4. SPECIMEN SPECIFICATIONS

4.1 Materials

The sheathing used for the shear wall panel tests was a 4′ wide x 8′ long x 7/16″ thick oriented strand board (OSB). The sheathing was American Plywood Association (APA) rated for exposure 1. OSB was selected since it is a common sheathing used in Utah and there were fewer tests performed in previous research with OSB than with plywood. In order to construct the 4′ x 8′ panels, the sheathing was stood on its 4′ edge and stapled to the perimeter studs with panel edge spacing and was then stapled to the interior studs with field spacing.

The studs used were stud grade spruce pine fir (SPF) studs. Some panels were constructed earlier than others. The panels built in 1999 had been stored inside for more than 2 years. The panels built in 2001 were only stored inside for a few days. On panel 5 the studs were well above 19% moisture content and the effect of this moisture was noticeable in the wall's performance. This panel was constructed with wet studs and tested within 24 hours.

Panels with double studs had the double end-wall studs and double plate studs "face" nailed together with 10d common nails (0.148″ diameter x 3″) at 6″ on-center. The plates were end nailed into the vertical studs with two 10d common nails. All common nails were hand driven.

The staples used were manufactured by Paslode and they were 16-gauge with a ½″ crown width and 2″ long. The minimum penetration per NER 272 for a 16-gauge staple is 1″ . The staples provided an additional ½″ of penetration into the studs. The staple length was selected specifically to determine if fastener withdrawal would occur during the tests with the longer penetration.

The hold-downs selected for this test were Simpson PHD2s. These hold-downs were selected because they are eccentric hold-downs that are predeflected during manufacturing to minimize material deformation. The PHDs are screwed into the end-wall studs that eliminates deflection found in hold-downs that are bolted into the end-wall studs. The screws used with the Simpson PHD hold-down are ten ¼″ diameter by 3″ long wood screws that penetrate into the outside end-wall studs 2 7/8″ minimum.

Anchor bolts used for both the hold-downs and the sill anchor bolts were 5/8″ diameter all-thread that was epoxied into the foundation with Epcon C6 epoxy. This epoxy set up in 1 hour and all 11 tests were performed with the same threaded-rod anchors without one anchor failure. Each 4′ panel had three anchor bolts and two hold-down bolts. The threaded rods used for the anchor bolts and hold-downs were A-307 zinc-coated, threaded-rod. The steel straps used to connect the end-wall studs and the anchor bolts beneath the sill plate were ST6236 flat straps manufactured by Simpson Strong-Tie Company. The straps were 2 1/16″ wide, 14 gauge galvanized steel with predrilled nail holes for 0.162″ diameter nails, and one predrilled ½″ diameter hole. This hole was drilled to fit the 5/8″ diameter threaded rods used for the hold-down anchor. In order to install the straps the hole in the end of the strap was slipped over the threaded rod used for the hold-down anchor bolt. The strap was laid flat while the panel was placed over it. The strap was then bent around the edge of the sill plate, and face nailed into the end-wall studs with Simpson N10 (0.148″ diameter x 1½″ long) hand driven nails. Figures 4.1 and 4.2 show how the strap was installed.

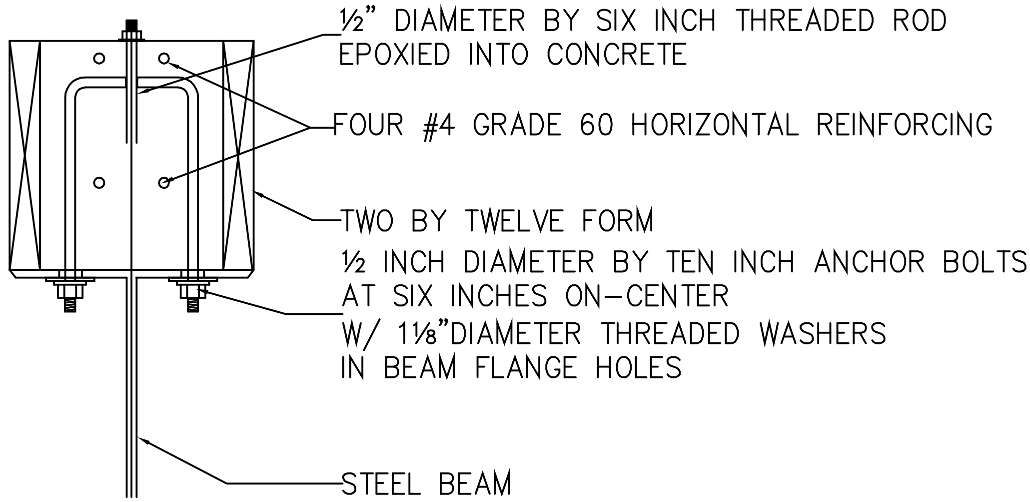
The panels were anchored to a concrete foundation that was approximately 12″ wide and 12″ thick cast onto a steel wide-flange beam. The beam was an integral part of the load frame. Cast into the concrete were ½″ diameter x 10″ long J-bolts that were threaded into 1 1/8″ diameter spacers and then bolted from below to prevent uplift. The spacers were specially machined for a snug fit in the existing holes in the steel beam's flange. These spacers prevented lateral movement of the concrete foundation as well as any horizontal sliding between the concrete and the steel frame, and they also allowed the concrete foundation to be removed and relocated by setting it back down onto the beam with the spacers fitting back into the holes. The foundation was reinforced with four No. 4 reinforcing bars longitudinally in the concrete. Figure 4.3 shows a cross-section through the concrete foundation.

4.2 Assemblies

Table 3.1 listed the different test panels and their properties. The first four test elements were part of a baseline set (panels 1, 2, 3, & 5). This set consisted of four 4′ x 8′ panels, sheathed with 7/16″ OSB, and fastened to 2″ x 4″ studs spaced at 16″ on-center with single top and bottom plates and double studs on the wall ends for hold-down installation. The double end-wall studs were representative of having a trimmer stud and a king stud adjacent to an opening. Sheathing fastening was performed with 16 gauge staples spaced at 2 ½″ on-center along the panel edges, and 6″ on-center field nailing. Two rows of staples were installed along the sides of the panel to each of the double end- wall studs.

Figure 4.1. End-wall strap before installation.

Figure 4.2. End-wall strap installed.



LIST OF MATERIALS:

(FOR FOUNDATION)

QUANTITY	MATERIAL
(28)	½" DIAMETER BY TEN INCH "J" BOLTS
(28)	1⅛" BY x ½" THICK TREADED WASHERS
(28 LINEAR FEET)	#4 GRADE 60 REBAR
(14)	½" X 6" THREADED ROD
(4)	2"X12" BY 8' LUMBER

Figure 4.3. Concrete foundation section.

This stapling pattern and sheathing thickness has an allowable load of 314 plf per NER 272. Limiting the staple spacing to 2 ½″ on-center eliminated the building code requirement for 3x-boundary members at panel edges and abutting sheathing joints per NER 272. This stapling pattern also represents design conditions found in typical single-family homes. Simpson PHD2 hold-downs were screwed into the double end-wall studs with Simpson SDS3 screws that were provided with the hold-downs. The hold-downs were then fastened to the 5/8″ diameter all-thread anchor.

The next set of test elements (panels 4 & 6) were identically constructed to panels 1, 2, 3, and 5, except on panels 4 and 6 the ST6236 strap was installed on each end of the shearwalls. In addition, on panel 6, the sill plate was blocked and the anchor bolts were connected through the double sill plate with 2″ x 2″ x 3/16″ square washers on top of the blocking. Blocking the sill plate allowed two rows of staples to be installed on the base of the panel in addition to the two rows up on the sides of the panel. Like panel 4, panel 6 had only one row of staples connecting the sheathing to the top plate. In summary, panel 4 added the strap to the panel 4 element, and panel 6 added the strap plus an additional row of staples to the sill plate.

The next set of 4′ x 8′ test elements were test panels 7, 8, and 9. These panels were constructed with one more component than the panel 6 design. Panels 7, 8, and 9 had a double sill and double top plate allowing for two rows of staples around the entire assembly; 2″ x 2″ x 3/16″ square washers clamped the double sill plate to the all-thread anchor bolts.

The final set of test elements consisted of two 8′ x 8′ panels. These panels were constructed from previously constructed 4′ x 8′ panels. The sheathing on these panels spanned vertically, and the two panels were placed standing side by side and bolted together with four ½″ diameter all-thread to try and connect the panels to create one 8′ long panel.

4.3 Code Published Wall Capacities

Design loads for the panels were determined in accordance with NER 272. In accordance with this evaluation report, the allowable load for 7/16″ OSB fastened to 2″ x 4″ SPF studs at 16″ on-center with 16 gauge x 2″ staples spaced at 2 ½″ on-center is 314 plf. This value was determined from Table 19 of NER 272, and adjusted for stud spacing and stud specific gravity. The calculations are: (1) per Footnote 11, 15/32 sheathing values (Table 20) may be used for 7/16″ thick sheathed panels when studs are spaced at 16″ on-center; (2) at a staple spacing of 2 ½″ on-center, the values from Table 21 are interpolated to 383 plf; and (3) this value is next reduced for the specific gravity of the wood per Footnote 2. The specific gravity of SPF studs is 0.42, and the reduction factor is 0.82. Multiplying the value of 383 plf by 0.82 gives a final design value of 314 plf. The hold-down design loads are based on the published loads in Simpson’s catalog. The allowable load is 3,285 pounds at a deflection of 0.031″. The uplift load on the hold-down at the panel’s design load, *314 plf multiplied by 4′ multiplied by 2* (h/w ratio) is 2,510 pounds (the ultimate load for the hold-down is 10.3 kips.) Therefore, the hold-downs should not be the limiting component for the first test elements. The end-wall studs should also be checked for design capacity. The bearing capacity of the sill plate is defined as the coefficient for duration of load (C_D) x the net area of studs (A) x allowable bearing of the sill plate. These calculations equate to *1.33 multiplied by 2 multiplied by 1.5 multiplied by 3.5 multiplied by 335 psi = 4.68 kips*. Similarly, the tension capacity of the end-wall studs is 3.73 kips; both loads are increased by one-third for duration of load. All of the components assembled have allowable loads within 25% of each other.

Two methods are used to determine the SLS capacity of the wood-shear walls. The strength of the element is a factor of all of its components. There can be wide variations in the assemblies; therefore the values determined must account for aspect ratio, hold-down efficiency, and dead-load bearing conditions on the top plate. One method presented in FEMA 274 shows that the ultimate cyclic load can be taken at 80% of the ultimate load determined in ATSM 72 static tests. This calculation is only valid for shear walls with aspect ratios of 1 or less, which agrees with the concerns of Nelson (2001).

Another method presented by FEMA 273 to determine the SLS capacity values is to multiply the allowable fastener design load by 2.8. Next, apply the appropriate load factors, except duration of load, which is included in the 2.8 factor. The SLS capacity is then determined by multiplying the strength design load by 1.25 (the reciprocal of 0.8). The values for determining the allowable loads and ultimate loads were determined from the in-grade testing program of AF and PA (FEMA 274 1997, pp. 8-6). The SLS capacity calculated for a 4′ x 8′ panel is 3.11 kips, and the YLS capacity (equivalent to the strength design load) is 2.49 kips. These values were determined using a 2 ½″ staple spacing along the base and top of the walls. The formula used was *52 pounds* (the allowable load for staples from NER 272 p. 7) *multiplied by 0.82* (wood-grade reduction) *multiplied by 2.8 multiplied by 20* (the number of nails along the base of the panel) *multiplied by 1.1* (increase for diaphragms).

The hold-down has a published ultimate load of 10.3 kips. At the ultimate load calculated above, the uplift force would be 6.22 kips, (*3.11 kips multiplied by 8′ divided by 4′*). This capacity is well below the hold-down's ultimate load, indicating that the hold-downs should perform well in the tests.

The displacement of the shear wall is calculated with the formula: $\Delta = 8vh^3/Eab + vh/Gt + .75he_n + h/b \times d_a$; The first term, $8vh^3/Eab$, is the deflection of a cantilever beam with a point load on the end; *h* is the wall height, v is the hear load in plf at the top of the wall, *A* is the area of the boundary elements (for wood-shear walls typically two studs), *b* is the wall length, and *E* is the modulus of elasticity of the wood. The second term, vh/Gt, is the load deformation for a shear panel. The two new variables not in the first equation, *G* and *t*, are the shear modules and effective thickness for shear. The second term is directly proportional to the height of the wall, whereas the first equation is proportional to the cube of the height of the wall. The third term, $.75\ he_n$, accounts for nail slip that is based on the load plf (nail spacing), and is proportional to the wall height. The last term, *h/w multiplied by* d_a, is hold-down slip proportional to the h/w ratio.

The terms of the deflection equation demonstrate how tall, narrow shear walls will deflect more with the same load per unit length than walls with a lower h/w ratio. Figure 4.4 shows how the *h/b* ratio affects each term in the deflection equation. The increase in flexibility of narrow shear walls exemplifies a need to stiffen these walls so they may resist their design loads at a lower deflection. The two most dramatic ways to accomplish the increase of the stiffness are to reduce the hold-down deflection, and to shorten the effective height of the wall.

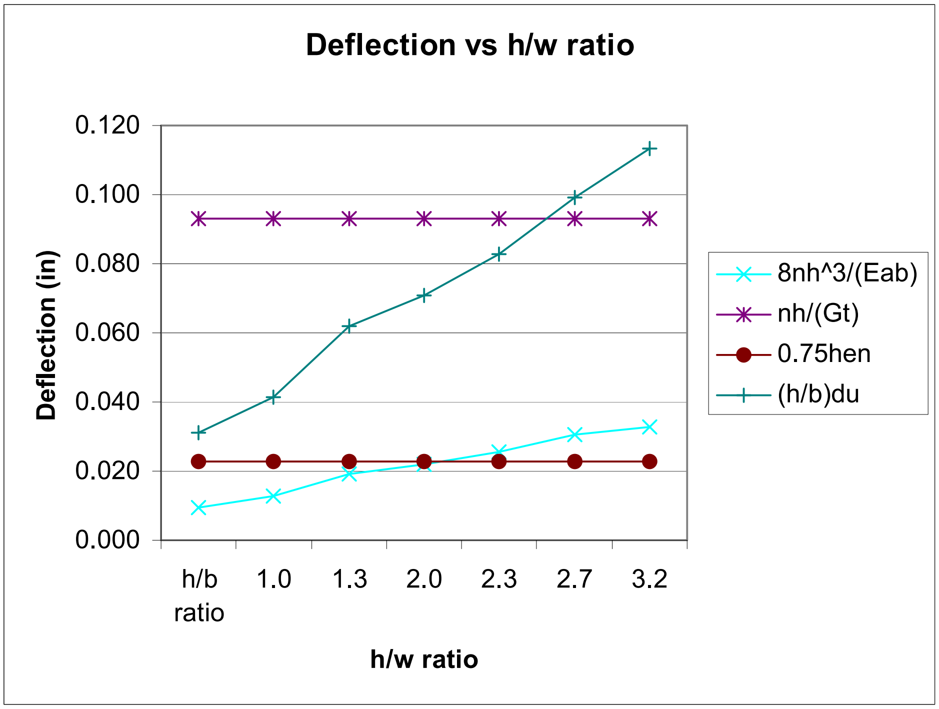


Figure 4.4 Shear wall deflection versus h/w ratio.

From the equation $\Delta = 8vh^3/Eab + vh/Gt + .75he_n + h/b \; x \; d_a$, the calculated deflection of a 4′ wide x 8′ tall shear wall with 7/16″ thick sheathing over 2″ x 4″ studs fastened with 16 gauge x 2″ long staples, and Simpson PHD2 hold-downs is 0.197″ at the design load of 314 plf. Table 4.1 demonstrates the calculations.

4.4 Free-Body Diagram of Hold-Down Configuration

In Figure 4.5 the diagrams show the free body diagrams of the eccentric hold-down, sheathing and hold-down strap. The arrows pointing up represent the uplift force from the sheathing on the end-wall studs. The arrows pointing down represent the

Table 4.1 Shear Wall Deflection Calculations

Shear wall deflection; Per UBC Standard 23-222

A =	10.5	area of chord cross-section (square inches)
b =	4.0	wall width (feet)
E =	1,600,000	elastic modules of boundary element (psi)
en =	0.0038	nail deformation (inch) table C NER 272 appendix
G =	90,000	modulus of rigidity of plywood in p from table 23-2-J
h =	8.0	wall height (feet)
t =	0.300	effective thickness of plywood, in shear, (inches) see table 23-2 H anc
v =	313.7	maximum shear due to design loads (plf)
Load per nail =	65.3	
nail spacing =	2.50	IN O/C.
delta 1	0.0191	in
delta 2	0.0232	in
delta 3	0.0228	in
delta 4	0.0660	in

h/b ratio	h	b	8vh^3/(Eab)	vh/(Gt)	0.75hen	(h/b)du	total
1.0	8	8	0.010	0.093	0.023	0.031	0.156
1.3	8	6	0.013	0.093	0.023	0.041	0.170
2.0	8	4	0.019	0.093	0.023	0.062	0.197
2.3	8	3.5	0.022	0.093	0.023	0.071	0.208
2.7	8	3	0.025	0.093	0.023	0.083	0.224
3.2	8	2.5	0.031	0.093	0.023	0.099	0.246
3.4	8	2.33	0.033	0.093	0.023	0.113	0.262

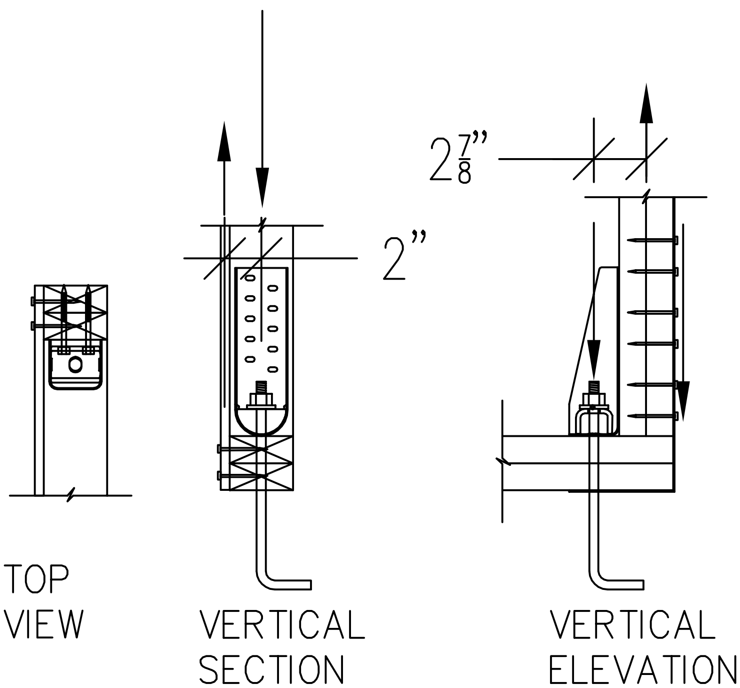


Figure 4.5. Free-body diagram of an eccentric hold-down. The arrows represent forces.

anchor-bolt force holding the wall down. The free-body diagram also shows the eccentricities. From the vertical section, there is a 2″ eccentricity across the sill plate. This eccentricity causes the sill plate fasteners to act in cross grain tension on the sides of the sill plates, which is where many sill plates fail.

From the vertical elevation, there is an almost 3″ eccentricity (assuming two rows of fasteners distributed evenly between each end-wall stud) between the anchor bolt and the uplift along the centerline of the fasteners on the end-wall studs. The modifications made to the hold-down configurations directly address both eccentricities. First, the double sill plate with the 2″ square washers clamps the sill plate to stiffen it against cross-grain bending from the sheathing fasteners. Second, the metal strap up the side of the end-wall studs keep the studs from being pulled off the sill plate as the wall rotates around the anchor bolt.

The strap causes the wall's sill plate to bend from the end of the wall to the anchor bolt. With a double sill plate, the resistance to the bending is much greater. From this review, it is clear that a single-sill plate, without a strap on the end wall, will have stress concentrations on the staples along the corners of the sill plate in addition to cross-grain bending in the sill-plate component. The following testing quantified the changes in shear-wall performance, when the modifications to the shear wall were made.