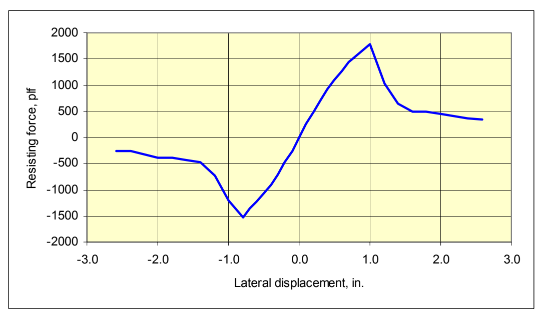


4by8-TB

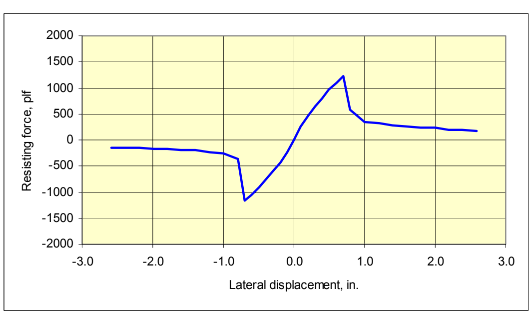
Figure 3. Resistance versus lateral displacement envelope curves for the sheet steel walls



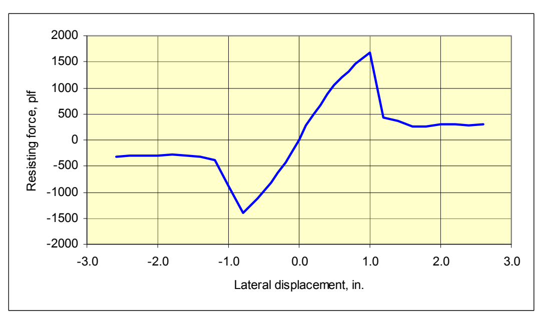
OSB6-TA



OSB6-TB



OSB12-TA



OSB12-TB

Figure 4. Resistance versus lateral displacement envelope curves for OSB sheathed walls

Sheet Steel Shear Walls: The overall response of the sheet steel walls was characterized by shear buckling and tension field action. In the 4 ft. x 8 ft. walls failure resulted from a loss the bond strength between the structural adhesive and sheet steel as the sheet buckled out of the plane of the wall. This behavior was followed by a progressive pull-out of pins from the framing, including pins at the interior studs. In the 2 ft. x 8 ft. specimens, failure resulted from local buckling in the chord studs at the web punchouts immediately above the holdowns. In this report, failure is defined by a decrease in wall resistance under increased lateral displacement/drift. Figure 5 shows the failure modes for all the sheet steel walls. In one test, 4by8-TA, bending of the top track was observed at one end of the wall. It appears that this behavior resulted from the combined effects of overall twisting of the chord studs, tension field action in the sheet steel and inadequate restraint provided by the round washer used to secure the top track of the wall to the load distribution member, at this end of the wall. When a square washer extending over a larger area of the web track was used, test 4by8-TB, top track bending was eliminated.

OSB Shear Walls: In the OSB walls, failure was observed to result from in-plane (rolling) shear in the structural panel. As shown in Figure 6, the adhesive bonded extremely well to both the steel framing and the OSB. Once bond was lost, a more sudden degradation of wall resistance was observed compared to the sheet steel walls and there was a progressive loss of resistance as a result of pin pullout from the framing, pin fracture and panel pullover.

(a) 2by8 sheet steel walls

(b) 4by8 sheet steel walls

Figure 5. Failure of sheet steel shear walls

Figure 6. Failure of OSB shear walls

INTERPRETATION and DISCUSSION OF TEST RESULTS

From a design/comparison perspective, one method of interpreting these test results is to use the criteria employed in the development of the seismic design values in the current model codes. In using this approach, it is important to keep in mind the limited number of tests conducted.

The seismic design values for CFS shear walls in the model codes are based on an assumed seismic response modification factor (R) for an expected wall behavior. The recommended design values were then interpreted independent of R. Specifically, the design values in the model codes were developed using a degraded (as opposed to peak) strength envelope as follows:

> The nominal capacity, P_{nom}, of a wall was taken as the lower of the maximum wall resistance, P_{max}, and 2.5 times the wall resistance defined by 0.5 in. of lateral displacement. The LRFD and ASD level capacities were then computed as 0.55 times the nominal capacity and the nominal capacity divided by 2.5, respectively.

Using the above method with the peak (non-degraded) strength envelope (Figures 3 and 4), the nominal, LRFD and ASD level capacities of the tested walls are summarized in Table 4.

Table 4. Interpreted design values

Specimen	P_{nom}, plf	Δ @ P_{nom}, in.	P_{LRFD}, plf	Δ @ P_{LRFD}, in.	P_{ASD}, plf
2by8-TA	*1165*	*1.094*	*641*	*0.433*	
2by8-TB	*1207*	*1.296*	*664*	*0.450*	
2by8 (average)	**1186**	**1.195**	**652**	**0.442**	**474**
4by8-TA	*1376*	*1.092*	*757*	*0.444*	
4by8-TB	*1121*	*1.099*	*616*	*0.396*	
4by8 (average)	**1248**	**1.110**	**686**	**0.420**	**499**
OSB6-TA	*1419*	*0.699*	*781*	*0.338*	
OSB6-TB	*1656*	*0.899*	*911*	*0.402*	
OSB6 (average)	**1537**	**0.799**	**846**	**0.370**	**615**
OSB12-TA	*1200*	*0.699*	*660*	*0.320*	
OSB12-TB	*1532*	*0.895*	*843*	*0.398*	
OSB12 (average)	**1366**	**0.797**	**751**	**0.359**	**546**

Per the data in Table 4, there appears to be no significant difference in capacity of the 2 ft. x 8 ft. and 4 ft. x 8 ft. sheet steel shear walls. Further, given the mode of failure in the 2 ft. x 8 ft. walls, it may be concluded that the capacity of these walls may have been higher if chord stud buckling was prevented (as required by current model codes). When the results for the OSB walls are analyzed, an apparent increase of approximately 12 percent in capacity of the wall is evident for pins are installed at 6 in. on center compared to a wall with pins at 12 in. on center.

A comparison the response curves for the 2 ft. x 8 ft. sheet steel walls in this test program, Figure 3, with the measured peak response of walls where the sheet steel is attached with No. 8 screws only (no structural adhesive), Figure 7, indicates that use of the adhesive results in a more rapid degradation in resistance after the maximum/peak resistance is attained.

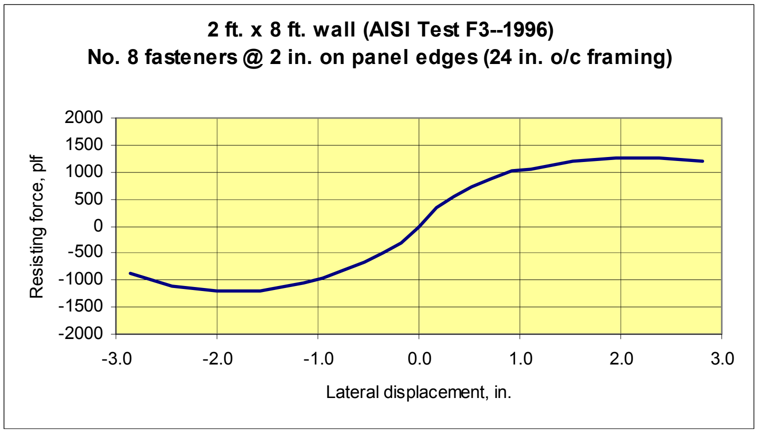


Figure 7. 27-mil 2 ft. x 8 ft. shear wall test from earlier AISI research (Serrette et al. 1997)

A comparison of the 2by8 wall performances (before buckling in the chords) with those of the 4by8 walls (see Figure 8) suggests that the stiffness of the narrower 2by8 walls was roughly the same as the 4by8 walls. One important observation made in the 2by8 tests was the fact fracture of the buckled studs from repeated reversal of load with increasing lateral displacements occurred (6 to 8) cycles after initial stud buckling.

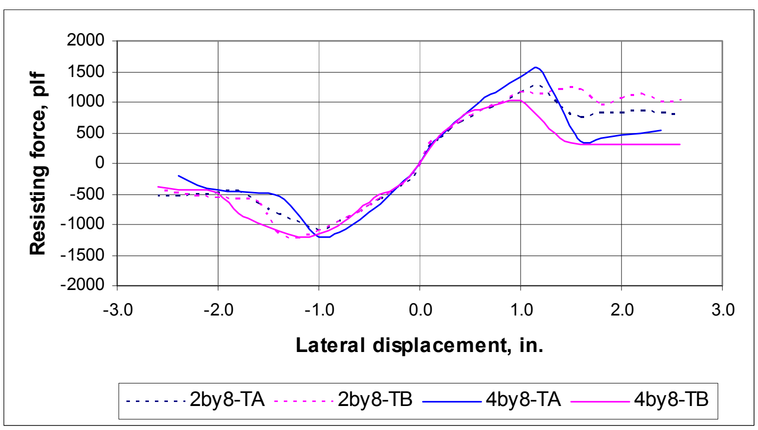


Figure 8. Comparison of sheet steel test results

An inspection of the response curves for the OSB walls indicates that the overall behavior of these walls was essentially linear elastic up to the nominal strength of the wall and there was no difference in wall stiffness for the two pin schedules. Further, although there was a rapid degradation of post-peak resistance, these walls were capable of maintaining a reduced or residual strength in the range of the ASD capacities in Table 4 (at lateral displacements exceeding 1.50 times the displacements at nominal strength). When evaluating the significance of these residual strength values it is important to note that at both ends of the wall there was a small gap between the structural panel and the test frame that permitted bearing of the sheathing on the reaction frame after the peak resistance was attained.

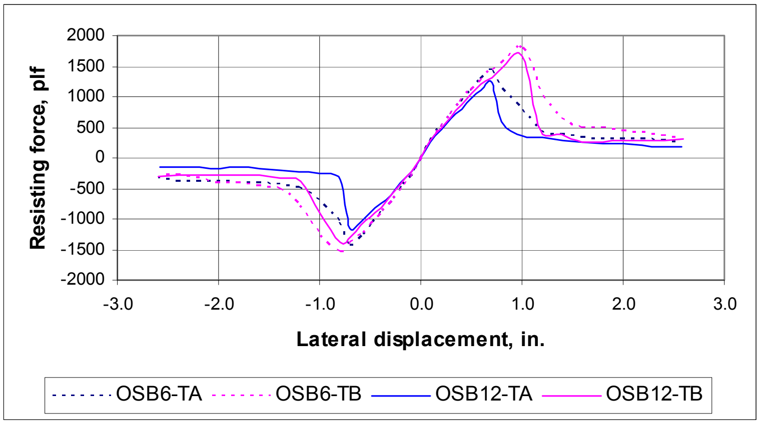


Figure 9. Comparison of OSB test results

Finally, Table 5 compares the recommended design values for these tests (Table 4) with values for similar (not identical) systems, as published in the 2003 IBC. The test to IBC values ranged from 1.04 to 2.20 suggesting that the structural adhesive application with steel pins may be a viable method for developing lateral resistance in cold-formed steel frame shear walls. For seismic design, further refinements to the interpretation of test data may be required given the rapid degradation in post-peak strength seen in these tests. These refinements will be more significant for areas of high seismicity (SDC D, E and F).

Table 5. Comparison of test data with 2003 IBC design values

Test No.	Wall Description	Nominal Resistance, plf		Test/2003 IBC
		2003 IBC [1]	Test	
2by8	Sheet steel sheathed wall with screws fasteners at 3 in. on panel edges	543 [2, 4, 5] (597) [3, 4, 5]	1186	2.18 (1.99)
4by8	Sheet steel sheathed wall with screws fasteners at 3 in. on panel edges and 12 in. in the field	1085 [2, 4] (1194) [3, 4]	1248	1.15 (1.04)
OSB6	OSB sheathed wall with screws fasteners at 6 in. on panel edges and 12 in. in the field	700 [2] (770) [3]	1537	2.20 (2.00)
OSB12	Not permitted in the 2003 IBC	--	1366	---

[1] IBC values are for applications with No. 8 self-drilling screw fasteners

[2] IBC values are based on a degraded strength

[3] IBC values increased 10% (conservatively) for expected peak (non-degraded) resistance

[4] Values interpreted, by linear interpolation, from 2 in./12 in. and 4 in./12 in. fastener schedules

[5] 50% reduction of 2:1 aspect ratio wall value for 4:1 aspect ratio wall

CONCLUSION

A series of eight shear walls (four sheet steel walls: two 2 ft. x 8 ft. walls and two 4 ft. x 8 ft. walls; and four 4 ft. x 8 ft. OSB walls) were tested to evaluate the reversed cyclic performance of cold-formed steel shear walls with structural sheathing attached using a combination of steel pin fasteners and a structural adhesive. Overall, except for the 2 ft. x 8 ft. sheet steel shear walls, the maximum resistances were governed by failure due to a degradation of the bond at the framing-sheathing interface. The 2 ft. x 8 ft. walls failed by buckling in the chord studs at the web punchouts above the holdowns.

The measured resistances exceeded values in the current model codes for similarly sheathed walls (sheathing attached with screw fasteners only). For the OSB walls, the measured responses up to the maximum wall resistances were approximately linear and this behavior was followed by a sudden degradation in strength. The sheet steel walls exhibited a more nonlinear behavior with a less severe reduction in strength after the maximum resistance. Based on these test results, the use of structural adhesives with pneumatically driven steel pins appears promising.

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APPENDIX A

Hysteresis Curves

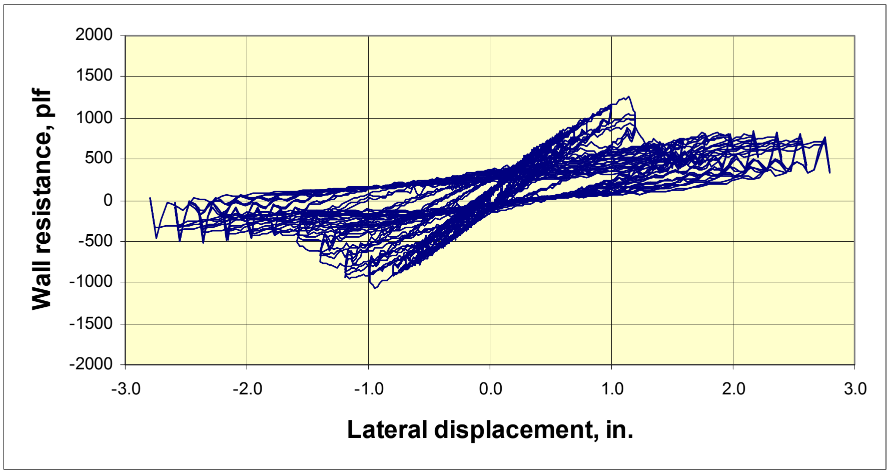

Figure A1. Hysteresis response curve for Test 2by8-TA

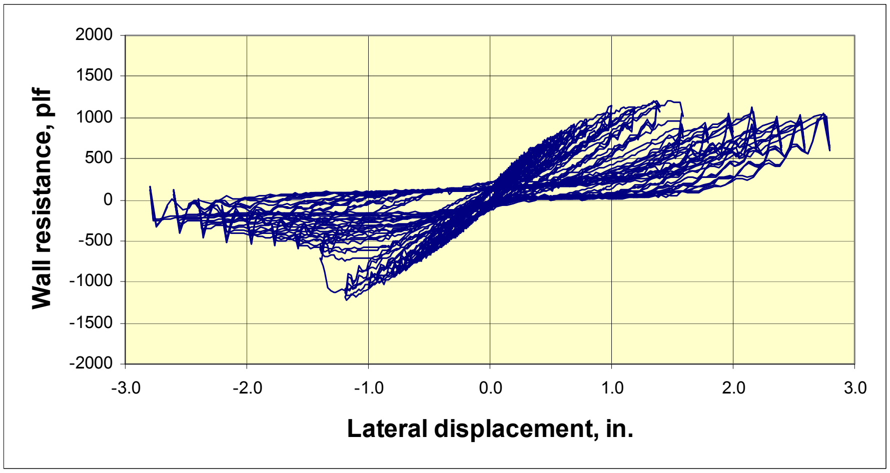

Figure A2. Hysteresis response curve for Test 2by8-TB

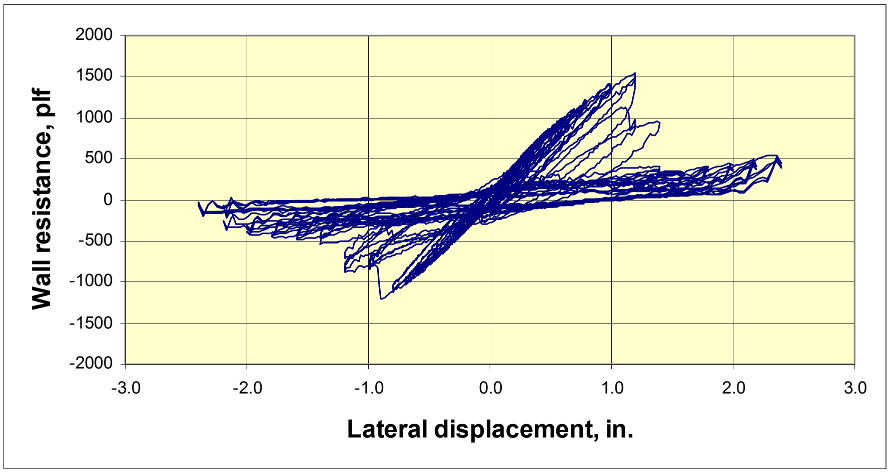

Figure A3. Hysteresis response curve for Test 4by8-TA

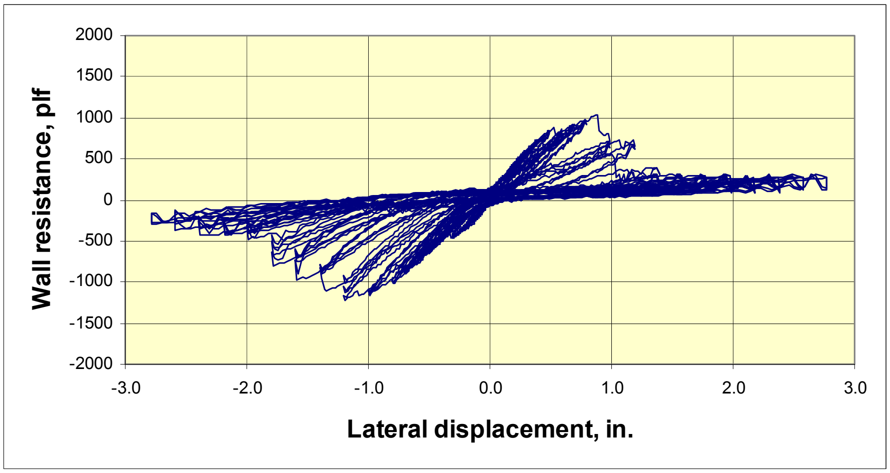

Figure A4. Hysteresis response curve for Test 4by8-TB

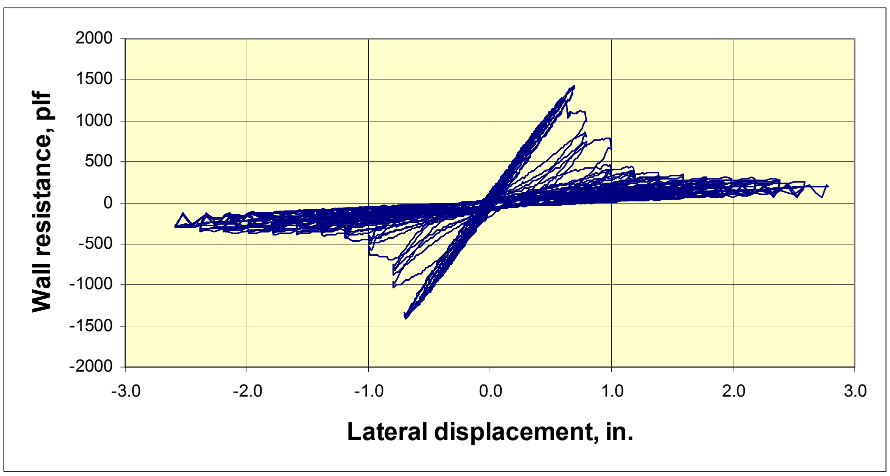

Figure A5. Hysteresis response curve for Test OSB6-TA

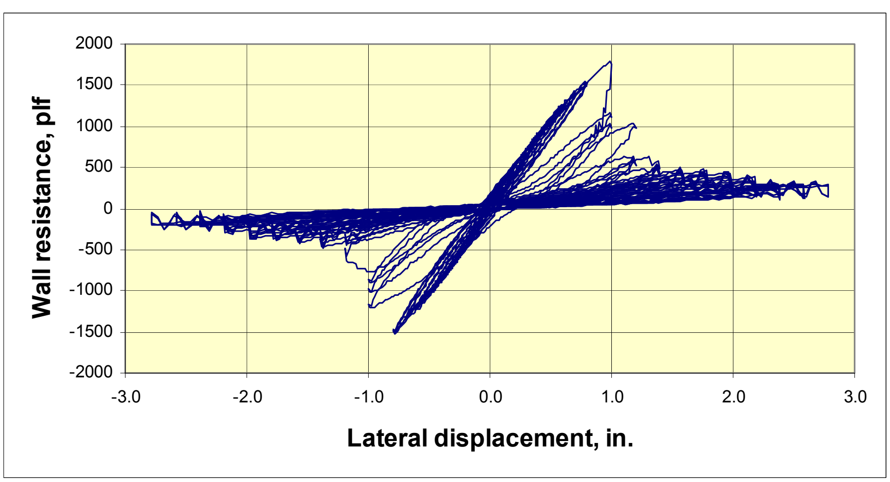

Figure A6. Hysteresis response curve for Test OSB6-TB

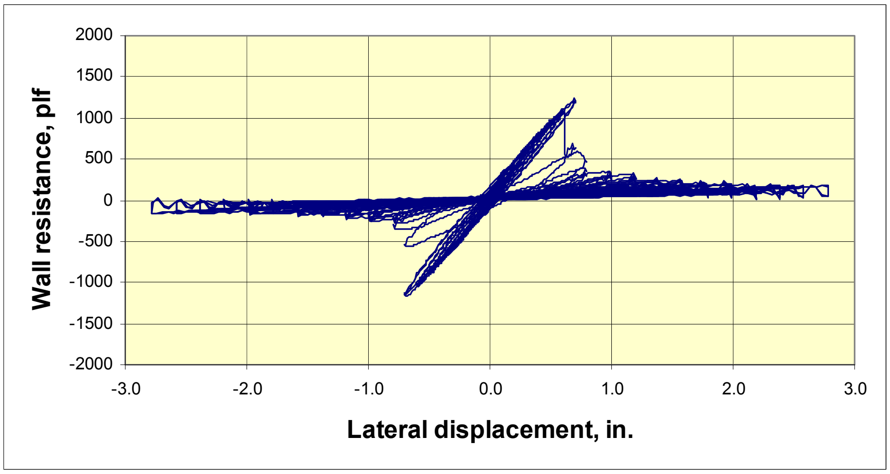

Figure A7. Hysteresis response curve for Test OSB12-TA

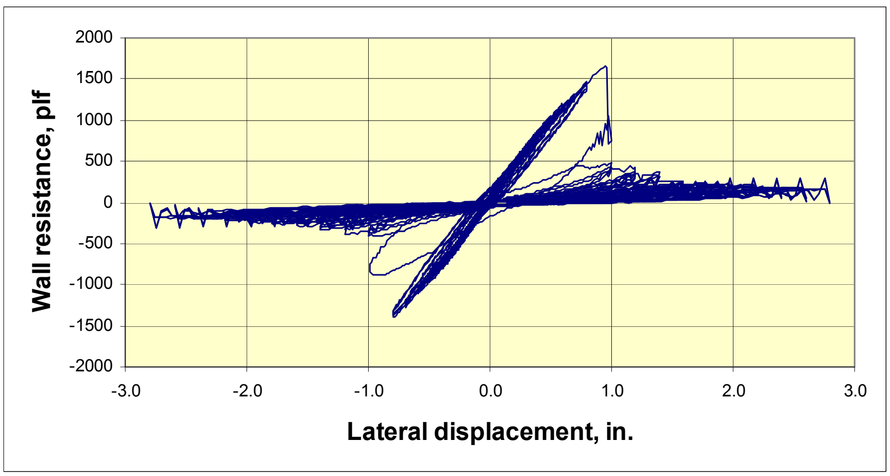

Figure A8. Hysteresis response curve for Test OSB12-TB