Design of Light-Gauge Steel Shearwalls for Residential Construction in High Wind and Seismic Conditions

by

Jay H. Crandell[1], Shawn McKee[1], William F. Freeborne[2]

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 wind and seismic forces has prompted research to fill this need. This paper reviews the current status of shearwall design data using light-gauge steel framing and wood-based structural panels. Pilot tests to investigate the applicability of a new design method for shearwalls, known as the "perforated shearwall" method, are presented. Findings from the pilot tests indicate that the method is applicable to light-gauge steel walls and that additional testing is needed to improve the method since it appears conservative.

KEYWORDS: design; light-gauge steel; seismic; shearwall; testing; wind

1. INTRODUCTION

Light-gauge steel framing has recently become a viable alternative construction method in the U.S. residential construction market. However, its competitive use is dependent on efficient methods of design, particularly in the area of wall bracing to provide resistance to lateral loads from wind and seismic forces. Recent research has provided the basic design data for light-gauge steel shearwalls, but an efficient method of design has not been developed, particularly for residential construction applications.

The objective of this paper is to present findings from pilot tests on long shearwalls with openings to investigate the applicability of an efficient shearwall design method known as "perforated shearwalls". This empirical method is believed to be an efficient design approach for residential construction and similar light-frame buildings.

2. BACKGROUND

About 93 percent of the houses in the U.S. are built using traditional wood framing practices, which have performed reasonably well in meeting the large and varied housing needs in the country. However, recent price increases in framing lumber have challenged builders and designers to seek alternative materials and methods for the construction of homes. Light-gauge steel framing has recently grown in its competitive capability as an alternative framing method for homes in the U.S.

While interest in steel framing and other alternative materials for home construction is greater than ever in the U.S., the use of such "non-conventional" materials face many barriers including:

- regulatory
- educational
- economic
- technical

[1]Structures and Materials Division, NAHB Research Center, Inc., Upper Marlboro, Maryland, USA, 20774-8731
[2]Department of Policy Development and Research, U.S. Department of Housing and Urban Development, Washington, DC, USA, 20464

The major regulatory barrier is related to the recognition of steel framing in prescriptive building code provisions. Homes in the U.S. are traditionally built using prescriptive building code requirements that specify connections, member sizes, and details for various components of the home. These provisions have historically focused on the application of light-frame wood construction to basic home design conditions. A builder can construct homes following these provisions and a building code official can inspect the construction following the prescriptive requirements in the building code.

This process has been very efficient, largely because builders and code officials are have become very familiar with the materials and methods through decades of use. Also, the prescriptive building code requirements can be applied without the added time and cost of generating an engineered design for every home. Homes and other structures built in this manner are considered "non-engineered" structures.

A recent public-private effort to develop prescriptive construction requirements for light-gauge steel framing in residential construction has resulted in adoption of steel framing in the major residential building code in the U.S. (CABO 1997; HUD 1996). Through time this should result in the removal of barriers to steel framing as local political jurisdictions in the U.S. adopt the newer building code provisions.

With the recent building code success mentioned above, educational barriers are one of the major issues facing use of light gauge steel framing in residential construction. Educational efforts are underway, to help building code officials and builders use light-gauge steel framing appropriately and efficiently. The framing system is very similar to traditional wood frame construction; however, cutting and fastening methods are somewhat different. Also, alignment of vertical and horizontal load bearing members is required for "in-line" framing -- a requirement which is not common or necessary in the traditional wood framing practices.

Economic issues surrounding alternative materials are driven primarily by labor and material costs associated with the final product. These costs also include the "up-front" costs that builders experience when switching to new building systems -- otherwise known as the "learning curve". However, to be competitive with the current home-building process in the U.S., the use of steel framing and other alternatives must be streamlined to allow for its introduction as a site-built or manufactured product with a "pre-approved" regulatory status. This lowers the builder's expenses prior to construction by minimizing design, planning, and approval activities through a straight forward prescriptive or "cook book" approach. The materials must also be readily available with minimal shipping and handling costs. Overall costs in the completed project must finally compare favorably with the builder's current practices to provide an incentive to change. One method to reduce these costs is to eliminate over-design through technological advancement.

Technical barriers have been largely solved through the development of the *Prescriptive Method for Cold-Formed Steel Framing* (HUD 1996) which was used as the basis for the approved building code provisions. However, these provisions are primarily based on conservative engineering specifications and calculations which add unnecessary costs to light-gauge steel framing. In particular, the construction requirements did not include wall bracing requirements in high wind and seismic load conditions because of the lack of an efficient design method for steel shearwalls and because the initial goal was to at least cover lower seismic and wind load conditions. Steel strap bracing was also not included because of the special detailing and anchoring required to achieve good performance with this bracing method, even in lower wind and seismic conditions. Therefore, a recent research goal has been to develop an efficient wall bracing approach for light gauge steel framing to resist lateral loads from wind and seismic forces. Other research has included system performance of back-to-back C-shape headers and walls under combined axial and bending loads. The objective of this research has been to make steel framing more efficient in the competitive residential construction market.

The traditional design method for shearwalls utilizes hold-down brackets on each wall segment to restrain it from rotating prior to developing the shear capacity of the panel. While this design method is widely used for engineered structures, it is not always the most efficient design method for light residential structures of either wood or steel framing.

Hold-downs require careful placement and pre-planning in the construction phase. In typical residential construction, this level of precision is difficult to control consistently. As a result, the use of many hold-down brackets which must be permanently embedded in the concrete prior to framing often results in construction errors that must be corrected either by changes in the framing layout or replacement of the anchor. These problems may be very costly. Therefore, a more efficient shearwall design method is needed to minimize the number of hold-down brackets, yet still provide adequate performance of shearwalls.

3. LITERATURE REVIEW

Recent racking tests of steel-framed walls with wood-based structural panel sheathing has provided data for use by engineers using traditional shearwall design methods (Serrette 1996). These tests included monotonic and cyclic loading on 4 ft.x 8 ft. (1.2 m x 2.4 m) and 8 ft. x 8 ft. (2.4 m x 2.4 m) wall specimens following the ASTM E 564 test method (ASTM 1995). The specimens included nominal 2x4 and 2x6 steel studs sheathed with two types of wood structural panels -- oriented strand board (OSB) and plywood. Thicknesses of steel studs were 33 mil (0.84 mm) and 43 mil (1.09 mm). Various screw patterns were used and some panels also included ½ in. (12.7 mm) gypsum wall board on the opposite side of the structural panels. Data from these tests has resulted in a recent building code approval giving shear capacities for light gauge steel-framed shearwalls (UBC 1997).

Yasumura and Sugiyama (1984) conducted several small scale tests on wood frame shearwalls with various opening amounts and arrangements. Data from these tests were used to develop an empirical equation for predicting the shear capacity of a wall with openings. The experimenters found that the shear capacity of the scaled walls with openings were related to the wall's capacity without openings by use of a parameter, r, known as the sheathing area ratio. The sheathing area ratio is calculated by the following equation:

$$r = \frac{1}{1 + \dfrac{A_o}{H \, \Sigma \, L_i}}$$

where,

r	=	sheathing area ratio
A_o	=	area of openings in the wall
H	=	height of the wall
L_i	=	length of the wall segment i without openings.

The sheathing area ratio, r, is then used to predict a shear load ratio, F, for the wall which is calculated by the following equation:

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

The shear load ratio, F, represents the ratio of the wall's capacity with openings to the wall's capacity without openings.

For a given wall with openings, its capacity can be calculated by first determining the sheathing area ratio, r, and then determining the shear load ratio, F. The capacity of the wall is then determined by multiplying the shear capacity of the wall without openings by F. The only restraints (hold-down brackets) necessary are located at the ends of the walls (see Figure 1). In actual construction this restraint would be placed at the major corners of the structure only. This shearwall design method is known as the "perforated shearwall method". Yasumura and Sugiyama (1984) also developed similar relationships to predict wall capacity at various deflection angles (drifts) up to the ultimate capacity.

Recent full-scale tests of wood frame walls have verified the applicability of the perforated shearwall method using both monotonic and cyclic loading (Dolan 1996). The walls were 8 ft. x 40 ft. (2.4 m x 12.2 m) with various opening amounts and arrangements. Hold-down brackets were placed only at the ends of the walls. A major U.S. building code has recently approved this design approach for wood frame shearwalls (SBC 1996).

In testing that is ongoing, shear capacities of wood frame walls designed using the segmented shearwall approach and the perforated shearwall approach have been compared (Dolan 1997). Conventionally-framed wood walls without any hold-down brackets were also tested with identical opening configurations. Additional testing is planned to quantify the restraint provided by corners so that wood frame shearwalls can be designed without the use of any hold-down brackets in lower lateral load conditions. None of these tests have included walls with steel framing and, therefore, a similar approach to the efficient design of light-gauge steel framing systems has not been found in the literature.

4. EXPERIMENTAL METHOD

To investigate the applicability of the perforated shearwall method to walls framed with light-gauge steel, four pilot tests were conducted using 8 ft. x 40 ft. (2.4 m x 12.2 m) walls with configurations of openings similar to that discussed in the literature search (Dolan 1996; Dolan 1997). The walls were constructed as shown in Figure 1 and Table 1. The construction was nearly identical to tests conducted by Dolan (1996) with the exception of light-gauge steel framing instead of wood framing and screws instead of nails. It should be noted that one of the walls was tested without hold-down brackets to give an indication of a "lower bound" condition.

A monotonic shear load was applied to a steel tube connected to the top of the wall by a hydraulic actuator with a load cell as shown in Figure 2. The load was applied at a rate of 0.6 inches per minute until failure. Deflections were measured using LVDTs. Readings from the load cell and the LVDTs were taken at 0.5 second intervals. The data was reduced to provide load and deflection plots for the tested walls closely following ASTM E 564 procedures (ASTM 1995).

5. TEST RESULTS AND DISCUSSION

Load deflection plots for the three walls are shown in Figure 3. The plots show a high initial stiffness of these walls and considerable shear capacity. However, the ductility appears to be lower than similar wood-frame construction. Ductility is defined as the capability of the wall to continue to deform after peak load without significant loss of capacity. While ductility is not so important for design against wind loads, it is important when designing for seismic loading. The primary failure mode for these walls was tear-out of the screw fasteners from the edges of the oriented strand board sheathing. One wall was tested without any hold-down restraints at the end of the wall (wall '2B'). As expected, the failure mode was bending of the bottom track at the first anchor bolt located about 1 ft. (0.3 m) from the loaded end of the wall. The capacity of this wall was considerably lower than the similar restrained wall (wall '2A'). For this wall, the inclusion of other system effects (i.e. corner framing, gravity loads, etc.) would likely improve the tested performance so that it more accurately represents the shear capacity actual construction.

The detailed data for walls 1, 2A, and 4 are presented in Table 2 along with a comparison of the actual shear load ratio, F, to that predicted by the empirical equation develop by Yasumura and Sugiyama (1984). It is evident from these pilot tests that the perforated shearwall method produces a conservative prediction of the wall shear capacity for the range of openings covered in this study (represented by the sheathing area ratio, r). It also appears that the relationship of F to r is somewhat linear for the range of sheathing area ratios, r, investigated in these tests. Additional testing is needed to adjust the empirical equation for F to better predict the performance of these types steel-framed walls and to provide validation of the empirical equation at lower sheathing area ratios.

6. CONCLUSIONS

The main findings of this study are as follows:

1. Adequate data now exists for the design of light-gauge steel frame walls sheathed with wood-based structural panels.

2. A new shearwall design method, known as the perforated shearwall method, has been pilot tested on long steel framed walls with openings. The test results indicate that this method is a viable design approach for light-gauge steel shearwalls resisting lateral loads from wind and earthquakes.

3. Light-gauge steel walls with wood-based structural sheathing materials are adequately stiff and strong when only the ends of the wall are restrained against uplift, not requiring multiple hold-downs at individual wall segments between openings.

4. Additional testing is needed to finalize and improve the perforated shearwall method when applied to light-gauge steel walls. Also, an appropriate consideration of the ductility of these walls is needed for the purposes of seismic design analysis.

7. ACKNOWLEDGMENTS

The authors gratefully acknowledge the financial support provided by the American Iron and Steel Institute, the National Association of Home Builders, and the U.S. Department of Housing and Urban Development.

8. REFERENCES

ASTM (1995), *Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings*, American Society for Testing and Materials (ASTM), West Conshohocken, Pennsylvania, 1995.

CABO (1997), *One and Two Family Dwelling Code 1996/1997 Amendments*, Council of American Building Officials (CABO), Falls Church, Virginia, 1997.

Dolan, J. Daniel (1996), report unpublished at the time of this writing, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.

Dolan, J. Daniel (1997), report unpublished at the time of this writing, Virginia Polytechnic Institute and State University, Blacksburg, Virginia.

HUD (1996), Prescriptive Method for Residential Cold-Formed Steel Framing - First Edition, prepared for the U.S. Department of Housing and Urban Development (HUD) by the NAHB Research Center, Inc., Upper Marlboro, Maryland, May 1996.

SBC (1996), *Standard Building Code - 1995/1996 Amendments*, Southern Building Code Congress International, Inc., Birmingham, Alabama, 1996.

Serrette, Reynaud (1996), Shear Wall Values for Light Weight Steel Framing, prepared for the American Iron and Steel Institute, Washington, DC, by Santa Clara University, Santa Clara, California, January 1996.

UBC (1997), *Uniform Building Code - Volume 2*, International Conference of Building Officials (ICBO), Whittier, California, 1997.

Yasumura, Motoi and Hideo Sugiyama (1984), "Shear Properties of Plywood-Sheathed Wall Panels with Opening", Trans. of A.I.J. No. 338, April, 1984 (English translations of work first reported in 1976).

Table 1: Wall materials and construction data

Component	Construction and Materials
Framing Members	1-5/8 in x 3-1/2 in. x 33 mil (41.2 mm x 88.9 mm x 0.84 mm) steel studs and 33 mil (0.84 mm) track. Stud connected to track w/one #8 wafer head, self-drilling tapping screw in each flange.
Sheathing	
Exterior	7/16 in. (11.1 mm) oriented strand board (OSB) with #8 bugle head, self-drilling tapping screws spaced at 6 in. (15.2 cm) on edges of panel and 12 in. (30.4 cm) in field of panel, 4 ft. x 8 ft. (1.2 m x 2.4 m) sheets installed vertically on framing.
Interior	1/2 in. (12.7 mm) gypsum wallboard with #6 screws spaced at 7 in. (17.8 mm) on edges and 10 in. (25.4 cm) in the field, 4 ft. x 8 ft. (1.2m x 2.4m) sheets installed vertically, joints spackled and taped.
Structural Base Connections (Bottom of Wall)	
Hold-downs	Simpson HD10 with 9,900 lb (44 kN) allowable load connected to single end studs only
Anchor Bolts	5/8 in. (15.9 mm) diameter tie rods with 6 in. (15.2 cm) stud section reinforcing track at anchor bolt locations.
Loading Tube Connections (Top of Wall)	
Above Openings	Two #10 hex head self-drilling tapping screws attaching headers and track to tube at a spacing of 24 in. (61 cm)[3]
Other Locations	1/2 in. (12.7 mm) diameter bolts with 1-$^{3}/8$ in. (34.9 mm) washers spaced at 24 in. (61 cm) connecting track to tube

[3]Anchor bolts were not used at header locations because there was no practical method of attachment with 1/2 in. (12.7 mm) diameter bolts. The #10 screws are considered to be a conservative substitution. Light-gauge steel headers were constructed of back-to-back C-shapes.

Table 2: Force-displacement data from monotonic tests

	Wall Specimens		
	1	2A	3
Sheathing Area Ratio, r	1.0	0.76	0.48
Predicted Shear Ratio, F[4]	1.0	0.51	0.24
Actual Shear Ratio, F	1.0	0.62	0.30
Peak Load	42.2 kips (187.7 kN)	26.2 kips (116.5 kN)	12.5 kips (55.6 kN)
Displacement@Peak Load	1.54 in. (3.9 cm)	1.64 in. (4.2 cm)	2.41 in. (6.1 cm)

[4]The predicted shear ratio is based on the empirical formula developed by Yasumura and Sugiyama (1984) for wood-framed shear walls.

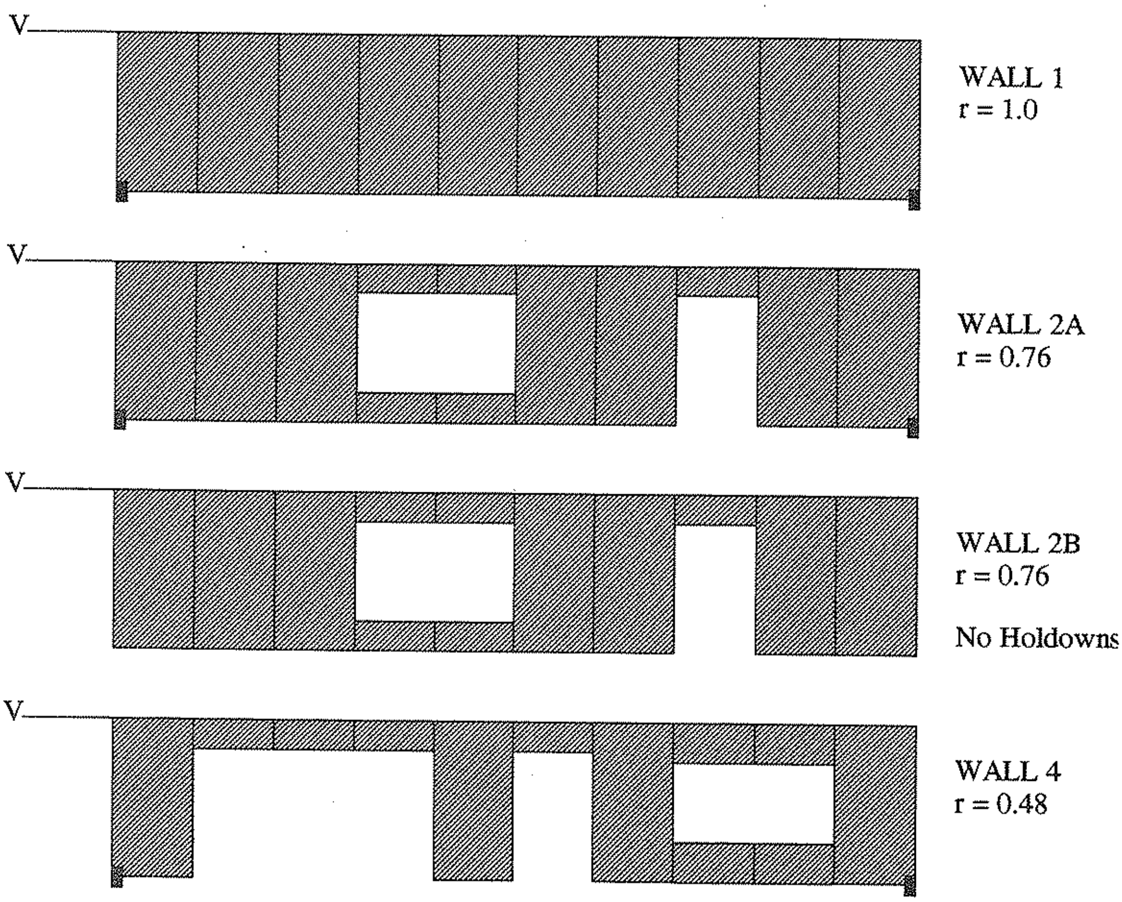

Figure 1: Wall construction

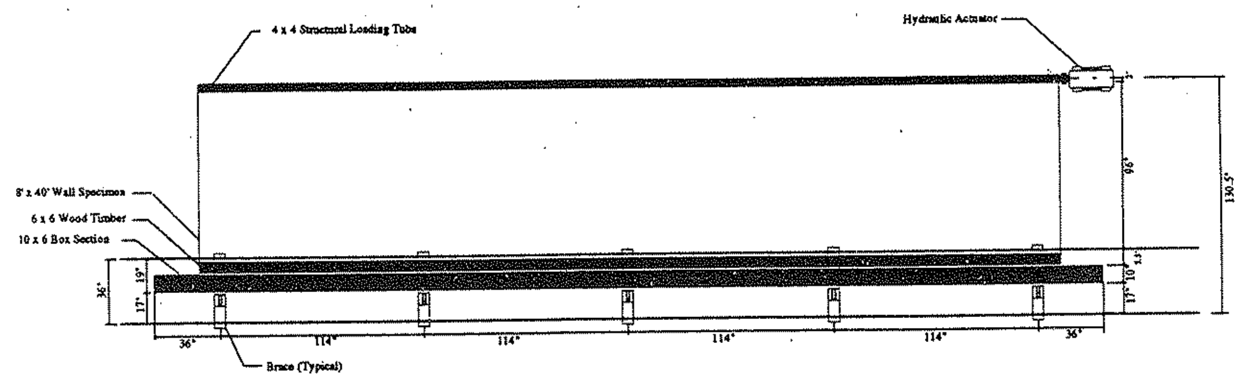

Figure 2: Test apparatus

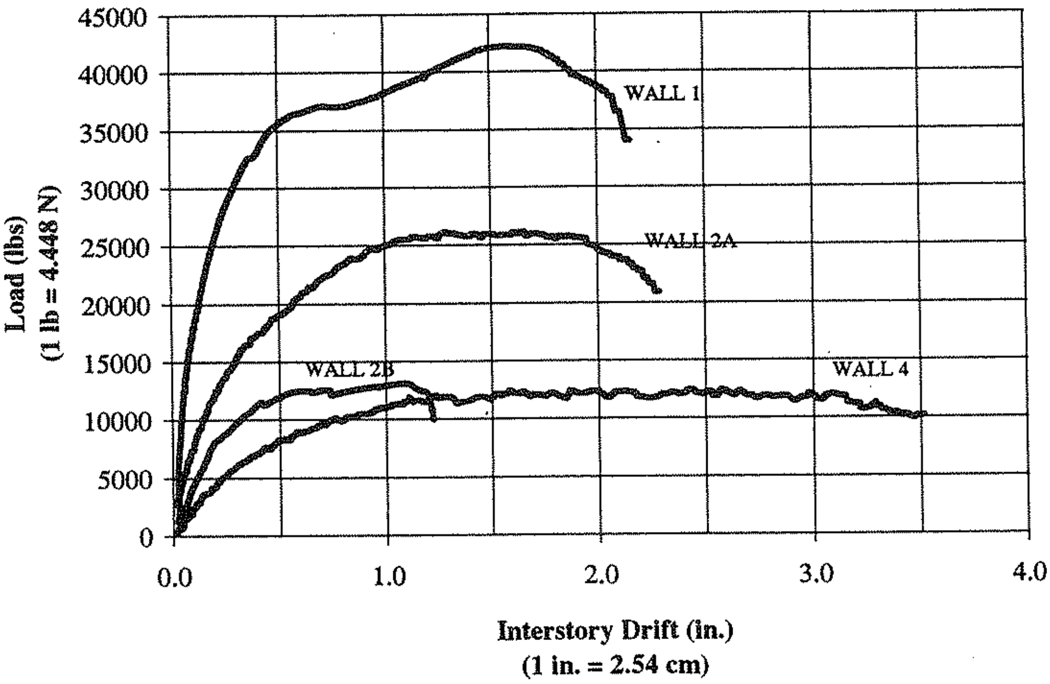

Figure 3: Force-displacement response