



U.S. Department of Housing and Urban Development
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The Performance
of Perforated Shear Walls with
Narrow Wall Segments,
Reduced Base Restraint, and
Alternative Framing Methods

The Performance of Perforated Shear Walls with Narrow Wall Segments, Reduced Base Restraint, and Alternative Framing Methods

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The Performance of Perforated Shear Walls with Narrow Wall Segments, Reduced Base Restraint, and Alternative Framing Methods

Introduction

Between one and two million new homes are built in the United States each year, predominantly with wood framing. For this reason, efficient utilization of our lumber supply is important. Ideally, the residential construction industry wants to build stronger, safer buildings that can withstand hurricane and earthquake loads while at the same time using material and labor resources more efficiently. In order to accomplish this goal, the actual load capacity and ductility of structures must be better understood from an engineering standpoint.

Shear walls are a primary lateral load resisting assembly in conventionally wood-framed construction. Traditional shear wall design requires fully sheathed wall sections restrained against overturning. Design of exterior shear walls containing openings, for windows and doors, involves the use of multiple shear wall segments and is required to be fully sheathed and have overturning restraint supplied by mechanical anchors. The design capacity of shear walls is assumed to be equal to the sum of the capacities for each full height shear wall segment. Sheathing above and below openings is typically not considered to contribute to the overall performance of the wall.

The traditional method of design described above is significantly different than wall bracing methods used historically in conventional construction. It is also more expensive than conventional construction while providing greater strength. However, there are significant opportunities to optimize this design process so that both safety and economy are achieved through more accurate design approaches. This report is a continuation of an effort to develop, confirm, and enhance such an approach. The ultimate goal is to provide both safety and economy to housing construction in all wind and seismic areas in the United States.

An alternate empirical-based approach to the design of shear walls with openings is the perforated shear wall method which appears in the *Standard Building Code* [1] and the *Wood Frame Construction Manual for One and Two Family Dwellings* [2]. The perforated shear wall method consists of a series of simple empirical equations used for the design of shear walls containing openings. When designing for a given load, shear walls resulting from this method will generally have a reduced number of overturning restraints than a similar shear wall constructed with multiple traditional shear wall segments. The inferred performance will be achieved due to the accuracy of the method. Only when strength demands exceed the capabilities of the perforated shear wall method will the more traditional engineering approach be more cost effective.

A significant number of monotonic and cyclic tests have provided verification of the perforated shear wall method [3] [4] [5] [6] [7] [8]. These studies include one-third scale model tests and full scale monotonic tests with 4 ft.(1.22 m) wall segments [3][4][5][6][7]. Another series of tests investigated the use of corners as end restraints instead of mechanical hold-down devices [8]. The following study provides additional information about the performance of full scale tests with 2 ft. (0.61 m) wall segments, reduced base restraint, and the use of alternative framing practices.

The Performance of Perforated Shear Walls with Narrow Wall Segments, Reduced Base Restraint, and Alternative Framing Methods

Background

Sugiyama [3] [4]

Yasumura and Sugiyama conducted tests studying one-third scale monotonic racking tests of wood stud, plywood sheathed shear walls with openings. The loads required to displace the wall at a shear deformation angle of 1/60, 1/75, 1/100, 1/150, and 1/300 were collected. The shear deformation angle is defined as displacement of the top of the wall minus the bottom of the wall divided by the total height.

Sugiyama defined r , the sheathing area ratio, in order to classify walls based on the amount of openings a wall contains. This value is determined by the ratio of the area of openings to the area of the wall with full height sheathing to the total length of the wall. The sheathing area ratio, r , is defined as

$$r = \frac{I}{I + \frac{A_o}{H \sum L_i}} \quad \text{(Equation 1)}$$

Where:

A_{go} = total area of openings

H = height of the wall

L_i = length of the full height wall segment

Sugiyama and Matsumoto determined an empirical equation to relate shear capacity and sheathing area ratio, based on the scaled tests. According to Sugiyama and Matsumoto the following empirical equation is applicable for the apparent shear deformation angle of 1/100 radians and for ultimate capacity:

$$F = r / (3 - 2r) \quad \text{(Equation 2)}$$

This equation relates the ratio, F , of the shear load for a wall with openings to the shear load of a fully sheathed wall at a particular shear deformation angle.

Dolan and Johnson [5]

The objective of the research conducted by Dolan and Johnson was two-fold. The first objective was to verify the work of Sugiyama using full scale tests, and the second objective was to determine a relationship between the ultimate capacities of a shear wall when tested monotonically versus the ultimate capacity of that same shear wall tested under cyclic loading.

Ten 40 ft. by 8 ft. (12.2 m by 2.4 m) walls were tested; all using identical framing, sheathing, nails, and nailing patterns. Five different sheathing area ratios were used, with each wall type tested once monotonically and once cyclically. The wall framing consisted of No. 2 spruce-pine-fir studs spaced 16 in. (406.4 mm) on center. Exterior sheathing was 15/32 in. (11.9 mm) plywood and the interior sheathing was 1/2 in. (12.7 mm) gypsum wallboard (GWB). Two hold-down anchors, one located at

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each end of the 40 ft. (12.2 m) wall specimen, were installed to provide the end restraint required by the perforated shear wall method. The tests were performed with the specimens in a horizontal position.

The predicted load capacities calculated from Sugiyama's empirical relationship were very close to the actual values measured (conservative by approximately 10 percent). All drywall tape joints around openings cracked and some tape joints between fully sheathed panels failed. Drywall nails near the corners began to fail. Bending of plywood and framing nails was observed near peak loads. Most nails tore through the edges of the plywood after peak capacity was reached. Hold-down anchors experienced no failure during the tests. In summary, the full scale tests confirmed the previous model scale tests by Sugiyama, and thus validated the perforated shear wall method.

Dolan and Heine [6]

There were two objectives of this research. The first was to quantify the effects of overturning restraint on full scale wood frame shear walls, with and without openings, tested monotonically. The second objective was to determine the applicability of the perforated shear wall method to conventionally built walls without considering overturning restraint provided by the hold-down devices.

Six 40 ft. by 8 ft. (12.2 m by 2.4 m) walls were tested; all using identical framing, sheathing, nails, and nailing patterns. Three different sheathing area ratios were used, with each wall type tested monotonically, once with no hold-down devices and once with the maximum number of hold-downs (one at the each end of the specimen and on both sides of all openings). The wall framing consisted of No. 2 spruce-pine-fir studs spaced 16 in. (406.4 mm) on center. Exterior sheathing was 7/16 in. (11.1 mm) oriented strand board (OSB) and the interior sheathing was 1/2 in. (12.7 mm) GWB. The tests were performed with the specimens in a horizontal position. Together with the results from Dolan and Johnson [5] three different wall configurations with three different end constraints were tested.

Shear walls designed according to the perforated shear wall method as opposed to traditional design had a lower ultimate capacity and stiffness. Ultimate capacity and stiffness were further reduced when hold-down anchors were omitted. A shear wall containing the maximum number of hold-down devices utilizes the overall material strength most efficiently, but the performance improvements may not be justified depending on the construction costs and design criteria. The data suggest that the perforated shear wall approach gives conservative design values for all of the tested wall configurations and conditions of restraint.

Dolan and Heine [7]

The objective of this research was to quantify the effects of overturning restraint on full scale Wood frame shear walls, with and without openings, tested cyclically. The test procedure Used in this study was a procedure proposed by the Structural Engineers Association of Southern California. It is a quasi-static test method which incorporates procedures included in the Sequential Phased Displacement (SPD) procedure proposed by M. L. Porter [9].

Six 40 ft. by 8 ft. (12.2 m by 2.4 m) walls were tested; all using identical framing, sheathing, nails, and nailing patterns. Three different sheathing area ratios were used, with each wall type tested cyclically, once with no hold-down devices and once with the maximum number of hold-downs (one at the each

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end of the specimen and on both sides of all openings). The wall framing consisted of No. 2 spruce-pine-fir studs spaced 16 in. (406.4 mm) on center. Exterior sheathing was 7/16 in. (11.1 mm) OSB and the interior sheathing was 1/2 in. (12.7 mm) GWB. The tests were performed with the specimens in a horizontal position. Together with the results from Dolan and Johnson [5] three different wall configurations with three different end constraints were examined.

The ultimate capacity of the walls subjected to SPD loading increased with increasing overturning restraint. The ultimate capacity reached during the SPD tests was 13 percent to 24 percent lower for walls with maximum overturning restraints when compared to monotonic test results. However, for the specimens with no hold-downs, ultimate capacities obtained from monotonic testing were 5 percent to 12 percent lower than capacities recorded during SPD testing.

Dolan and Heine [8]

The objective of this experiment was to quantify the effects of corners on uplift restraint of wood frame shear walls tested cyclically. The test procedure used in this study is a procedure proposed by the Structural Engineers Association of Southern California. The quasi-static test method incorporates procedures included in the Sequential Phased Displacement Procedure (SPD) proposed by M. L. Porter [9].

Four walls, 12 ft. (3.66 m) in length and 8 ft. (2.44 m) in height were tested using a SPD pattern. Attached to the ends were 4 ft. by 8 ft. (1.22 m by 2.44 m) and 2 ft. by 8 ft. (0.61 m by 2.44 m) corner segments. Each wall configuration was tested twice. Each specimen was constructed using identical framing, sheathing, nails, and nailing patterns. The wall framing consisted of No. 2 spruce-pine-fir studs spaced 16 in. (406.4 mm) on center. Exterior sheathing was 7/16 in. (11.1 mm) OSB and the interior sheathing was 1/2 in. (12.7 mm) GWB. The tests were performed with the specimens in a horizontal position.

Corner framing generally provided a hold-down effect that increased wall capacity when compared with straight walls with no overturning restraint and no perpendicular walls attached. On average, walls with 4 ft. (1.22 m) corner returns reached higher ultimate capacities than specimens with 2 ft. (0.61 m) corner returns. The hold-down effect provided by the corner framing was sufficient to provide for development of the unit shear capacity slightly lower than, but comparable to, straight walls with hold-downs. The 2 ft. (0.61m) and 4 ft. (1.22 m) corner returns provided sufficient end restraint to allow 85 percent and 90 percent, respectively, of the fully-restrained wall's tested unit shear to be realized.

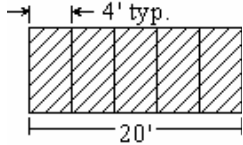
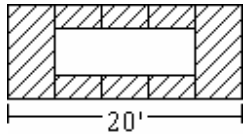
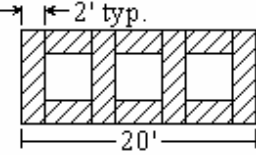
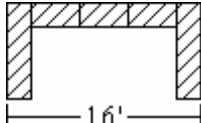
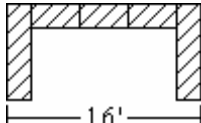
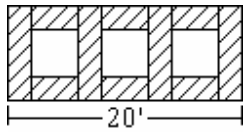
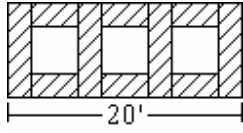
Experimental Program

Wall Specimens

A total of 7 shear wall specimens were tested in this investigation (Table 1). Wall 1, Wall 2, Wall 3, and Wall 4 followed the perforated shear wall approach. Two hold-down anchors were used on each of these specimens - one at each end. In addition to the hold-downs, 5/8 in. (15.9 mm) diameter bolts were used to anchor the bottom plate of the specimen at 2 ft. (0.61 m) on center.

Table 1

The Performance of Perforated Shear Walls with Narrow Wall Segments, Reduced Base Restraint, and Alternative Framing Methods

Shear Wall Configurations					
Specimen	Wall Configuration	Openings	Sheathing Area Ratio, r^1	Anchor Bolt Spacing	Hold-downs
Wall 1		None	1.0	2' o.c.	Ends
Wall 2		(1) - 12' x 4'	0.57	2' o.c.	Ends
Wall 3		(3) - 4' x 4'	0.57	2' o.c.	Ends
Wall 4		(1) - 12' x 6'-8"	0.29	2' o.c.	Ends
Wall 5 ²		(1) - 12' x 6'-8"	0.29	2' o.c.	Ends
Wall 6 ²		(3) - 4' x 4'	0.57	2' o.c.	None
Wall 7		(3) - 4' x 4'	0.57	6' o.c.	Ends

For SI: 1 ft. = 0.3048 m, 1 in. = 25.4 mm.

¹ Calculated from Equation 1.

² Alternative framing methods used. See section 3.1 Wall Specimens for framing details.

Wall 1 was fully sheathed and served as the control from which shear ratios were derived for walls with openings. Also, 1-5/8 in. (41.3 mm) diameter flat washers were used with the 5/8 in. (15.9

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mm) diameter anchor bolts throughout the testing program. Previous tests utilized a 3 x 3 x 1/4 in. (76.2 x 76.2 x 6.4 mm) steel plate washer on each anchor bolt [5][6][7][8].

Two specimens (Wall 5 and Wall 6) in this research program were constructed using alternative framing practices. Wall 5 was constructed using one 1,000 pound capacity strap on each end of the specimen. The continuous strap was connected to the exterior sheathing side of the jack stud assembly using 9-10d common nails, wrapped over the header/double top plate, and connected to the interior sheathing side of the jack stud assembly using 9-10d common nails. The header spanned the total length of the specimen less the thickness of the end studs. The end stud was face nailed to the end grain of the header with four 16d common nails. One king stud and one jack stud were used at each end of the specimen. Two additional jack studs were located 24 in. from the end of the specimen. A photograph of the detail is given in Figure 1. Wall 6 was constructed using truss plates to reinforce framing joints at wall corners and opening corners (Figure 2). The truss plates were installed on each side of the framing using a mallet. The truss plates were 3 in. by 6 in. (76.2 mm by 152.4 mm) with 144 teeth per truss plate. No hold-downs were installed on this specimen (Figure 3).

Wall 7 was constructed with hold-downs at the ends and anchor bolts spaced 6 ft. (1.83 m) on center. This anchor bolt spacing represents typical practice in residential construction [10]. Again, 1-5/8 in. (41.3 mm) diameter flat washers were used with the 5/8 in. (15.9 mm) diameter anchor bolts. The bolts were located in the center of each 2 ft. (0.61 m) full height wall segment.



Figure 1
Alternative Framing Practices of Wall 5

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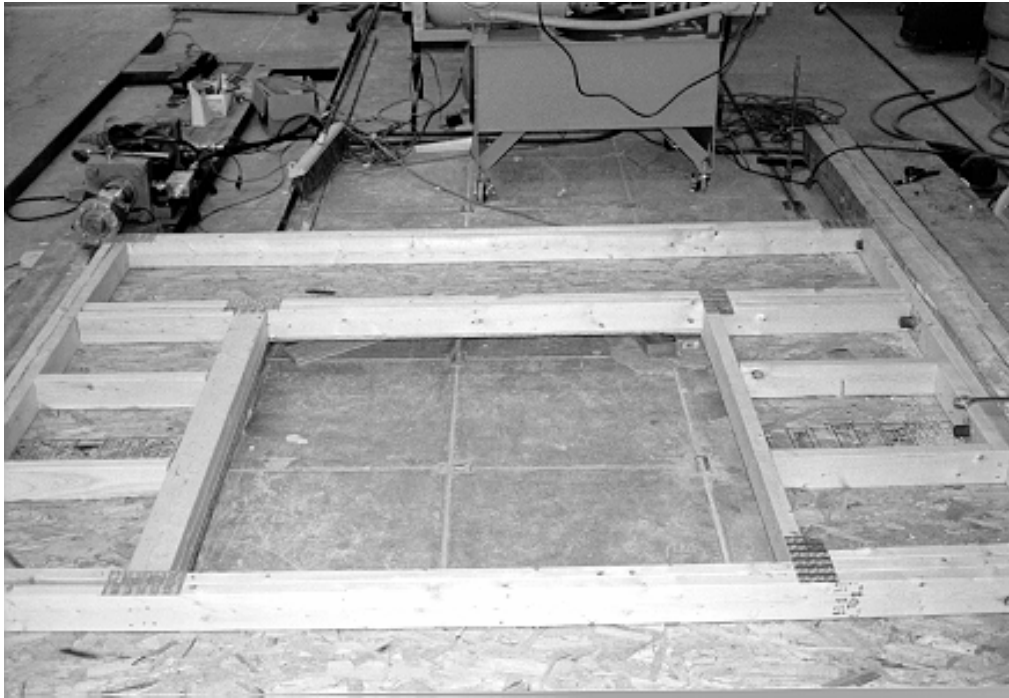


Figure 2
Truss Plates Reinforced Wall Corners and Opening Corners (Wall 6)

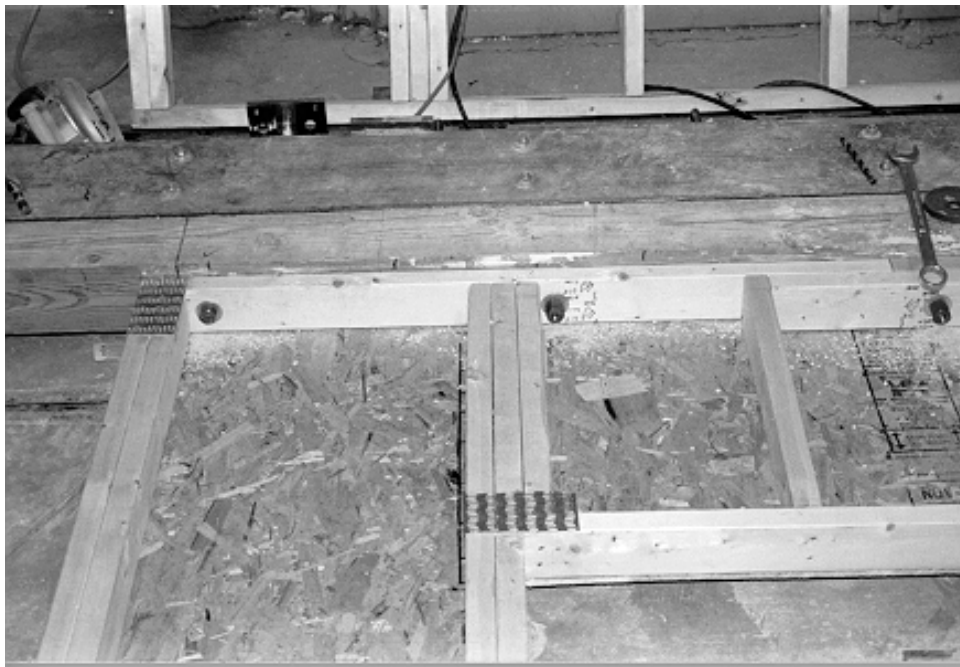


Figure 3
Close Up of Corner Framing and Anchor Bolt (Wall 6)

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All specimens were constructed with spruce-pine-fir, stud-grade lumber. Studs were spaced 16 in. (406.4 mm) on center for Wall 1 and Wall 2. Stud spacing was increased to 24 in. (609.6 mm) on center for the remaining specimens to investigate the effects of 2 ft. (0.61 m) wall segments. Headers and window sills were constructed to span openings, and king and jack studs were also used on either side of openings. Exterior sheathing consisted of 7/16 in. (11.1 mm) OSB, oriented vertically. The OSB was attached using 8d common nails spaced 6 in. (152.4 mm) along the perimeter and 12 in. (304.8 mm) in the field of the panels. Interior sheathing consisted of 1/2 in. (12.7 mm) GWB, oriented vertically. The GWB was attached with #6 screws spaced 7 in. (177.8 mm) along the perimeter and 10 in. (254.0 mm) in the field. Both interior and exterior sheathing were cut to fit above and below the doors and windows. A summary of the wall materials and construction data can be found in Table 2 and Table 3.

Table 2
Wall Materials and Construction Data

Component	Construction and Materials
Framing Members	Stud, Spruce-Pine-Fir, 2x4.
Sheathing	
Exterior	7/16 in. OSB, 8d common nails with 6 in. spacing on panel edges and 12 in. spacing along panel field (sheets installed vertically).
Interior	1/2 in. Gypsum Wallboard, #6 screws with 7 in. spacing on panel edge and 10 in. spacing along panel field (installed vertically, joints taped).
Headers	
4'-0" opening	2-2x4 with. One jack and one king stud at each end.
12'-0" opening	2-2x12 with an intermediate layer of 7/16 in. OSB. Two jack studs and one king stud at each end. ¹
Structural Base Connections (Bottom of Wall)	
Hold-down	Simpson HTT 22, nailed to end studs with 32-16d sinker nails, 5/8 in. diameter tie rod to connect to reaction beam.
Anchor Bolts	5/8 in. diameter tie rods with 1-5/8 in. diameter flat washer.
Loading Tube Connections (Top of Wall)	
No Openings	1/2 in. diameter bolts with 1-5/8 in. flat washer @ 2 ft. on center.

For SI: 1 ft. = 0.3048 m, 1 in. = 25.4 mm.

¹The header in Wall 5 spanned the entire wall less the thickness of the end studs. One king stud and one jack stud were used at each end of the specimen. The king stud was face nailed into the end grain of the header using four 16d nails. Two additional jack studs were located 24 in. from the end of the specimen (Figure 1). The sheathing panels were fastened with no alteration of the fastening schedule. In Wall 4, the header ended at the edge of the opening.

Table 3

The Performance of Perforated Shear Walls with Narrow Wall Segments, Reduced Base Restraint, and Alternative Framing Methods

Fastening Schedule

Connection Description	Type of Connector	Spacing
Framing:		
Top Plate to Top Plate (face-nailed)	16d common	12 in. on center
Top/Bottom Plate to Stud (end-nailed)	2-16d common	per connection
Stud to Stud (face-nailed)	2-16d common	24 in. on center
Stud to Header (toe-nailed)	2-16d common	per stud
Stud to Sill (end-nailed)	2-16d common	per stud
Header to Header (face nailed)	2-16d common	16in. on center
Hold-down (face nailed)	32-16d sinker	per hold-down
Sheathing:		
OSB	8d	6 in. edge/12 in. field
GWB	#6 screws	7 in. edge/10 in. field

For SI: 1in. = 25.4 mm.

Test Procedures

The shear walls were tested in a horizontal position according to ASTM E564 [11]. A hydraulic actuator, with a range of 12 in. (304.8 mm) and capacity of 115,000 lb. (511.5 kN), applied the load to the top right corner of each shear wall through a 4x4 structural steel tube (Figure 4). A 1/2 in. (12.7 mm) thick steel plate was welded to the end of the tube to provide a uniform loading area for the actuator. A 50,000 lb. (222.4 kN) capacity load cell was attached to the end of the actuator to enable load recordings. The load cell was calibrated immediately prior to the tests using the NAHB Research Center's Universal Test Machine. Casters, which were attached to the tubing, and roller-plate assemblies were used to allow horizontal motion. The casters and the roller-plate assemblies were positioned parallel to the direction of loading (Figure 5).

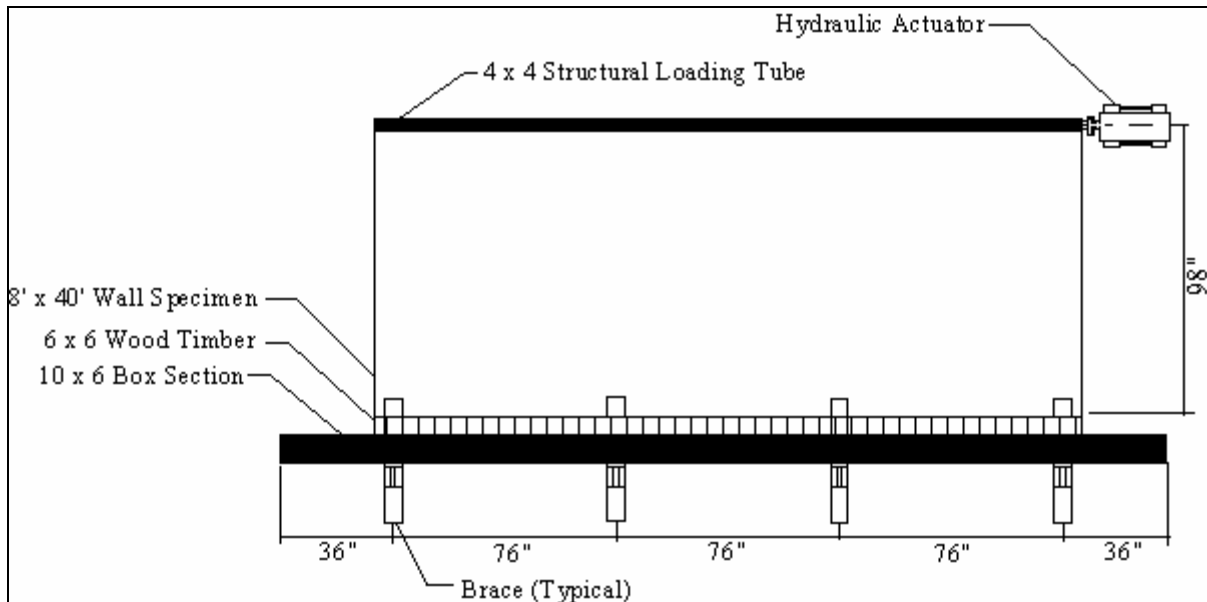


Figure 4
Plan View of Shear Wall Testing Layout

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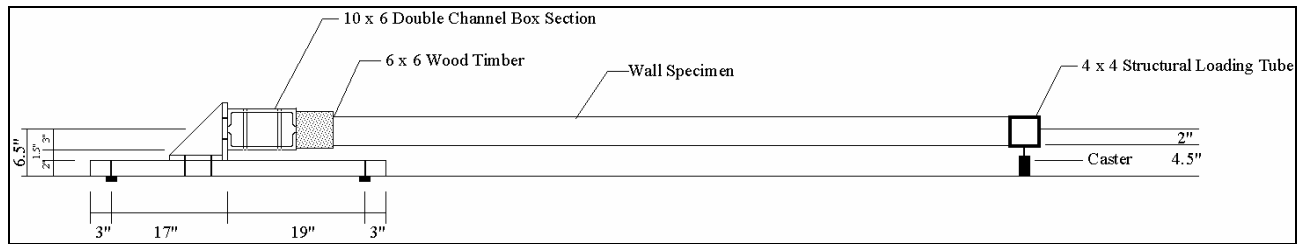


Figure 5
Side View of Shear Wall Testing Layout

Three linear variable differential transformers (LVDT) were used to measure the displacement of the specimens during the test. The LVDTs measured the horizontal displacement of the top of the wall, the horizontal displacement, or slip, of the bottom sill plate of the specimen, and the uplift of the end studs relative to the foundation.

All tests were one directional, displacing the top of the wall to a maximum of six inches over a twenty minute period. Data from the load cell and 3 LVDTs were collected 2 times per second. Each of the seven wall configurations was tested once. Items of interest are ultimate load capacity, load distribution, initial stiffness, energy dissipated, slip of the sill plate, and uplift. Load-displacement curves were plotted for each of the wall specimens to better understand and compare the behavior of the walls during the test.

It should be noted that a spacer between the wall and 6x6 wood timber was inadvertently omitted from the first few tests. The spacer's purpose is to provide room for unobstructed rotation of the sheathing panels. The spacer was not applied to the remaining tests for consistency. Therefore, it is likely that the tests were affected to some small degree by sheathing panel interference with the 6x6 timber. However, the results from the tests in this report compare well with previous tests that include the spacers [6].

Results and Discussion

Force-Displacement Response

The response of the shear wall specimens to the loading history are shown in the force-displacement curves of Figures 6, 7, and 8. The initial response to load was linear. The ultimate load, F_{max} , as well as the corresponding displacement, $\Delta_{F_{max}}$, was gathered directly from the data. Resistance at failure was determined as the capacity of the specimen immediately prior to a significant decrease in strength or when the load dropped to $0.8F_{max}$, whichever occurred first. These loads and displacements are presented in Table 4.

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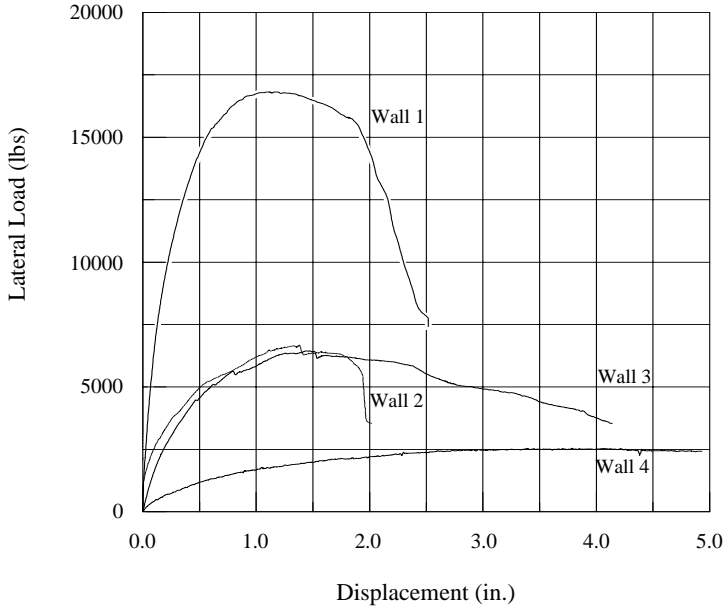


Figure 6
Force-Displacement Response for Walls 1, 2, 3, and 4

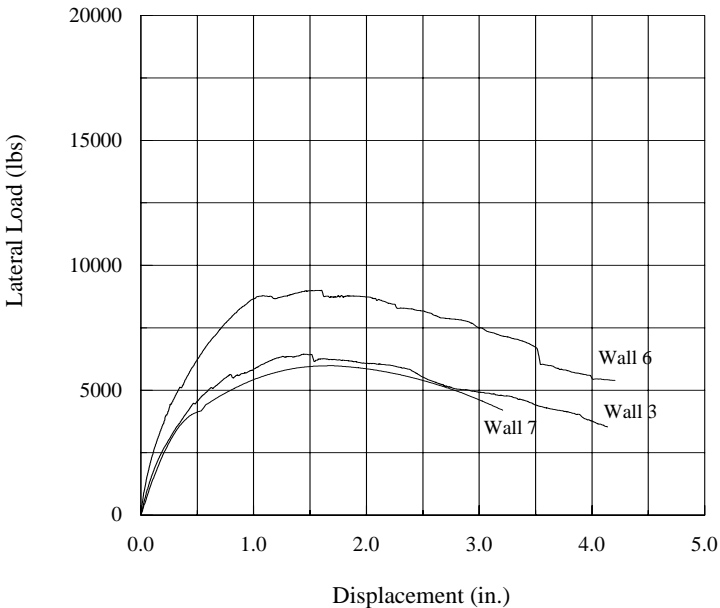


Figure 7
Force-Displacement Response for Walls 3, 6, and 7

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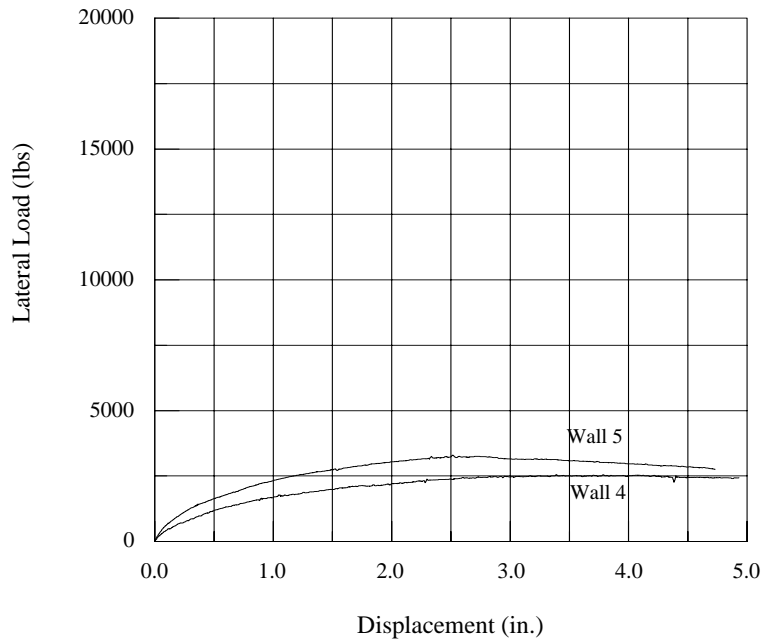


Figure 8
Force-Displacement Response for Walls 4 and 5

Table 4
Force-Displacement Data from Testing

	Wall Specimens						
	1	2	3	4	5	6	7
Sheathing Area Ratio	1.00	0.57	0.57	0.29	0.29	0.57	0.57
Calculated Shear Ratio ¹	1.00	0.31	0.31	0.12	0.12	0.31	0.31
Actual Shear Ratio	1.00	0.42	0.38	0.16	0.20	0.54	0.35
Actual/Predicted	1.00	1.36	1.25	1.32	1.66	1.76	1.16
F_{max} (kips)	16.9	7.0	6.5	2.7	3.4	9.1	6.0
Δ_{Fmax} (in.)	1.15	1.39	1.51	3.39	2.33	1.46	1.60
$F_{failure}$ (kips)	13.5	5.6	5.2	2.4	2.8	7.3	4.8
$\Delta_{failure}$ (in.)	2.06	1.92	2.65	4.93	4.73	3.14	2.91
Initial Stiffness (kips/in)	63.2	15.6	13.4	2.6	4.0	18.5	12.3
Energy Dissipated (kips*in)	15.0	5.2	7.1	5.1	6.3	11.9	6.8
Δ_{uplift} (in.)	0.11	0.18	0.16	0.39	0.18	0.37	0.18

¹The predicted shear ratio is based on the empirical formula, $F = r/(3-2r)$, developed by Sugiyama and Matsumoto for wood-framed shear walls.

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As expected, the specimens with the larger sheathing area ratio experienced larger ultimate loads. The 4 ft. (1.22 m) wall segments of Wall 2 resulted in slightly greater ultimate load than that of Wall 3 (narrow wall segments), but experienced failure at a much lower displacement than that of Wall 3. Both of the specimens using 4 ft. (1.22m) and 2 ft. (0.61 m) wall segments performed in conservative agreement with the perforated shear wall predictions as shown in Figure 9. The use of truss plates without hold-downs in Wall 6 increased the ultimate capacity of the specimen by 40 percent. Increasing the anchor bolt spacing to 6 ft. on center (Wall 7) resulted in a slightly lower ultimate load and an earlier failure than that of Wall 3. However, both performed in agreement with the current perforated shear wall prediction. The alternative framing practices in Wall 5 resulted in over a 25 percent increase in ultimate capacity compared to that of Wall 4.

Predicted shear load ratios, F , were determined using Equation 2 and are presented with the actual shear load ratios in Table 4. The ratio of actual to predicted is also presented in Table 4, where a ratio greater than 1.0 indicates a conservative prediction. Figure 9 plots actual capacities and shear load ratios found from the testing. As shown in Figure 9, Equation 2 conservatively estimates the capacity for all wall configurations in this investigation.

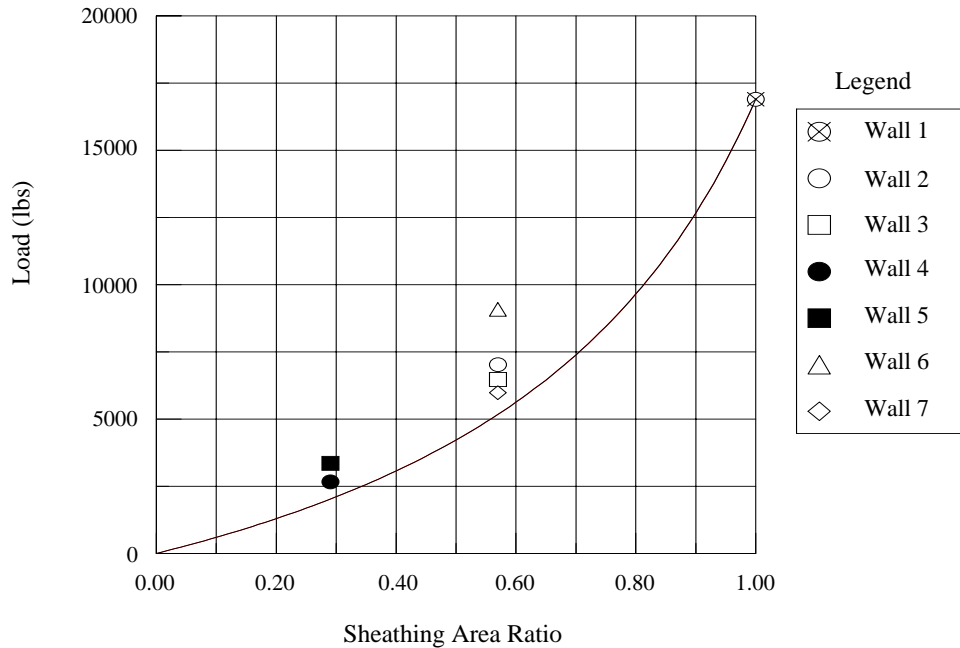


Figure 9
Ultimate Capacity vs. Sheathing Area Ratio

Due to the conservative predictions of Equation 2, the following equation was used as an alternative to predict the shear load ratios [3] [4]:

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$$F = r / (2-r)$$

Equation 3

The additional predicted shear load ratios, F , were determined using Equation 3 and are presented with the actual shear load ratios in Table 5. The ratio of actual to predicted is also presented in Table 5, where a ratio equal to 1.0 is an exact prediction. Figure 10 plots actual capacities and shear load ratios found from the testing. As shown in Figure 10, Equation 3 estimates the capacity for Walls 1 through 6 more accurately than that of Equation 2. However, Equation 3 overpredicts the shear capacity of Wall 7 with a 6 ft. (1.83m) on center anchor bolt spacing.

Table 5
Predicted Shear Load Ratios, $F = r/(2-r)$

	Wall Specimens						
	1	2	3	4	5	6	7
Sheathing Area Ratio	1.00	0.57	0.57	0.29	0.29	0.57	0.57
Calculated Shear Ratio ¹	1.00	0.40	0.40	0.17	0.17	0.40	0.40
Actual Shear Ratio	1.00	0.42	0.38	0.16	0.20	0.54	0.35
Actual/Predicted	1.00	1.05	0.95	0.94	1.18	1.35	0.88

¹ The predicted shear ratio is based on the empirical formula, $F = r / (2 - r)$.

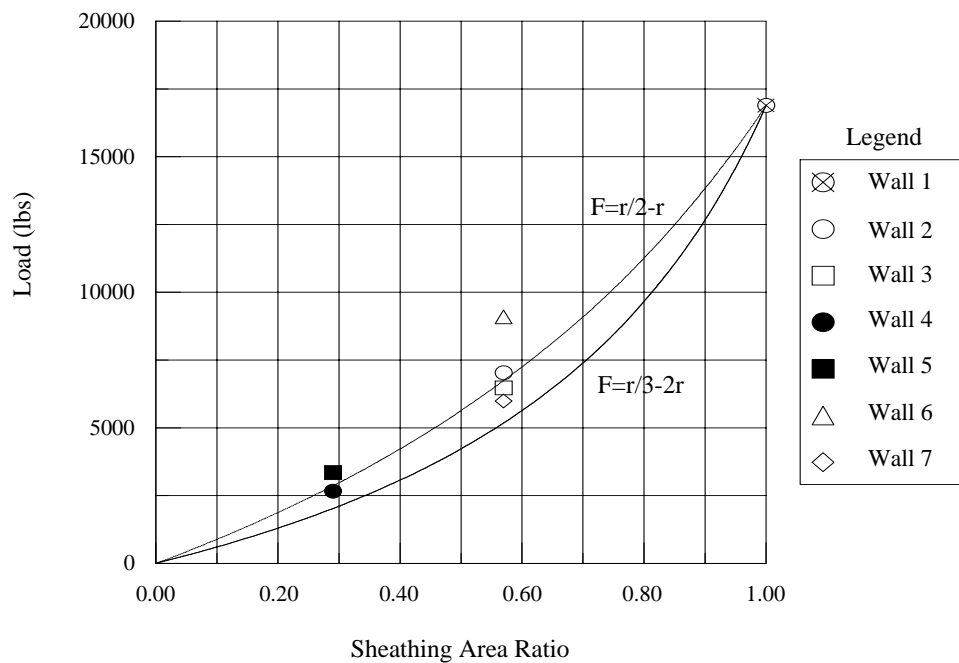


Figure 10
Ultimate Capacity vs. Sheathing Area Ratio

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Load Distribution

The unit shear for Wall 1 was,

$$\text{unit shear} = 16.9 \text{ kips} / 20 \text{ ft} = 845 \text{ lbs/ft}$$

which compares well with the previous tests mentioned in the Literature Review (865 lbs/ft) [6]?

Wall 2 recorded an ultimate load of 7.0 kips (31.1 kN). From the unit shear capacity of Wall 1, the leading restrained panel theoretically absorbed,

$$845 \text{ lbs/ft} \times 4 \text{ ft} = 3,380 \text{ lbs}$$

which is less than half of the recorded ultimate load. Therefore, the majority of the shear was not concentrated in the leading 4 ft. by 8 ft. (1.22 m by 2.44 m) panel. In fact, a significant portion of the load was shared with the rest of the wall because of the shear transfer through the sheathing above and below the openings.

In a similar comparison, Wall 3 recorded an ultimate load of 6.5 kips (28.9 kN). The leading restrained panel theoretically absorbed,

$$845 \text{ lbs/ft} \times 2 \text{ ft} = 1,690 \text{ lbs}$$

Again, the shear was not concentrated in the leading 2 ft. by 8 ft. (0.61 m by 2.44 m) panel which was restrained. In fact, a significant portion of the load was shared with the rest of the wall because of the shear transfer through the sheathing above and below the openings.

Initial Stiffness

The initial portion of the force-displacement curves were fitted with a linear least-squares trend, the slope of which is taken as the initial stiffness. That portion of the curve for which the magnitude of the force did not exceed 40 percent of the peak load was used in the calculation. Generally, the force-displacement data in this range demonstrates a strong linear relation. The initial stiffnesses are listed in Table 4.

In general, initial stiffness was proportional to the sheathing area ratio. Wall 2 experienced a larger initial stiffness than that of Wall 3. The truss plate reinforcement of Wall 6 increased the initial stiffness. Increasing the anchor bolt spacing to 6 ft. (1.83 m) on center had little effect on the specimen's stiffness.

Energy Dissipated

The toughness of a wall can be quantified by its ability to dissipate energy while deforming. Cumulative energy dissipation was obtained by calculating the area under each force-displacement curve up to $0.80F_{\max}$ using Simpson's Method. These values are listed in Table 4.

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Although the 4 ft. (1.22 m) wall segments in Wall 2 had a slightly greater ultimate capacity and initial stiffness than did Wall 3, the energy dissipated by the specimen was 35 percent less than that of Wall 3 (narrow wall segments). The addition of truss plates also provided a significant improvement in this area of performance. The increased anchor bolt spacing had little effect on the energy dissipation.

End Stud Uplift

Loading each wall resulted in uplift zones at the end of the walls. The vertical displacement of the end stud was measured by a LVDT. The uplift at F_{max} is given in Table 4.

The vertical displacement of the end stud remained fairly constant for Wall 2, Wall 3, and Wall 7. That is, the narrow wall segments and increased anchor bolt spacing had no effect on the uplift at the end stud. Wall 6 experienced greater uplift at ultimate load than did the former three specimens due to the lack of hold-down restraints. The alternative framing detail used on the garage opening in Wall 5 did help prevent uplift in comparison to the conventional framing of the garage opening in Wall 4.

Failure Modes

All walls tested had similar failure characteristics. The initial loading was highly linear until the interior sheathing, OSB, began to pull through the screws. This resulted in a slight reduction in stiffness. As the load approached ultimate capacity the OSB sheathing near the loaded end began to buckle and bending of OSB and framing nails was observed elsewhere. Racking of full height OSB panels was observed, while the OSB above and below openings acted as a rigid body. After ultimate capacity, the nails tore through the edges of the OSB. As failure progressed the nails failed along the bottom plate in the walls with openings. This failure was more prevalent in the wall section that had no hold-down anchor to resist overturning on the tension (uplift) side of the wall specimen (i.e. toward the loaded end).

Although the above failure mechanisms were consistent throughout the testing some differences were observed which explain the results discussed above. Comparing Wall 2 (4 ft. (1.22 m) wall segments) and Wall 3 (2 ft. (0.61m) wall segments), the slightly larger ultimate load and initial stiffness of Wall 2 can be attributed to the wider wall segments. However, the wider wall segments caused Wall 2 to fail at a much lower displacement than that of Wall 3. Once ultimate load was achieved, the large opening in Wall 2 was no longer square and the two end sheets of OSB had pulled through the nails causing a sudden decrease in load. However, the intermediate narrow wall segment in Wall 3 prevented the racking of the openings after ultimate load. This was evident from the tearing of the full height OSB along the top of the openings. The bottom of these intermediate sheets remained relatively stable while the rigid body motion of the sheathing above the openings caused tearing of the full height OSB at the top opening corners.

While the behavior of Wall 4 and Wall 5 was very similar the results were quite different. The alternative framing practices of Wall 5 provided a 26 percent increase in ultimate capacity, a 54 percent increase in initial stiffness, and a 24 percent increase in energy dissipated.

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The added truss plate connectors at the specimen corners and opening corners of Wall 6 improved the overall performance of Wall 3, even with hold-down restraints omitted from the ends of Wall 6. The lack of hold-down restraints in Wall 6 is only evident in the vertical stud displacement at the end of the wall. The truss plates attached to the corner of the specimen provided enough restraint to prevent separation of the bottom plate-double end stud connection. This along with the reinforced opening corners allowed for a greater ultimate load, initial stiffness, and energy dissipation. The bottom plate began to split only after the ultimate load was reached. This may have been prevented with the use of a larger plate washer at the end stud anchor bolt, in lieu of the 1-5/8 in. (41.3 mm) diameter flat washers.

Increasing the anchor bolt spacing to 6 ft. (1.83 m) on center in Wall 7 only resulted in a slight decrease in ultimate capacity, initial stiffness and energy dissipation compared to that of Wall 3. This preliminary test gives promise to the use of the perforated shear wall method with reduced base restraint. Future testing will be directed toward this issue.

Summary and Conclusions

The perforated shear wall method was first developed by conducting tests on one-third scale monotonic racking tests of shear walls. Equation 2 was developed to predict the shear load capacity for shear walls with openings [3][4]. The perforated shear wall method was confirmed (conservative by approximately 10%) using full scale tests of 40 ft. (12.19 m) long shear walls with openings constructed with 4 ft. (1.22 m) wall segments [5]. Additional testing was conducted to determine the effect of overturning restraints. Again, it was concluded that the perforated shear wall method results in conservative design values for shear walls [6]. The next phase of testing quantified the effects of corners on uplift restraint. The 2 ft. (0.61m) and 4 ft. (1.22 m) corner returns provided sufficient end restraint to allow 85 percent and 90 percent, respectively, of the fully-restrained wall's tested unit shear to be realized [8]. Each of the aforementioned phases of research refined the perforated shear wall method resulting in a more efficient and economical shear wall design. This report provides additional refinement to the perforated shear wall method.

The data presented provides verification of the perforated shear all method using 2 ft. (0.61 m) wall segments. Resistance to drift histories was similar to that of specimens with 4 ft. (1.22 m) wall segments. The calculated shear capacity using the empirical equation developed by Sugiyama and Matsumoto (Equation 2) conservatively estimates the capacity of all specimens tested. The use of the alternative empirical equation (Equation 3) results in a more accurate prediction of capacity on average. Also, increasing anchor bolt spacing slightly decreases the ultimate capacity, initial stiffness, and energy dissipated. Despite these decreases, the empirical equation (Equation 2) developed by Sugiyama and Matsumoto conservatively estimates the capacity with increased anchor bolt spacing. The alternative framing practices investigated in this report show promise for high-wind and high-seismic applications.

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Recommendations

Additional testing should be done to build on the findings of this study. Future topics of study should include:

- Verification of perforated shear wall approach with varying degrees of sill plate connection (i.e. nailed sill plate connection in lieu of anchor bolts);
- Determination of the effect of dead load on unrestrained, perforated shear walls;
- Continued investigation of the effect of alternative framing practices (the use of truss plate connectors and/or strapping) for use in high-wind and high-seismic conditions;
- Quantification of the effect of non-structural items present in finished walls (i.e. windows and doors); and,
- Investigation of the benefit of adhesives for sheathing attachment to better resist wind loads.

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