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DESIGN GUIDE 6

Monotonic Tests of Cold-Formed Steel Shear Walls with Openings

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MONOTONIC TESTS OF COLD-FORMED STEEL SHEAR WALLS WITH OPENINGS

Prepared for

The U.S. Department of Housing and Urban Development
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The American Iron and Steel Institute
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The National Association of Home Builders
Washington, DC

by

NAHB Research Center, Inc.
Upper Marlboro, MD

INTRODUCTION

This publication was developed by the National Association of Home Builders (NAHB) Research Center for the American Iron and Steel Institute, the US Department of Housing and Urban Development (HUD), and the NAHB. It is intended to provide more affordable design and construction techniques of residential buildings using cold-formed steel framing. AISI believes that the information contained in this publication substantially represents practice and related scientific and technical information, but the information is not intended to represent an official position of AISI or to restrict or exclude any other construction or design technique.

The following publication has been developed for the American Iron and Steel Institute (AISI) which is comprised of representatives of steel producers in North America. In the production of this publication, due diligence has been exercised in consulting a wide range of pertinent authorities and experiences and efforts have been made to present accurate, reliable and useful information. AISI and the NAHB Research Center, Inc., expresses great appreciation to the sponsors of this work in view of its relevance to more affordable design and construction of residential buildings using cold-formed steel framing. AISI recognizes the principal authors of this publication: Shawn McKee, Principal Investigator, Jay Crandell, P.E., Technical Reviewer, Nader Elhajj, P.E., Project Manager, Kevin Bielat, Technical Support of the NAHB Research Center.

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ABSTRACT

Light-gauge steel-framing has recently become recognized as a viable alternative framing method for residential construction in the U.S. With this recognition, the need to efficiently design for lateral loads produced by winds and seismic forces has prompted research to fill this need. This paper presents monotonic load tests of 40-foot long, cold-formed steel-framed shear walls with openings. Lateral force resistance and displacement response are discussed. Predicted capacities, using an empirical design approach known as “Perforated Shearwalls”, are compared with measured capacities. Also, ultimate capacity, initial stiffness, and energy dissipation are highlighted. Findings from these tests indicate that light-gauge steel-framed shear walls are a viable alternative for residential construction in high wind and seismic regions.

INTRODUCTION

For decades home builders in the United States have made wood their material of choice because of its satisfactory performance, abundant supply, and relatively low cost. However, increases and unpredictable fluctuations in the price of framing lumber, as well as problems with its quality, are causing builders to seek alternative framing methods. The use of light-gauge steel framing in the residential market has steadily increased in recent years.

Much has been done in the area of improving the design approach for wood shear walls. Based on the success of wood-framed wall testing, it is expected that a similar test program will optimize steel-framed shear wall design, particularly for use in areas with extreme wind and seismic conditions. The current *Prescriptive Method for Residential Cold-Formed Steel Framing*[1] considers only full-height sheathing as adding lateral strength to shear wall systems. Full height sheathing is defined as structural sheathing segments which are at least 48 inches in length and spans the full height of the wall. This test program will allow for greater design flexibility in designing wall systems with openings and more accurately account for their impact on the lateral strength of buildings.

A research program was initiated at the NAHB Research Center to study the performance of steel-framed shear walls and to improve the efficiency of engineering methods. An experimental study was designed to determine the capacity of full scale steel-framed shear walls as influenced by the presence of openings. A secondary goal of the program was to briefly investigate the effects of reduced anchoring constraints in view of future research to account for restraint provided by corners without including hold-down brackets.

LITERATURE REVIEW

<u>Tarpy and Girard</u> (1982)

[1] U.S. Department of Housing and Urban Development, *Prescriptive Method for Residential Cold-Formed Steel Framing* - First Edition prepared by the NAHB Research Center, Upper Marlboro, MD, 1996.

The paper by Tarpy and Girard presents results of an experimental test program for determining the shear resistance of steel-framed stud wall panels with different construction details and sheathing materials without the use of diagonal cross-bracing. The objective of the test program was: (1) to determine the effect of different construction techniques and anchorage details on the in-plane shear resistance of steel stud shear walls with different types of sheathing, and (2) to determine thresholds for damage of the walls due to lateral in-plane displacement. The study considered five main parameters:

- The effect of using light gauge clip angles and powder-actuated fasteners in place of bolts and washers to anchor the base of the wall panel.
- The effect of anchoring the wall panel through transverse floor joists.
- The effects of plywood or gypsum exterior sheathing in place of gypsum wallboard as a diaphragm material.
- The effect of using fillet welds instead of self drilling screws to attach the studs to runner racks.
- The effects of using 16-inch rather than a 24-inch stud spacing.

According to the authors, these conditions were considered to have significant influence on the wall performance based on previous research.

The individual panels were constructed using 3.5 in. web x 1.5 in. flange x 0.5 in. lip C-shaped studs with a base metal thickness of 0.0359 in. (nominal 20 gauge). The steel studs were attached to 3.625 in. web by 1.5 in. flange structural steel runner track with a base metal thickness of 0.0359 in. (nominal 20 gauge). The test procedure followed ASTM E 564-95 static test method for determining the shear resistance of framed walls. Only two of the walls were tested with a structural sheathing material.

All wall types tested experienced similar types of failure. The initial signs of distress were the runner tracks deforming around the anchorage device at the tension or uplift corner of the wall. Increased loads caused cracking of gypsum wallboard at the same locations from the corner fasteners to the edge of the wall board. The use of construction grade plywood resulted in a 25% increase in the ultimate capacity compared with the gypsum sheathing. Welding studs to the runner track was considered as effective as using self-drilling screws. Also decreasing the stud spacing from 24 inches on-center to 16 inches on center added only slightly to the ultimate load capacity.

The elimination of clip angles at interior locations had little effect on the shear strength or stiffness. Bolt and washer anchoring resulted in a 24% decrease in shear strength compared with powder actuated and corner angle anchorage. Anchoring specimens through floor joists had a detrimental effect on ultimate shear capacity due to the reduction in rigidity at the anchors.

<u>Dolan</u> (1989)

The primary objective of the research conducted by Dolan was to develop a numerical model that was capable of predicting the deflection at the top of timber shear walls when subjected to racking and dynamic earthquake loadings.

Connection tests were performed to determine the load-deflection curves for the connections between the timber components. These connection tests were conducted in order to determine the parameters defining the load-deflection curves for the sheathing and corner connection. Plywood and oriented strand board (OSB) were the structural sheathings used in the study. The results were as follows:

- There was no obvious decline in performance due to the grain orientation.
- There was little difference in the overall performance of the two sheathings.
- There was little difference in the initial deformations, but as the deflection increased the plywood connection stiffness decreased more quickly than did the OSB connection stiffness. Also, the ultimate capacity of the plywood-sheathed walls was slightly lower, and decreased more rapidly once the peak load was reached.

A total of seven 8 ft. x 8 ft. wood shear walls were tested according to the ASTM E564-95 Standard. Four walls were sheathed with OSB and three walls were sheathed with plywood. All sheathing panels were oriented parallel to the studs. The initial stiffness of the OSB sheathed walls was marginally higher than that of the plywood sheathed walls. The plywood shear walls were considerably more flexible than the OSB. The average peak loads for the plywood walls was higher than the average peak loads for the OSB specimens. The corresponding displacement for the plywood wall was higher than that for the OSB walls. The author concluded there is not a significant difference between the two types of construction.

<u>AISI and Santa Clara University</u> (1996)

The purpose of the research program by the Light Gauge Steel Research Group from Santa Clara University was to investigate the behavior of light-gauge steel-framed shear walls sheathed with plywood, OSB, and gypsum wall board (GWB). The project was divided into three phases. The objective of Phase 1 was to investigate the differences in the static behavior of plywood and OSB shear walls. Phase 2 included static tests on OSB and GWB walls. The final phase included cyclic testing of OSB and plywood walls. Specimens with different fastener schedules were also tested in this phase.

A total of 42 walls were tested in the program, of which 32 were sheathed on one side (either OSB or plywood), 6 were sheathed with OSB on one side and GWB on the other, and 4 were sheathed with GWB on both sides. Typical steel framing was used to construct the 8ft. x 8ft. and 4ft. x 8ft. wall specimens. The steel studs consisted of 3.5 in. web by 1.625 in. flange by 0.5 in. lip with a minimum base metal thickness of 0.033 in. (nominal 20 gauge). The steel studs were connected to 3.5 in. by 1.25 in. flange steel runner track (same thickness as the studs) with No. 8 x 0.625 in. self drilling wafer-head screws. The hold-down and anchor bolts were overdesigned in order to develop the full capacity based on the sheathing and its connection to the steel framing.

The overall behavior of the plywood and OSB panel assemblies were similar for both static and cyclic tests. Racking of the wall resulted in the pressing of the head and shank of the screw into the panel and a bending of the flange around the screw. As the lateral displacement of the wall increased, the panel pulled over the screw heads. The authors determined this to be the main source of energy dissipation. For walls with tighter screw schedules, 3 in. edge spacing and 2 in. edge spacing, the studs crippled locally at the position of the web punch-out. In the static tests, walls with panels perpendicular to the framing (with horizontal blocking at mid height) performed slightly better than similar walls with panels parallel to framing.

The authors concluded from the static tests that the 4 ft. x 8 ft. and 8 ft. x 8 ft. wall specimens have the same ultimate capacity per linear foot provided the same sheathing and orientation is followed. The maximum strength and deformation capacity of the OSB walls was found to be somewhat less than that of the plywood specimens. This contradicts Dolan's study of wood-framed shear walls where OSB was found to be slightly stiffer and stronger. Also, tighter screw schedules provided significant increase in shear capacity, but more attention must be given to the sizing of studs to develop the nominal capacity of the wall. The shear values of GWB walls were much lower as was expected.

Sugiyama

Sugiyama and Yasumura (1984) 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 recorded. The shear deformation angle is defined as displacement of the top of the wall minus slippage at the bottom of the wall divided by the total wall height.

Sugiyama (1981) defined "r", the "sheathing area ratio", in order to classify walls based on the amount of openings a wall contains. The sheathing area ratio, r, is defined as

$$r = \frac{1}{1 + \frac{A_o}{H \sum L_i}}$$

where
A_o= total area of openings
H = height of the wall
L_i = length of the full height wall segment

Sugiyama and Matsumoto (1994) 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:

$$F = r/(3-2r)$$

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

Virginia Polytechnic Institute and State University, Dolan (Draft, 1996)

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 capacity 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. walls were tested all using identical framing, sheathing, nails, and nailing patterns. Five different sheathing area ratios were used, with each wall tested once monotonically and once cyclically. The wall framing consisted of No. 2 spruce-pine-fur studs spaced 16 inches on center. Exterior sheathing was 15/32 in. plywood and the interior sheathing was 1/2 in. gypsum wallboard. Two hold-down anchors, one located at each end of the 40 ft. wall specimens, were installed to provide the end restraint required to apply 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%). 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. Nails tore through the edges of the plywood once peak capacity was reached. Hold-down anchors experienced no failure during the tests.

EXPERIMENTAL WORK

Wall Specimens

A total of four 8 ft. x 40 ft. shear wall specimens were tested in this investigation (Table 1 and Figure 1). Wall 1, Wall 2A, and Wall 4 all followed the same construction details (Table 2). Two hold-down anchors were used on each of these walls - one at each end. In addition to the hold-downs, 5/8 in. diameter bolts were used to anchor the bottom track of the specimen at 2 ft. on center. Wall 1 was fully sheathed and serves as the control wall from which shear ratios can be derived for walls with openings.

Three specimens (Wall 1, Wall 2A, and Wall 4) in this research program are intended to assess the suitability of the perforated shear wall method for light-gauge steel-framed shear walls. These walls also provide direct comparisons between wood-framed (Dolan, 1996) and steel-framed shear walls. Specimen 2B was constructed without hold-down anchors, emulating a conventional framing approach without accounting for corner or gravity load restraining forces.

All specimens were constructed of 33 mil steel. The wall framing was consistent with usual construction practices. Studs were spaced 24 inches on center, headers were constructed to span openings, and king and jack studs were also used around openings. Exterior sheathing consisted of 7/16" OSB. All panels used were 4 ft. x 8 ft. and oriented vertically. The OSB was attached with #8 screws spaced 6 in. along the perimeter and 12 in. in the field of the panels. Interior sheathing was 4 ft. by 8 ft. sheets of 1/2 in. GWB oriented vertically. The GWB was attached with #6 screws spaced 7 in. along the perimeter and 10 in. in the field. Both exterior and interior 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.

TABLE 1
Shear Wall Configurations

Wall No.	Openings		Sheathing		Anchor Bolt Spacing	Hold-downs
	Doors	Windows	Area Ratio	% Full Ht.		
1	None	None	1	100	2' o.c.	Ends
2A	6' - 8" x 4'	5' - 8" x 7' - 10 $^{1}/_{2}$"	0.76	70	2' o.c.	Ends
2B	6' - 8" x 4'	5' - 8" x 7' - 10 $^{1}/_{2}$"	0.76	70	6' o.c.	None
4	6' - 8" x 4' 6' - 8" x 12'	4' x 7' - 10 $^{1}/_{2}$"	0.48	40	2' o.c.	Ends

TABLE 2
Wall Materials and Construction Data

COMPONENT	CONSTRUCTION AND MATERIALS
Framing Members	33 mil (20 gauge) studs and track. Stud connected to track w/one #8 screw in each flange. Fy = 52.4 ksi
SHEATHING	
Exterior	7/16" OSB, #8 screws with 6-12 spacing, 4' x 8' sheets installed vertically.
Interior	1/2" Gypsum Wallboard, #6 screws with 7-10 spacing, 4' x 8' sheets installed vertically, joints taped.
HEADERS	
4'-00" opening	2-2"x4"x43mil connected back-to-back with two #10 screws 24" o.c., one jack and two king studs at each end. Header connected to king stud with clip angle and four #10 screws.
7'-10½" opening	2-2"x8"x54mil connected back-to-back with two #10 screws 24" o.c., one jack and two king studs at each end. Header connected to king stud with clip angle and four #10 screws.
11'-10½" opening	2-2"x10"x68 mil connected back-to-back with two #10 screws 24" o.c., one jack and two king studs at each end. Header connected to king stud with clip angle and four #10 screws.
STRUCTURAL BASE CONNECTIONS (BOTTOM OF WALL)	
Hold-down	Simpson Strong-Tie HD10, with #10 hex head self tapping screws and 5/8" diameter bolts.
Anchor Bolts	5/8" diameter bolts and washers with 6" stud section reinforcing track at anchor bolt locations.
LOADING TUBE CONNECTIONS (TOP OF WALL)	
Above Openings	Two #10 hex head screws attaching header to tube @ 2' o.c.
No Openings	1/2" diameter bolts with 13/8" diameter washers @ 2' o.c.
Note: 1 mil = 1/1000 inch	

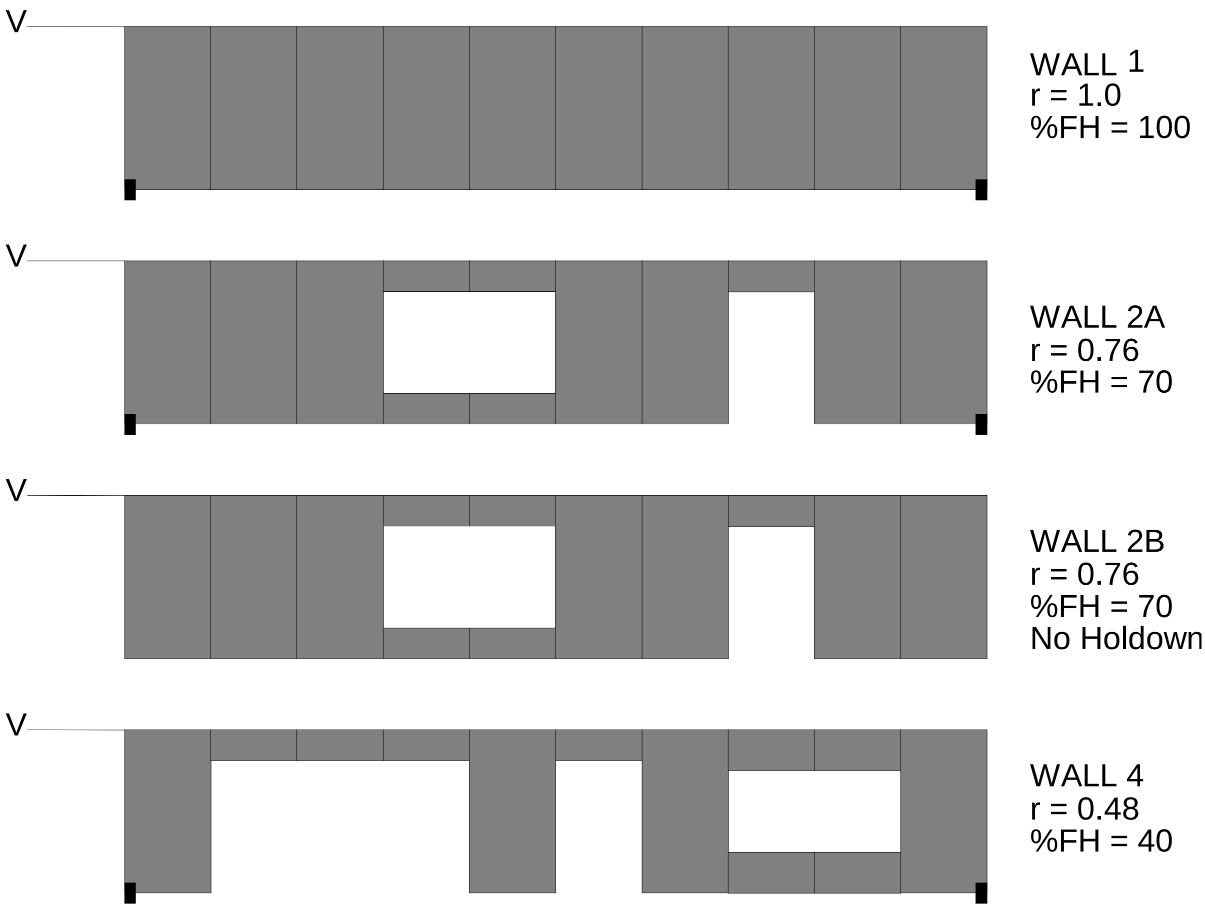


FIGURE 1
Shear Wall Configurations

Monotonic Test Procedures

The shear walls were tested in a horizontal position. A hydraulic actuator, with a range of 12 inches and capacity of 115,000 lbs, applied the load to the top right corner of each shear wall through a 4x4 structural steel tube (Figure 2). A 1/2" thick steel plate was welded to the end of the tube to provide a uniform loading area for the actuator. A 50,000 lbs. 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. 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 3). It should be noted that the sheathing was installed such that interference with the top and bottom load beams was minimized.

FIGURE 2
Plan View of Shear Wall Testing Layout

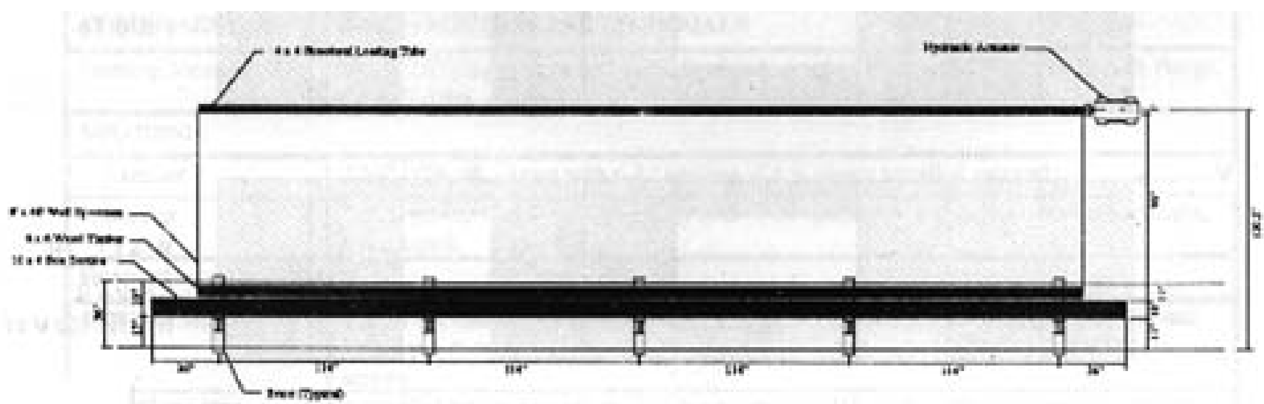
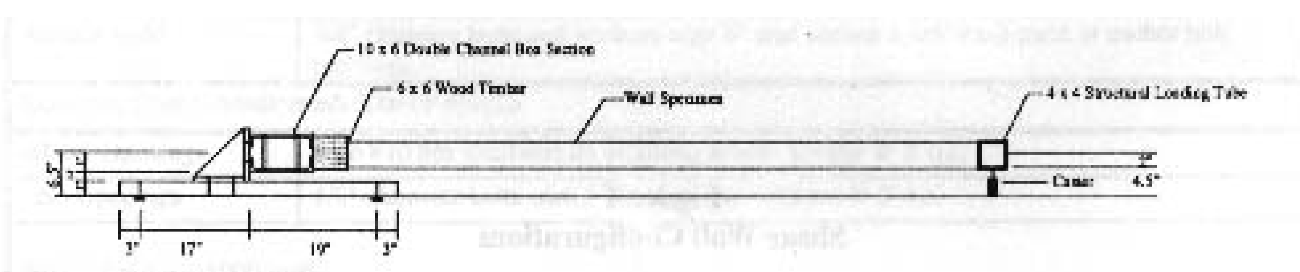


FIGURE 3
Elevation of Shear Wall Testing Layout



Three linear variable differential transformers (LVDT) were used to measure the displacement of the specimens during the test. LVDT #1 measured the horizontal displacement of the top of the wall. LVDT #2 measured the horizontal displacement, or slip, of the bottom track of the specimen. LVDT #3 was used to measure 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 ten minute period. Data from the load cell and 3 LVDTs were collected 1 time per second. Each of the four wall configurations was tested once. Items of interest are ultimate shear load capacity, stiffness, and failure modes of the walls. Load-displacement curves were plotted for each of the wall specimens to better understand and compare the behavior of the walls during the test.

RESULTS

Force-Displacement Response

The response of all shear wall specimens to the monotonic loading history are shown in the force-displacement curves of Figure 4. Initial response to load was linear and was characterized by large stiffness. The peak load, as well as the corresponding displacement, was gathered directly from the data. These loads and displacements are listed in Table 3. The equation developed by Sugiyama and Matsumoto conservatively predicts the ultimate capacity of the steel-framed specimens (Figure 5). Figure 5 suggests the relation between sheathing area ratio and peak load more closely follows the equation F = r/(2-r) for the sheathing area ratios tested. Additional testing should be conducted to confirm this finding.

FIGURE 4
Force-Displacement Response

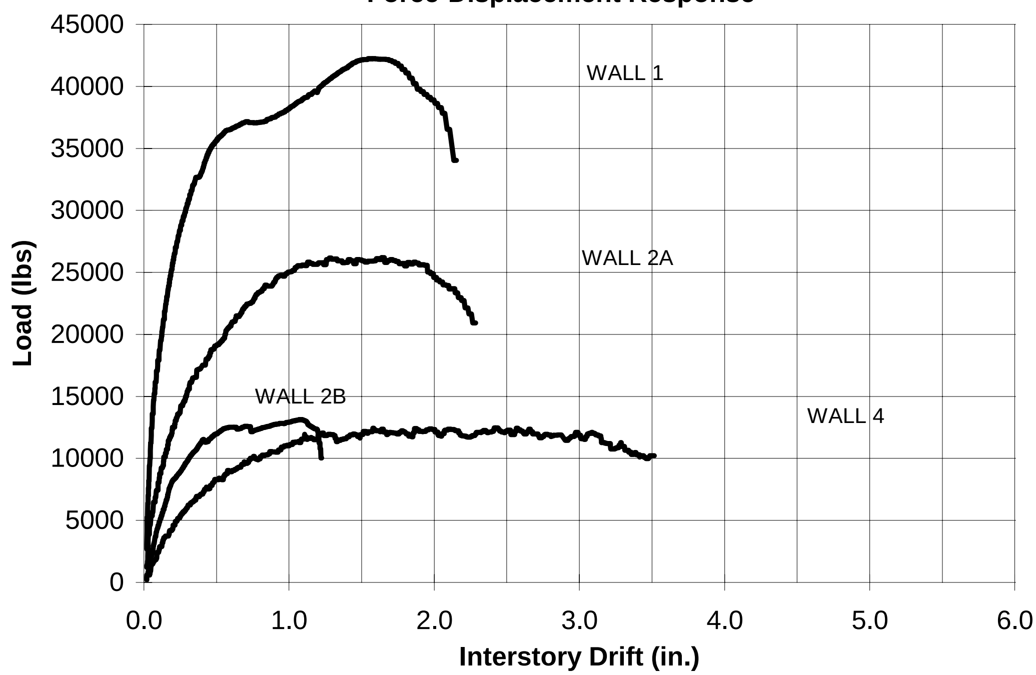
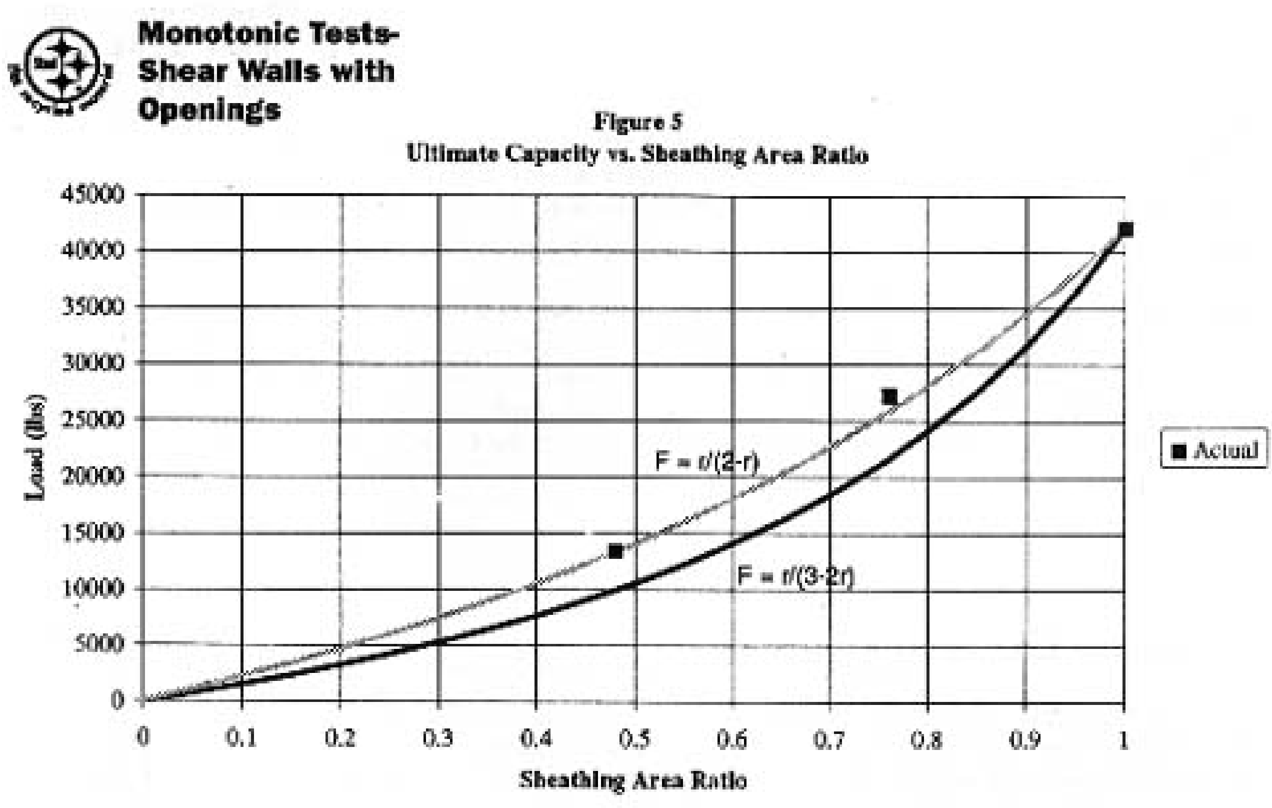


Figure 5
Ultimate Capacity vs. Sheathing Area Ration



The initial portion of the force-displacement curves were fitted with a linear least-squares trend, the slope of which is taken as initial stiffness. That portion of the curve for which the magnitude of force did not exceed 40 percent of the peak load was used in the calculation (safety factor of 2.5). Generally, the force-displacement data in this range demonstrate a strong linear relation. The initial stiffnesses are listed in Table 3. As was expected the specimens with the large sheathing area ratio experienced a larger initial stiffness.

The toughness of a wall can be quantified by its ability to dissipate energy while deforming. Cumulative energy dissipation was obtain by calculating the area under each force-displacement curves using Simpson's Method (Donaldson, 1993). Again, the walls with the larger sheathing area ratio experienced a greater ability to dissipate energy (Table 3).

TABLE 3
Force-Displacement Data from Monotonic Tests

	Wall Specimens		
	1	2A	3
Sheathing Area Ratio	1.0	0.76	0.48
Predicted Shear Ratio[1]	1.0	0.51	0.24
Actual Shear Ratio	1.0	0.62	0.30
Peak Load (kips)	42.2	26.2	12.5
Displacement (in) @ Peak Load	1.54	1.64	2.41
Initial Stiffness (kip/in)	227.8	72.3	24.0
Energy Dissipated (kip*in)	77.3	49.7	36.7

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

Failure Modes

Similar modes of failure were observed in the specimens containing the hold-down anchors. The initial loading was highly linear until the interior sheathing, GWB, began to pull through the screws. This resulted in a slight reduction in stiffness. As the load approached ultimate capacity the OSB experienced cracking at perimeter screw connections and usually tore out at the top track connections. Also, the screws attaching the loading tube to the specimens above the openings withdrew from the headers near the ultimate capacity. These combined failures resulted in a reduction in load and led to failure of the specimen. It was noted during deconstruction of the specimens that many of the studs experienced weak axis bending approximately 12 inches from the top of the specimen. This failure followed the OSB tear-out around screw connections at the top of the shear walls. In effect, the OSB unzipped from the top until a sufficient weak axis moment was induced to cause failure of the studs.

An additional wall specimen, Wall 2B, was tested at the end of the test program. This wall was constructed the with no hold-downs and with anchor bolts 6 ft. on center. Strength enhancing effects, such as gravity loads and corner framing, realized in actual construction applications, were not investigated. Table 4 lists relevant data collected. Specimen 2B experienced failure of the interior sheathing as did the specimens containing hold-downs. However, at the first anchor bolt located 12 inches from the end of the wall, the bottom track and OSB were unable to distribute the uplift forces carried by the hold-down in Specimen 2A. The OSB was in tact at the completion of the test, except at the location of the first anchor bolt where the bottom track failed in bending due to uplift. Specimen 2A and Specimen 4 also experienced significant uplift at openings where no hold-downs were present because of the flexibility of the bottom track. Without additional research to quantify the restraint provided by gravity loads and corner framing, it appears that the perforated shear wall method would yield unconservative results when hold-downs are omitted.

TABLE 4
Force-Displacement Data from Monotonic Test (No Hold-downs)

	Specimen 2B
Sheathing Area Ratio	0.76
Predicted Shear Ratio[1]	0.51
Actual Shear Ratio	0.31
Peak Load (kips)	13.1
Displacement (in) @ Peak Load	1.08
Initial Stiffness (kip/in)	51.2
Energy Dissipated (kip*in)	12.8

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

CONCLUSIONS

The data presented suggests that light-gauge steel shear walls with wood-based structural panels are viable shear resisting system for light construction applications. Resistance to drift histories is stable and features a relatively large initial stiffness. The calculated shear capacity using the empirical equation developed by Sugiyama and Matsumoto appears valid, but conservatively estimates the ultimate capacity. A more accurate empirical relationship needs additional validation. The lateral load resisting mechanisms for both wood and steel shear walls seems to be similar. Specimens with hold-downs eventually experienced failure of the OSB at edge connections which caused the ultimate reduction in capacity. For the testing conditions, hold-downs reduced uplift and increased the ultimate capacity by allowing a greater number of sheathing fasteners to actively participate in resisting shear.

RECOMMENDATIONS

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

- Study additional sheathing area ratios to verify a curve similar to that of Sugiyama and Matsumoto for wood-framed shear walls and confirm the empirical equation F = r/(2-r);
- Variations in anchoring schematics (i.e. hold-downs at ends, hold-downs at ends and at openings, no hold-downs, etc.) for development of efficient design practices for all levels of lateral loads;
- Study the effects of gravity loads on shear capacity as predicted by the perforated shear wall method;
- Study the effects of corner framing on end restraint of perforated shear walls;
- Repeat tests using an acceptable cyclic test protocol;
- Develop a design practice guideline for perforated shear walls using light-gauge steel-framing.

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APPENDIX A

Metric Conversions

1 mil = 1/1000 inch

1 kip = 1000 lbs

1 inch = 25.40 mm

1 kip = 4.448 kN