STRUCTURAL PERFORMANCE OF STAPLED WOOD SHEAR WALLS UNDER DYNAMIC LOADS

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ABSTRACT :

Wood shear walls are the most common element in the lateral force resisting system of residential construction. Recent developments have made the use of staples, as a sheathing to stud connection, much more feasible and practical. Dynamic cyclic tests of wood shear walls using staples as connectors of sheathing to the studs were performed to determine load and displacement capacities. Enhanced details from standard construction were used to improve the performance of the global system including a double sill plate, a new refined panel corner detail, double staples along blocked edges, and backup bolts for hold-downs. The experiments have shown that the stapled wood shear walls with the enhanced details performed at a level above that allowed by the International Building Code-IBC (ICC 2006) regarding peak load capacity but slightly less regarding peak drift capacity.

KEYWORDS: Deflections, Ductility, Dynamic, Shear walls, Staples, Wood.

1. INTRODUCTION

The majority of testing to determine the allowable strength and ultimate capacity of wood shear walls has been performed using nails to connect the sheathing to the framing studs. Stapling the sheathing to the studs is a very common practice in many locations. In the absence of direct test results of wall panels utilizing staples as the primary connection of sheathing to the frame, building codes have used individual staple strength and test results using nailed connectors to estimate the allowable loads. The International Building Code-IBC (ICC 2006) includes allowable shear values for seismic forces for wood structural panel shear walls with stapled sheathing. For the same spacing of fasteners the allowable values for staple fasteners are approximately 2/3 of the values for nail fasteners. No design values for staples are published in the ASCE 41 Standard (ASCE 2007). However, a commonly used rule of thumb has in the past been to use twice the number of 16 gauge staples than the number of common nails required.

A review of wood shear wall testing and modeling was presented by Van de Lindt (2004). Most of the literature regarding the seismic performance of wood shear walls is focused on shear walls using nail fasteners. Crilly (2003) performed dynamic cyclic tests of Oriented Strand Board (OSB) sheathed walls with staple fasteners. A series of eleven 4 ft x 8 ft wood shear wall panels were constructed according to the 1997 UBC Code (ICBO 1997) and tested under dynamic cyclic loading to determine load capacity, stiffness, ductility and overall strength. All panels were constructed using Spruce Pine Fir 2 in. x 4 in. studs, 7/16-in. OSB sheathing attached to the frame using 2 in.-long, 16 gauge staples with a ½-in. crown at 2.5 in. on center along the panel edges, and 6 in. spacing in the field. The lateral load resisted in stapled panels with hold-down modifications was 40% greater than the allowable load specified in the National Evaluation Report 272 (NES 1997), of 314 lb/ft.

Pardoen et al. (2003) carried out an extensive shear wall test program including one-story and two-story configurations. OSB sheathing was used for most shear walls. Gun-driven 8d box nails as well as gun-driven staples were used. The staples used to attach the sheathing to the frames were 1 ⅜-in. long, ½-in. crown, 16 gauge staples. Both nails and staples were applied at 6 in. perimeter spacing. It was found that for the fully-sheathed one-story OSB-sheathed shear wall, the peak loads for the nail-fastened group and stapled-fastened group were comparable, whereas the post-peak loads for the nail-fastened shear walls were 30% higher than those fastened with staples.

2. TEST PANELS AND EXPERIMENTAL PROGRAM

2.1. Test Panels and Test Setup

The frames for all panels tested were constructed with Douglas Fir-Larch, 2 in. x 4 in. studs 8 ft in height. The top and bottom plates, and both end studs of all panels consisted of double studs. The sheathing used was 7/16 in. thick OSB sheathing grade, Exposure 1 in accordance with US DOC PS2 (IBC 2006). The staples used to attach the sheathing to the frames were 2 in. long, ½-in. crown, 16 gauge galvanized staples. The IBC Code (ICC 2006) specifies the minimum crown width of staples to be used in shear walls as 7/16 in. and the minimum fastener penetration in the framing member as 1 in., so the minimum staple length for 7/16 in. sheathing is 1 7/16-in. The 2 in.-long staples used in these tests exceed that minimum, thereby reducing the potential of any staple withdrawing during the tests. This size of staple is commonly available. The length of the staples used in this study is longer than the 1 ⅜-in. staples used in the study by Pardoen et al. (2003).

The RSP4 type 1 connector, manufactured by Simpson Strong-Tie (SST 2007), which has an allowable uplift load of 315 lbs as listed in the manufacturer's catalog, was used to connect the studs to the sill plate and thereby inhibit the interior studs from lifting off the bottom plate. This was done to help simulate the effects that gravity loads would have on a wall in a real life application. The RSP4 connector also reduces the eccentricity that sheathing forces have relative to the anchor bolts at the bottom plate. The hold-downs used in the tests for all panels were PHD5's, manufactured by Simpson Strong-Tie (SST 2007), that have an allowable tension load of 4.7 kips. This eccentric hold-down is attached to the foundation with a ⅝ in. diameter rod, and connected to the inside edge of the end post of the shear wall, with 14 screws, ¼ in. in diameter by 3 in. long. This length of screw penetrates into both the inside and outside end-post studs. This allows for the load to be more evenly distributed to the hold-down from the dual stud end-post. The anchor bolts had a ⅝ in. diameter. The washer for the anchor bolts was a 3 in. x 3 in. x ¼ in. steel plate.

In addition to the hold-downs, a strap was used to minimize the amount of bending of the end-post, due to the eccentricity of the hold-downs. A flat strap, ST6224, manufactured by Simpson Strong-Tie (SST 2007) was used at both ends of the wall panel, for all panel lengths, as shown in Fig. 1(b). The straps are made of 16 gauge metal, 2 1/16 in. wide and 23 5/16 in. long. On each end of the strap there are pre-punched ½ in. holes, and smaller pre-punched holes along each side that were used to attach the strap to the end post. The strap accommodates a total of 28 nails when placed in a straight line configuration; the strap was applied at the bottom corners, and the bottom half of the strap was attached using the anchor bolt, thus only 16 to18 nails were used to attach the remaining portion of the strap to the wall. The manufacturer calls for 16d nails to be used, but for this application it was decided that Simpson Strong-Tie N10 nails (SST 2007) would be used. The nails are a 9 gauge smooth shank nail, 1½-in. long. The pre-punched ½ in. diameter end hole was enlarged to fit over the 5/8 in. hold-down anchors on either panel end. Each strap was set in place before the panel was set onto the concrete foundation. The panel was anchored with the PHD5's, each strap was bent up, pulled tight around the beveled corner of the panel, and nailed flush to the outside of the double 2 in. x 4 in. end-post.

Twelve panels were tested, which were constructed in a similar manner. Fig. 1(a) shows the constructed panel frames with the end-posts. The top and bottom plates consisted of two 2 in. x 4 in. studs face nailed together. The interior studs were cut to a length of 90 in., and located 16 in. on-center for all the panels, except for the 3 ft long panels, where the interior stud was placed at 18 in. The vertical members were end nailed with two 10d common nails to the inside 2 in. x 4 in. stud of the top and bottom plates. The second 2 in. x 4 in. plate member was face nailed to the top and bottom plates with 10d common nails at 6 in. on-center. The panel hold-down anchors and shear anchor locations are shown in Fig. 1(a). The concrete foundation had a 12 in. x 12 in. cross-section, as shown in Fig. 2(b). Figure 1(b) shows the panel modification of cutting the bottom stud of the bottom double plate at both ends on a 45-degree angle. In previous tests by Crilly (2003), the tension on the strap that was wrapped around the square corner would crush the corner of the sill plate as the panel rocked back and forth; as a result of this crushing, the overall panel deflection increased, because as the corner was crushed, the extra slack in the strap allowed the wall to deflect further before the strap could effectively resist the force applied to the wall, and limit deflection. The strap force reduces the eccentricity on the double end stud as shown in Fig. 1(b).

The panel sheathing was attached using a stapling pattern of 2 in. on the edges, and 4 in. in the field. The sheathing for the 3-ft and 4-ft panels was placed vertically; the sheathing for the 8-ft and 12-ft panels was placed horizontally. The sheathing was blocked using Douglas- Fir Larch 2 in. x 4 in. blocks cut 14 in. long, and placed with the 3 ½ in. flat against the sheathing joint (Fig. 2a). Two rows of staples were placed along the perimeter of the panels where double studs were located, so each row had staples placed at 4 in. on-center, which provided the equivalent spacing of 2 in. Because of the tight nailing pattern, the IBC Code (ICC 2006) requires that framing at adjoining panel edges shall be 3 in. nominal or wider; 3 in. x 3 in. studs were placed as vertical studs in the 12-ft panels, as shown in Fig. 1(a).

Figure 2(b) shows the test frame setup that was built within the multi-purpose frame of the University of Utah Structures Laboratory. It was made up of steel wide flange columns and beams rigidly connected. The frame was also braced at the end where the actuator was located. The actuator was connected to a column and applied the load to the panel through an I-beam, which was also attached to the top of the wall panel. Figure 2(b) shows the load distribution I-beam, which was an S 4x7.7 section with a 3 5/8 in. x 3/16 in. steel plate welded to the bottom flange. The I-beam was attached to the top of the panels with two rows of ⅜ in. diameter A-307 bolts; the bolts were side by side 6 in. on center for the first 5 ft closest to the actuator, and 6 in. on center at alternating sides thereafter. The test assembly utilized two guide beams, which hung from the top beam of the load frame, ("END VIEW" in Fig. 2b) and each guide beam was hung by two ½ in. diameter threaded rods. Both guide beams were restricted from lateral movement with cantilever braces attached to the interior columns of the load frame. This limited out-of-plane movement of the walls during testing. Each panel was instrumented with four displacement transducers (DT). Figure 2(b) shows where each DT was located and its assigned number. DT-1 measured the vertical uplift displacement of the west panel edge form a fixed reference point on the concrete base, DT-2 measured the panel's vertical uplift displacement at the east panel edge from the concrete base, DT-3 measured the amount of slip the panel experienced at its base, and DT-4 measured the displacement of the top of the wall. The data acquisition system for the actuator and the DT's was programmed to record data at a rate of 50 data points per second.

2.2. Test Procedure

The panels were tested using cyclic loads in accordance with SEAOSC's "Standard Method of Cyclic Load Test for Shear Resistance of Framed Walls in Buildings" protocol (SEAOSC 1997). The dynamic load was applied at a rate of 1 cycle per second. Figure 3 shows a typical displacement protocol. The loading protocol has a significant effect on the response of shear walls (Gatto and Uang 2002). The protocol used in this research (SEAOSC 1997), was found to result in a 25% average reduction in peak strength and 47% average reduction in deformation capacity compared to the CUREE-Caltech standard protocol (Krawinkler et al. 2001) in a study carried out by Gatto and Uang (2002) using 8 ft x 8 ft wood shear walls with nail fasteners.

4. EXPERIMENTAL RESULTS AND COMPARISON WITH OTHER STUDIES

Figure 4 shows the hysteresis loops and the SEAOSC bilinear curve for panel 8-C, which is representative of all panels tested (Talbot 2007). Testing was stopped when the shear wall could not resist significant horizontal loads; that is, the load resistance had essentially been destroyed. Typically this occurred shortly after the peak horizontal load.

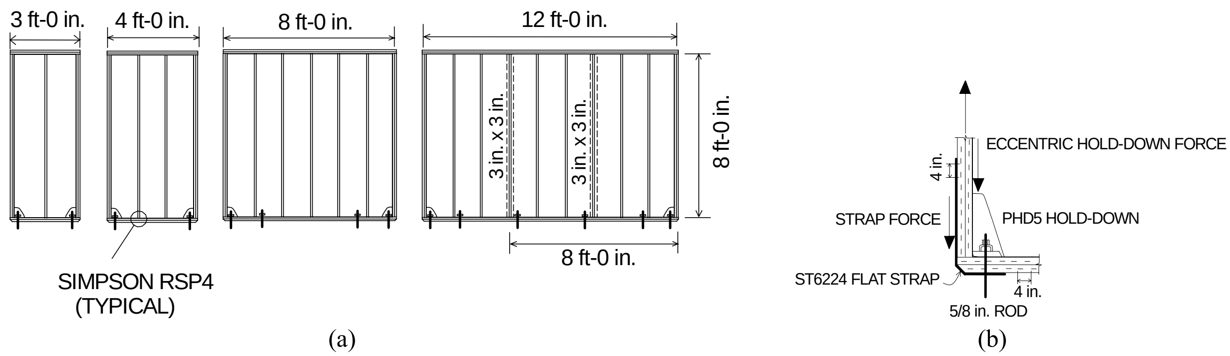


Figure 1. Panel framing layout specifications: (a) 3-ft, 4-ft, 8-ft, and 12-ft panels; (b) end-post detail.

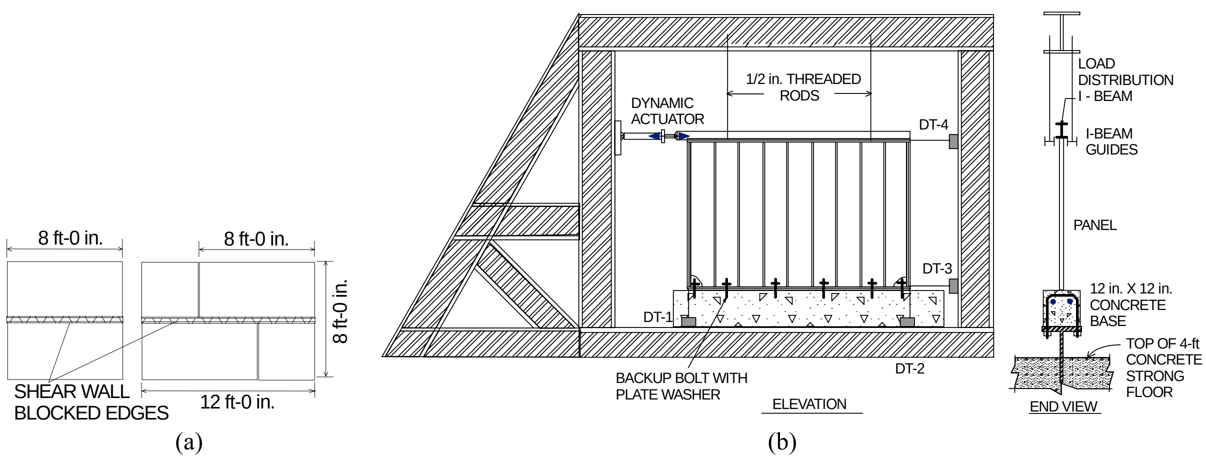


Figure 2. Test assembly and setup: (a) sheathing orientation and layout; (b) test setup.

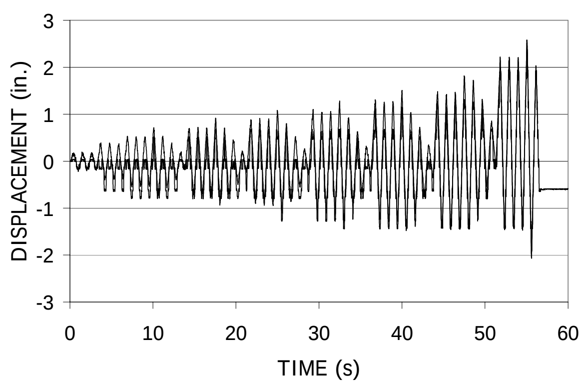


Figure 3. Applied displacement protocol for panel 8-C.

The SEAOSC report (SEAOSC 1997) states that a bilinear curve is to be generated from the hysteresis loops to determine shear wall properties. The SEAOSC report defines a limit state as an event, which marks the demarcation between two behavior states, when the behavior of the element or system is altered significantly. The first point on the bilinear curve is the yield limit state (YLS), where the difference in the forces at the first and fourth cycle, at the same displacement, does not exceed 5%. The second point is the strength limit state (SLS), defined as the point in the force-displacement relationship, corresponding to the maximum displacement at the peak force attained by the element or system. Table 1 describes the ultimate failure mode of each of the panels tested and location of the failure, and the average peak strength and displacement capacity. The 3-ft panels acted like a beam fixed at the base, with the load applied at the free end. In most of the tests, staple progressive withdrawal was the predominant failure mode; this was followed by separation of the sheathing from the framing members. The specimens had a withdrawal failure due to uplift of the sheathing relative to the bottom plate. No fatigue failure of the staple itself was observed. Instead, the large number of loading cycles caused progressive withdrawal of the staples. This is demonstrated in Fig. 5(a) for panel 3-C where progressive withdrawal of the staples occurred from bottom to mid-height, and in Fig. 5(b) for panel 4-C where progressive staple withdrawal occurred mainly at the bottom corner. The end–post metal strap is shown intact in both figures. Panels 8-A and 8-B had ultimate failures occurring at the bottom corners of the panels, once blocking line failures had occurred. Blocking failure occurred due to differential movement of the sheathing in the horizontal direction.

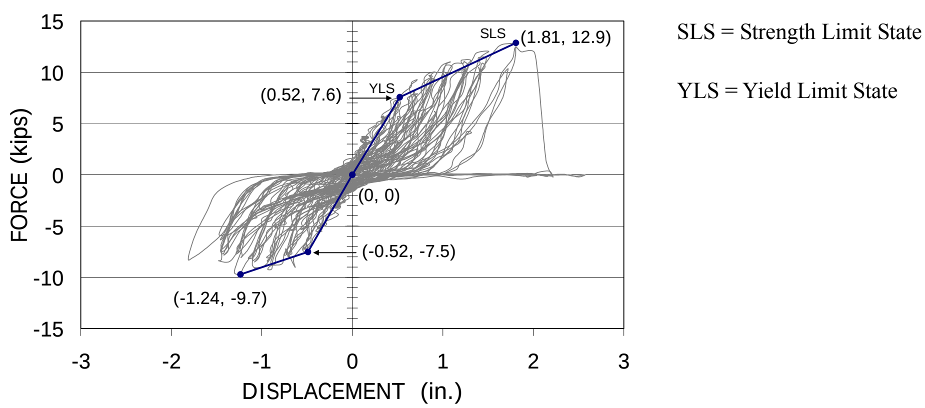


Figure 4. Hysteresis curve for panel 8-C and SEAOSC bilinear curve.

(a) (b)

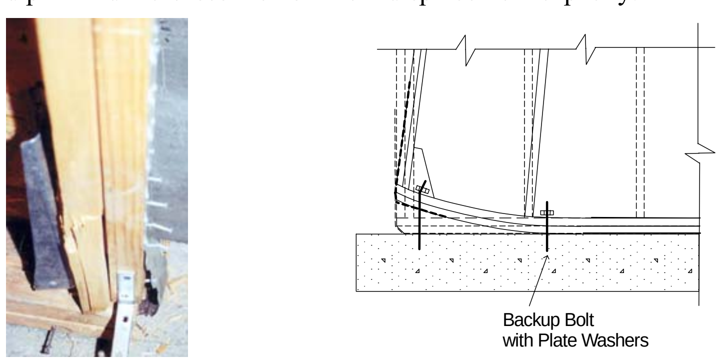
Figure 5. Staple withdrawal failure mode of panels: (a) 3-C, (b) 4-C.

The allowable shear load for the panels tested is 395 lb/ft, except for the 3-ft wall for which it is 296 lb/ft, as listed in the IBC Code (ICC 2006). This allowable load is for panels with an edge staple spacing of 2 in. All panels were stapled according to the above spacing schedule. The blocking for panel 8-C, and 12-A through 12-C was applied with twice as many staples in the blocking region with four instead of two rows of staples. Table 1 shows that the average peak strength increased from 9.7 kips for panel 8-A and panel 8-B to 11.3 kips for panel 8-C with the increased staples in the blocking region. The extra staples improved the resistance of the wall. The failure of panel 8-C was an anchor rod failure and splitting of the double sill plate due to cross-grain bending, as shown in Fig. 6(a). It is clear that there is an eccentricity (e) between the sheathing and anchor rod forces, as shown in Fig. 6(b) that contributes to sill plate damage. Due to the eccentricity, the sheathing uplifts relative to the bottom plate, which causes cross-grain bending and sill splitting; this sheathing uplift also causes progressive withdrawal of the staples. Splitting of the sill plate was observed during the 1994 Northridge earthquake (EERI 1994), in tests of nailed walls by Gatto and Uang (2002) who used single sill plates, and in tests of stapled walls by Crilly (2003). Furthermore, full-scale shake table tests of wood-frame houses subjected to the 1994 Northridge earthquake at the State University of New York at Buffalo (Filiatrault et al. 2007) have shown that the simulated earthquake severely damaged the sill plate.

(a) (b)

Figure 6. (a) Anchor rod failure and sill plate damage for panel 8-C; (b) hold-down detail.

The failure mode of the 12-ft panels involved progressive staple withdrawal and splitting of the top plate, and strap, anchor, and end-post failure at the hold-down, as shown in Fig. 7(a) for panel 12-B. Cross-grain bending of the top plate was also evident due to eccentricity between the sheathing and the 3/8 in. diameter A-307 bolts. No failure was observed involving the RSP4 sill plate connector in any of the tests. The end-post metal strap is shown buckled in Fig. 7(a). The PHD5 hold-downs used in the tests along with the backup bolt resisted a tension created by the overturning moment ranging from 2.1 to 3.8 times the allowable tension load of 4.7 kips, as listed in the manufacturer's catalog. The backup bolt with plate washers served as a force helping to hold down the bottom double plate; the plate bends upwards at the end and assists the hold-down device in resisting the panel overturning, as shown in Fig. 7(b). Regardless of how tight the hold-downs and shear anchors were at the beginning of the test, they were all loose at the end of the test. Loosening of the nuts occurred due to either crushing of the sill plate under the washers or hold-downs, or the working motion applied to the concrete wedge anchors which tried to pull them out of the foundation. Figure 7(b) shows that the backup bolt with plate washers provides an alternate load path and increases the nonlinear displacement capacity.



(a) (b)

Figure 7. (a) Strap, anchor, and end-post failure at the hold-down for panel 12-B; (b) backup bolt with plate washer hold-down (exaggerated displacement).

Table 1. Panel failure modes

Panel ID	Length (ft)	Failure Mode (East End)	Failure Mode (West End)	Average Peak Strength (kips)	Average Displacement (in.)
3-A#1	3	Staple partial withdrawal	Staple partial withdrawal	3.9	2.2
3-A#2	3	Staple withdrawal at bottom corner	Staple withdrawal at bottom corner	3.9	2.6
3-B	3	Initial staple withdrawal, and sheathing separation from framing at end-post	N/A+	4.0	2.4
3-C	3	Initial staple withdrawal, and sheathing separation from framing at end-post	Initial staple withdrawal, and sheathing separation from framing at end-post	3.7	2.6
4-A	4	Initial staple withdrawal, and sheathing separation from framing at end-post	Initial staple withdrawal, sheathing separation from framing at end-post	5.5	2.2
4-B	4	N/A	Anchor rod pullout from concrete	5.2	2.0
4-C	4	Staple withdrawal at bottom corner, and end-post failure	Staple withdrawal on end-post	4.9	2.2
8-A	8	Staple shear at blocking	Staple shear at blocking	9.0	1.5
8-B	8	Staple shear at blocking	Staple shear at blocking	10.4	1.3
8-C**	8	Anchor rod failure and splitting of sill plate	N/A	11.3	1.8
12-A**	12	Staple withdrawal and splitting along top chord	Staple withdrawal and splitting of top chord sheathing	21.3	2.1
12-B**	12	Strap hole at anchor and end-post at hold-down	N/A	18.8	2.0
12-C**	12	Staple withdrawal, and sheathing separation from framing at end-post	Staple withdrawal, and sheathing separation from framing at end-post	19.9	2.1

+ Not Applicable ; ** Double spacing of staples along blocking line as shown in Fig. 8(b)

5. CONCLUSIONS

Stapled wood shear walls of better than standard construction were tested under dynamic cyclic loads. A series of 8-ft high panels with lengths of 3-ft, 4-ft, 8-ft, and 12-ft were tested. The tests show that staples are a viable means of assembling wood shear walls with the details provided herein to resist seismic forces. The advantage of staples over nails is that staples can be placed much closer to each other, which allows designers to respond specifically to areas of increased demand without splitting the host wood members, which is a problem with closely spaced nails. The shear walls had upgraded details including a double bottom plate with end-post metal straps, a backup anchor bolt with a washer plate, and an increased number of staples along blocked joints. Addition of the end-post metal straps to the bottom corners effectively reduced the eccentric forces on the end-posts, developed due to the hold-downs. The straps reduced the bending moment in the jamb members and the forces that the connectors were required to resist; this postponed failure to higher loads. Beveling of the bottom panel corners eliminated crushing of the corners. A comparison with stapled wall tests carried out by Crilly (2003), which were otherwise identical to those tested in this research, shows an improvement of 30% in the strength limit state when the hold-down detail is used with the special wall system. The backup bolt with steel plate washers was effective in helping to hold down the bottom double plate from bending upwards, thereby increasing the non-linear capacity of the walls. The hold-downs used in the tests along with the backup bolt resisted a tension created by the overturning moment ranging from 2.1 to 3.8 times the published allowable tension load capacity of the hold-down.

The predominant failure mode was progressive staple withdrawal followed by separation of the sheathing from the framing. Staple shear at blocking was rectified by doubling the number of staples at the blocked joints; this increased the peak strength by 10%. Splitting of the sill plate was another failure mode that was also observed along the top double 2 in. x 4 in. framing of the wall. The stapled wood shear walls with the enhanced details performed at a strength level above that allowed by the IBC (ICC 2006) by at least a factor of 1.5 at the yield limit state and 2.8 at the strength limit state. However, the stapled wood shear walls achieved only 88% of the allowable drift capacity of 2.5%. The authors believe that this result is due to the loading protocol used in this research (SEAOSC 1997).

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