Cold-formed Steel Frame Shear Wall Applications with Structural Adhesives

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Abstract

This paper presents the results from a series of shear wall tests that were conducted to evaluate the performance of sheet steel and OSB structural-use panels attached to cold-formed steel framing with a structural adhesive and pneumatically driven steel pins. The walls were tested under reversed cyclic loading similar to the procedures used to develop the seismic values for cold-formed steel shear walls in the model US codes. The measured wall resistances exceeded values in the current model codes for similarly sheathed walls. The measured responses of the OSB walls, up to the peak wall resistances, were approximately linear and this behavior was followed by a sudden post-peak 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 and warrants a more comprehensive evaluation.

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Introduction

The use of adhesives to bond sheathing to light framing is not an entirely new concept. Adhesives are used in wood frame construction to address serviceability issues like diaphragm stiffness and floor squeak, for example. In addressing serviceability, there is often a beneficial increase in the strength. This increase, however, is not systematically accounted for design because of the intended use of the adhesive. As such, the structural performance of a diaphragm is governed primarily by the performance of the mechanical fastener connections. In the application presented in this paper, the role of the adhesive is reversed. The adhesive is used to provide the primary structural bond between the framing and attached sheathing for cold-formed steel shear walls with a reduced contribution from mechanical fasteners. Specifically, this report presents the results of a project that was undertaken to evaluate shear wall applications with 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.

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

Experimental Program

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) cold-formed steel frame shear walls. The eight tests comprised 4 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) panel.

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 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 in. 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 0.105 in. knurled steel pins at 3 in. on center at sheet edges and 12 in. on center in the field. Additional details regarding the configuration of the sheet steel shear walls are provided in Table 1.

The OSB shear walls were identical except for the spacing of mechanical fasteners 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 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 pins at 12 in. on center through the webs. Additional details of the OSB shear walls are provided in Table 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. shear bolts 12 in. in from each holdown
OSB6-TA OSB6-TB	4 ft. x 8 ft. 7/16-in. OSB rated sheathing (24/16 span rating)	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. shear 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. shear 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.

Test Setup and Procedure

Each wall was tested in a horizontal orientation. 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). 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.

After a wall was installed in the test frame, displacement transducers were attached to monitor and record the wall performance. The transducers (see Figure 1) 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. The resisting load was measured directly by a load cell in line with the load distribution member and the hydraulic ram.

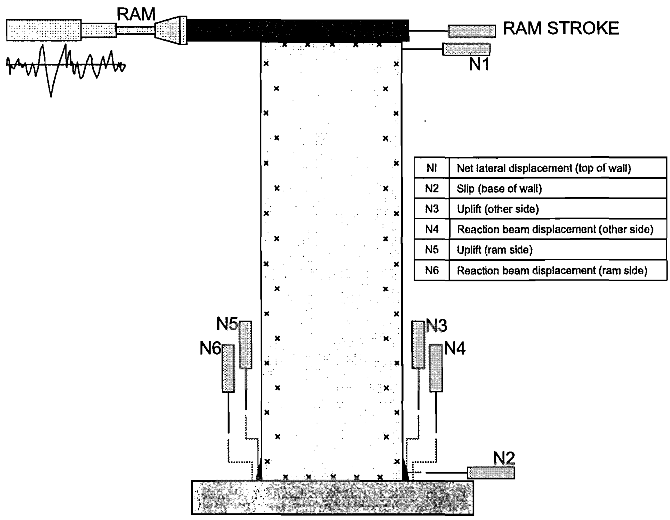


Figure 1. Instrumentation and test setup

The reversed cyclic test procedure used in this program required cycling a wall through a series of specified target displacements up to 2.8 in.. Target displacements and the corresponding number of cycles at these displacements are given in Table 2. The incremental displacement from one target displacement to the next was approximately 8 percent of the maximum inelastic drift permitted in the model codes (UBC, IBC and NFPA) for an 8-ft. wall. 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/protocol

Target Displacement, in.	No. of Cycles
0.2	3
0.4	3
0.6	3
0.8	3
1.0	3
1.2	3
1.4	3
1.6	3

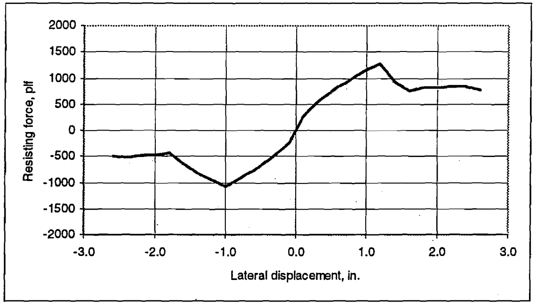
Target Displacement, in.	No. of Cycles
1.8	3
2.0	3
2.2	3
2.4	3
2.6	3
2.8	3

Test Results

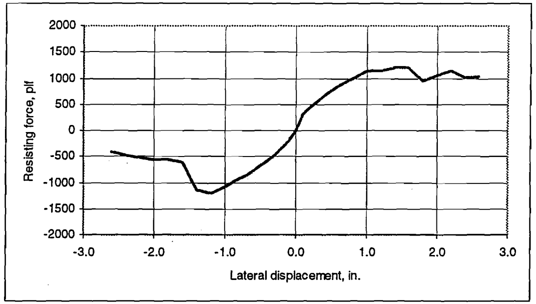
General: Table 3 summarizes the failure modes, maximum resistances and corresponding lateral displacements/drifts for the eight wall tests. The values in Table 3 are averages for the "push" and "pull" responses. Figures 2 and 3 show the envelope (backbone) curves derived from the hysteretic response of the sheet steel and OSB walls, respectively.

Table 3. Test results

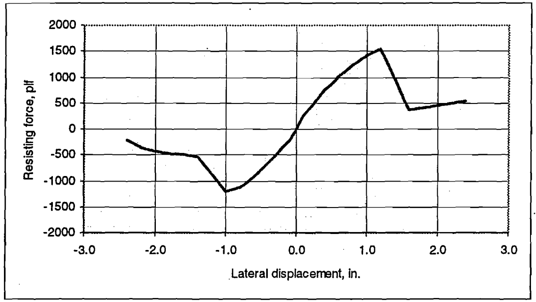
Test No.	General Wall Description	Measured Resistance		Mode of Failure
		Maximum Load, plf	Total Drift @ Maximum Load, in.	
2by8-TA 2by8-TB	2 ft. x 8 ft. wall; 27-mil sheet steel; steel pins at 3" o/c	1165 1207	1.094 1.296	Buckling in the chord (boundary) studs at the web punchout.
4by8-TA 4by8-TB	4 ft. x 8 ft. wall; 27-mil sheet steel; pins at 3" o/c at edges and 12" o/c in the field	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; 7/16-in. OSB; pins at 6" o/c at edges and 12" o/c in the field	1419 1656	0.699 0.899	In-plane (rolling) shear failure in the OSB; fastener pullout from the framing, fastener fracture and panel pullover.
OSB12-TA OSB12-TB	4 ft. x 8 ft. wall; 7/16-in. OSB; pins at 12" o/c at edges and 12" o/c in the field	1200 1532	0.699 0.895	In-plane (rolling) shear failure in the OSB; fastener pullout from the framing, fastener fracture and panel pullover.



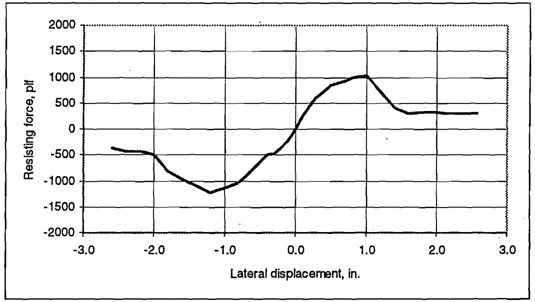
2by8-TA



2by8-TB



4by8-TA



4by8-TB

Figure 2. Resistance versus lateral displacement peak strength envelope curves for the sheet steel walls

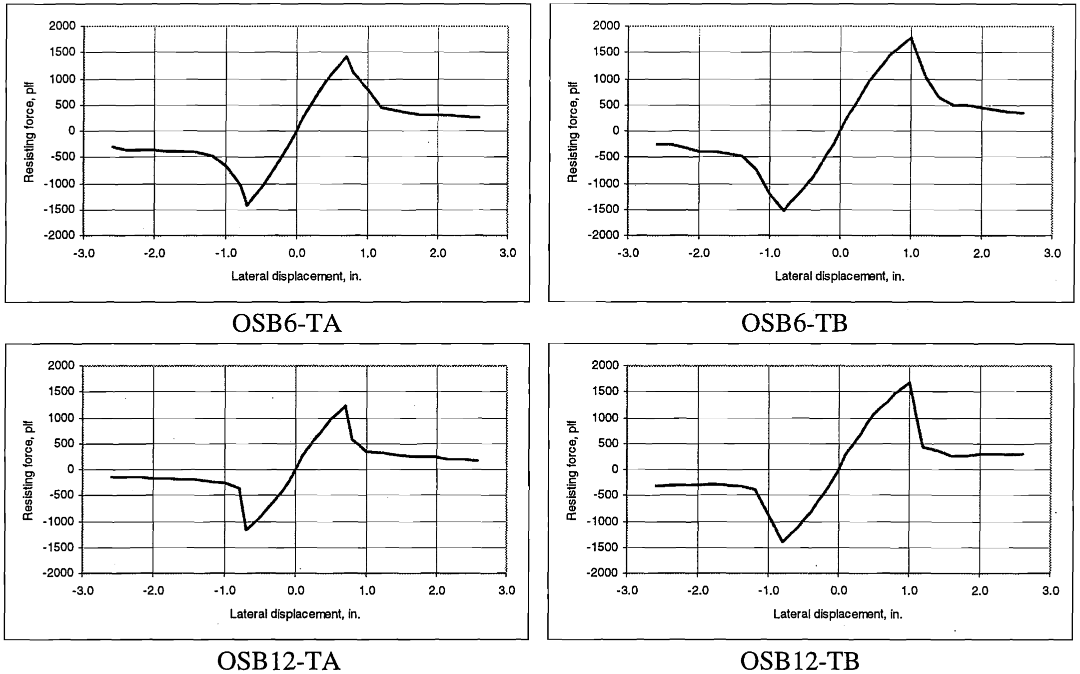
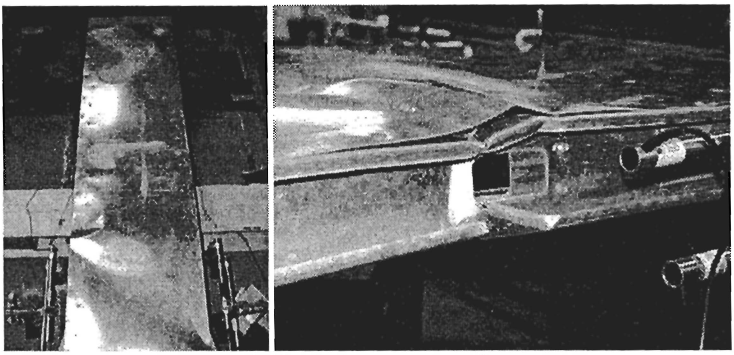


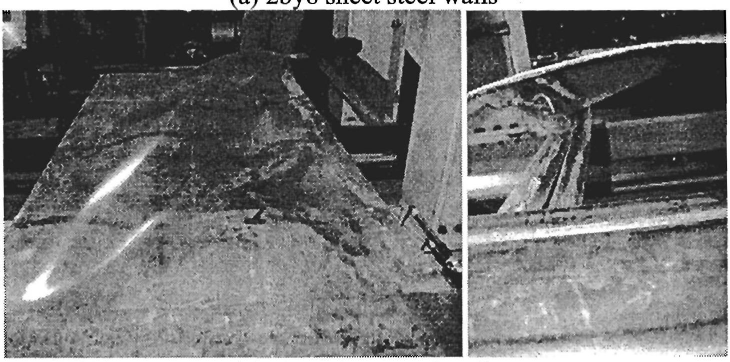
Figure 3. Resistance versus lateral displacement peak strength envelope curves for OSB sheathed walls

Sheet Steel Shear Walls: The overall response of the sheet steel walls was characterized by shear buckling/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. Figure 4 shows the observed deformation of the sheet steel walls.

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 5, 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.



(a) 2by8 sheet steel walls



(b) 4by8 sheet steel walls

Figure 4. Failure of sheet steel shear walls

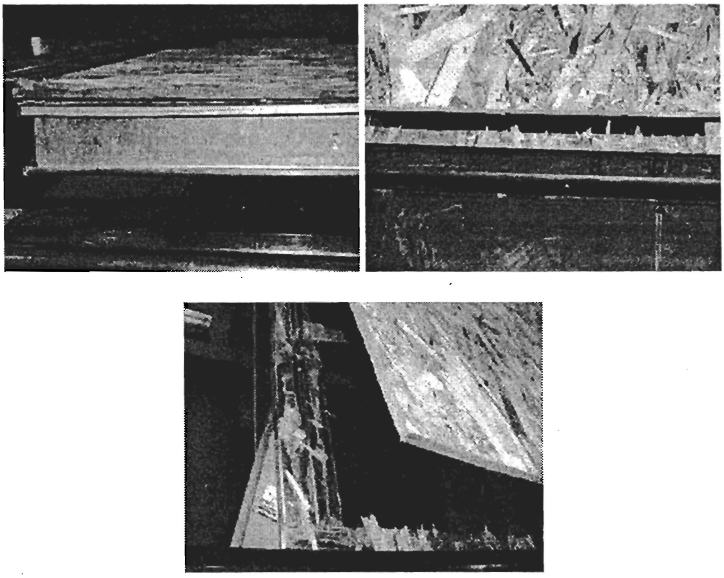


Figure 5. Failure of OSB shear walls

Interpretation and Discussion of Test Results

From a design 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 and the assumptions used in deriving the model code values.

The seismic design values for CFS shear walls in the model codes are based on an assumed seismic response modification factor (R) for light frame construction. The recommended design values were then interpreted independent of R. In addition, the design values in the model codes were developed using a degraded strength envelope as opposed to the peak strength envelopes shown in Figures 2 and 3. The nominal, LRFD and ASD level shear wall capacities were derived 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 2 and 3), the nominal, LRFD and ASD level capacities of the tested walls are summarized in Table 4.

Table 4. Interpreted design values

Specimen	$P_{nominal}$, plf	Δ @ $P_{nominal}$, 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 prohibited (as required by current model codes). When the results for the OSB walls are analyzed, an increase of approximately 12 percent in capacity of the wall was evident when pins are installed at 6 in. on center compared to a wall with pins at 12 in. on center.

A comparison of the 2by8 wall performances (before buckling in the chords) with those of the 4by8 walls (see Figure 6) indicates that the stiffness of the narrower 2by8 walls was roughly the same as the 4by8 walls.

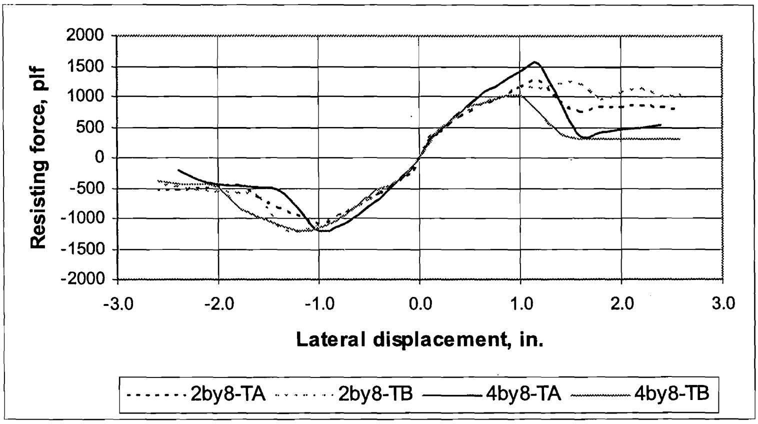


Figure 6. Comparison of sheet steel test results

An evaluation 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 different pin schedules (see Figure 7). 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 at lateral displacements exceeding 1.50 times the displacements at nominal strength.

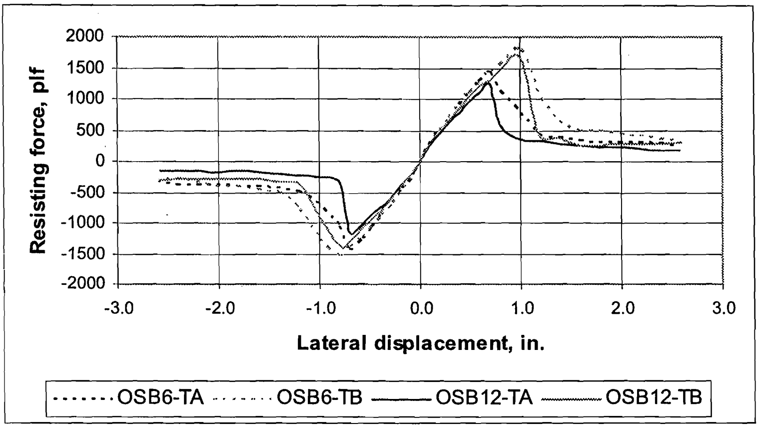


Figure 7. Comparison of OSB test results

Finally, Table 5 compares the recommended design values from these tests (Table 4) with published (IBC 2003) values for similar systems. 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 sharp degradation in post-peak strength seen in these tests.

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 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. For the OSB walls, the measured responses up to the maximum/peak 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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