Monotonic Tests of Long Shear Walls with Openings

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INTRODUCTION

Wood frame shear walls are a primary lateral force resisting element in wood frame structures. Traditional shear wall design requires fully sheathed wall sections restrained against overturning. Their behavior is often considered analogous to a deep cantilever beam with the end framing members acting as "flanges" or "chords" to resist overturning moment forces and the panels acting as a "web" to resist shear. This analogy is generally considered appropriate for wind and seismic design. Overturning and shear restraint, and chord forces are easily calculated using principles of engineering mechanics. While shear resistance can be calculated as well, tabulated shear resistance's for varying fastener schedules are often used.

Traditional design of exterior shear walls containing openings, for windows and doors, involves the use of multiple shear wall segments. Each is required to be fully sheathed and have overturning restraint supplied by structure weight and/or mechanical anchors. The design capacity of shear walls is assumed to equal the sum of the capacities for each shear wall segment. Sheathing above and below openings is typically not considered to contribute to the overall performance of the wall.

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 1996 Revised Edition* and the *Wood Frame Construction Manual for One- and Two- Family Dwellings - 1995 High Wind Edition.* The perforated shear wall method consists of a combination of prescriptive provisions and empirical adjustments to design values in shear wall selection tables for the design of shear wall segments containing openings. When designing for a given load, shear walls resulting from this method will have a reduced number of overturning restraints than a similar shear wall constructed with multiple traditional shear wall segments.

A significant number of monotonic tests of one-third scale models and shorter full-sized walls provide validation of the perforated shear wall method. This study provides additional information about the performance of long, full-sized, perforated shear walls tested under monotonic and cyclic loads. Cyclic tests were performed to establish conservative estimates of performance during a seismic event. Results of monotonic tests are presented in this report (TE-1996-001) and cyclic test results are reported in Dolan and Johnson (1996) (TE-1996-002). A detailed description and discussion of the complete investigation is presented in Johnson (1997).

OBJECTIVES

Results of an experimental study of the performance of shear walls meeting the requirements of the perforated shear wall method are reported. The objectives of this study were 1) determine the effects of openings on full-size wood frame shear walls tested monotonically and cyclically, 2) determine if the perforated shear wall method conservatively predicts capacity.

BACKGROUND

The perforated shear wall design method appearing in the *Standard Building Code 1996 Revised Edition (SBC)* and the *Wood Frame Construction Manual for One- and Two- Family Dwellings - 1995 High Wind Edition (WFCM)* is based on an empirical equation which relates the strength of a shear wall segment with openings to one without openings. The empirical equation developed by Sugiyama (1993) forms the basis of adjustment factors in Table 2313.2.2 in the SBC and Supplement Table 3B in the WFCM. Tabulated adjustment factors are used to reduce the strength of a traditional fully sheathed shear wall segment for the presence of openings.

In accordance with SBC and WFCM, and for the purposes of this study, a perforated shear wall must have: 1) mechanical overturning and shear restraint; 2) tie-downs to provide overturning restraint and maintain a continuous load path to the foundation where any plan discontinuities occur in the wall line; 3) minimum length of full-height sheathing at each end of the wall (based on height-to-length ratio for blocked shear wall segments as prescribed by the applicable building code.); 4) maximum ultimate shear capacity of less than or equal to 1500 plf.

Prescriptive provisions and empirical adjustments are based on parameters of various studies conducted on shear walls with openings. Many of the prescriptive provisions are necessary to meet conditions for which walls in previous studies were tested. Empirically derived adjustment factors, or shear capacity ratios, for the perforated shear wall method are based on an equation developed by Sugiyama (1994) for predicting shear capacity ratios.

The shear capacity ratio, or the ratio of the strength (or stiffness) of a shear wall segment with openings to the strength (or stiffness) of a fully sheathed shear wall segment without openings, is determined by Equation (1):

$$F = r / (3 - 2 \cdot r) \tag{1}$$

where 'F' is shear capacity ratio and 'r' is the sheathing area ratio.

Sheathing area ratio is used to classify walls based on the size of openings present. It is determined by: a) the ratio of the area of openings to the area of wall and b) the length of wall with full height sheathing to the total length of the wall. Sheathing area ratio parameters are illustrated in Figure 1, and the ratio can be calculated by the following expression:

$$r = \frac{1}{(1 + \frac{\alpha}{\beta})} \tag{2}$$

$$\alpha = \frac{\text{Area of openings}}{H \cdot L} \tag{3}$$

$$\beta = \frac{\sum L_i}{L} \tag{4}$$

where α = opening area ratio,

β = wall length ratio,

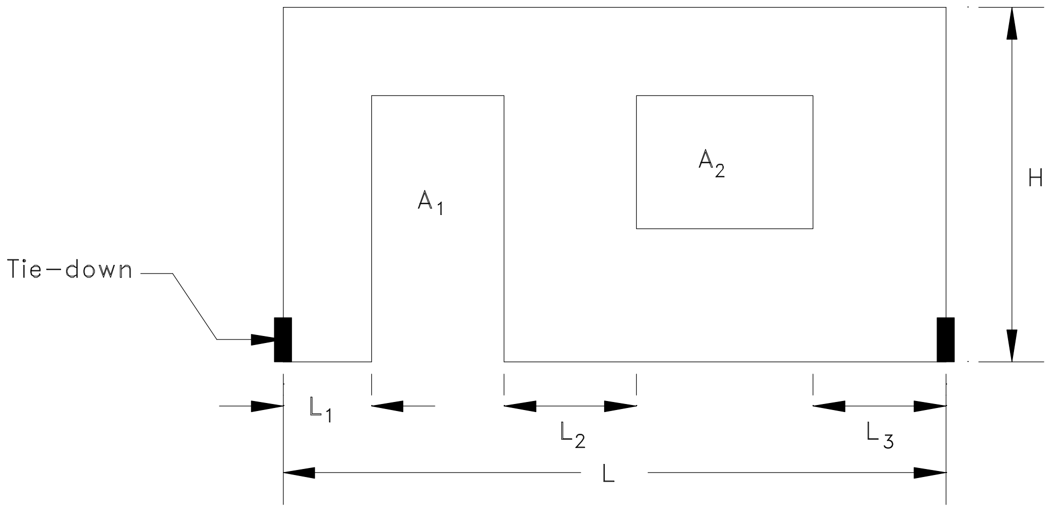


Figure 1: Sheathing area ratio variables

ΣL_i = sum of the length of full height sheathing,
L = shear wall length, and
H = wall height.

Tabulated shear capacity ratios or opening adjustment factors appearing in the SBC and WFCM are based on Equation (1) assuming that the height of all openings in a wall are equal to the largest opening height. The result is that SBC and WFCM tabulated shear capacity ratios or opening adjustment factors for walls containing openings of varying height are smaller than would be calculated using Equation (1). For this study, Equation (1) will be used to predict the performance of shear walls with openings constructed in accordance with the parameters of the perforated shear wall design method. Tabulated shear capacity ratios appearing in the SBC and WFCM result in slightly more conservative estimates of performance.

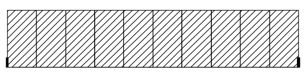
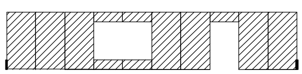
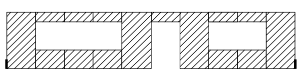
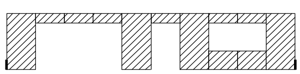
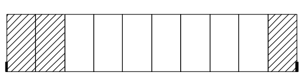
TEST PROGRAM

Monotonic and cyclic tests were conducted on pairs of walls for each of the five wall configurations shown in Table 1. Size and placement of openings was selected to cover the range of sheathing area ratios, 'r', encountered in light wood frame construction. Perforated shear wall performance under monotonic and cyclic loads over the range of sheathing area ratios as well as a comparison between monotonic and cyclic performance is reported in this study. Results of monotonic tests are presented in this report (TE-1996-001) and cyclic test results are reported in Dolan and Johnson (1996) (TE-1996-002).

Specimen Configuration

Five 40 feet long by 8 feet tall walls were included in the monotonic investigation. Each wall used the same type of framing, sheathing, nails, and nailing patterns. Table 1 lists the opening dimensions and illustrates the opening locations for each wall configuration included in the investigation. Wall A (r = 1.0) has no openings and is necessary for determining the capacity of the fully sheathed condition. The ratio of strength of Walls B through E to Wall A will be compared directly to the shear capacity ratio, F, calculated using Equation (1).

Table 1: Opening sizes for each wall configuration included in investigation

Wall Configuration[1]	Wall Type	Sheathing Area Ratio, (r)	Opening Size	
			Door	Window[2]
	A	1.0	-	-
	B	0.76	6'-8" x 4'-0"	5'-8" x 7'-$10^{1}/_{2}$"
	C	0.55	6'-8" x 4'-0"	4'-0" x 11'-$10^{1}/_{2}$" 4'-0" x 7'-$10^{1}/_{2}$"
	D	0.48	6'-8" x 4'-0" 6'-8" x 12'-0"	4'-0" x 7'-$10^{1}/_{2}$"
	E	0.30	(Sheathed at ends)[3] 8'-0" x 28'-0"	-

1: All walls are framed with studs spaced at 16 inches on center. Shaded areas represent sheathing.

2: The top of each window is located 16 inches from the top of the wall.

2: Wall E has studs along the full length of wall but is sheathed only at the ends of the wall.

Materials and Fabrication Details

Table 2 summarizes materials and construction details used for the wall specimens. Included are the size of headers and jack studs used around openings.

Wall framing consisted of double top plates, single bottom plates, double end studs, and double or triple studs around doors and windows. Studs were spaced 16 in. on center. All framing consisted of No. 2 grade spruce-pine-fir purchased from a local lumber yard. Members were arbitrarily chosen when placed in the wall specimens.

Exterior sheathing was 15/32 in., 4 ply, structural I plywood. All full height panels were 4 ft. by 8 ft. and oriented vertically. To accommodate openings, the plywood was cut to fit above and below the doors and windows.

Interior sheathing was 4 ft. by 8 ft. sheets of 1/2 in. gypsum wallboard, oriented vertically. As with plywood, the gypsum was cut to fit above and below the doors and windows. All joints in the interior sheathing were taped and covered with drywall compound. Taped joints were allowed to dry for a minimum of 3 days prior to testing.

Table 2: Wall materials and construction data

Component	Fabrication and Materials
Framing Members	No. 2, Spruce-Pine-Fir, 2 x 4 inch nominal
Sheathing:	
Exterior	Plywood, 15/32 in., 4 ply, Structural I. 4 ft. x 8 ft. sheets installed vertically.
Interior	Gypsum wallboard, 1/2 in., installed vertically, joints taped
Headers:	
4'-0" opening	(2) 2x4's with intermediate layer of 15/32 in. plywood. One jack stud at each end.
7'-10$^{1}/_{2}$" opening	(2) 2x8's with intermediate layer of 15/32 in. plywood. Two jack studs at each end.
11' - 10$^{1}/_{2}$" opening	(2) 2x12's with intermediate layer of 15/32 in. plywood. Two jack studs at each end.
Tie-down	Simpson HTT 22, nailed to end studs with 32 16d sinker nails. 5/8 in. diameter A307 bolt to connect to foundation.
Anchor Bolts	5/8 in. diameter A307 bolt with 3 in. square x 1/4 in. steel plate washers.

Both exterior and interior sheathing were able to rotate past the test fixture at the top and bottom (i.e. the steel test fixture was narrower than the wood framing used for top and bottom plates.)

Two tie-down anchors were used on each wall, one at each double stud at the ends of the wall specimens (approximately 40 feet apart.) Simpson Tie-down model HTT22 were used. Tie-down anchors were attached to the bottom of the end studs by thirty-two (32) 16d (0.148 in. diameter and 3.25 in. length) sinker nails. A 5/8 in. diameter bolt connected the tie-down, through the bottom plate, to the rigid structural steel tube test fixture.

Table 3 shows the fastener schedule used in constructing the wall specimens. Four different types of nails were used. 16d (0.162 in. diameter and 3.5 in. length) bright common nails connected the framing, 8d (0.131 in. diameter and 2.5 in. length) bright common nails attached the plywood sheathing to the frame, 16d (0.148 in. diameter and 3.25 in. length) sinker nails attached tie-down anchors to the end studs, and 13 gage x 1-1/2 in. drywall nails attached gypsum wallboard to the frame. A nail spacing of 6 in. perimeter and 12 in. field was used for the plywood sheathing and 7 in. perimeter and 10 in. field for the gypsum wallboard. Tie-down anchors were attached to the double end studs using the 16d sinker nails, one located in each of the 32 pre-punched holes in the metal anchor.

Table 3: Fastener schedule

Connection Description	No. and Type of Connector	Connector Spacing
Framing		
Top Plate to Top Plate (Face-nailed)	16d common	per foot
Top / Bottom Plate to Stud (End-nailed)	2-16d common	per stud
Stud to Stud (Face-nailed)	2-16d common	24 in. o.c.
Stud to Header (Toe-nailed)	2-16d common	per stud
Header to Header (Face-nailed)	16d common	16 in. o.c. along edges
Tie-down Anchor/ Anchor Bolts		
Tie-down Anchor to Stud (Face-nailed)	32-16d sinker	per tie-down
Tie-down Anchor to Foundation	1-A307 5/8 in. dia. bolt	per tie-down
Anchor bolts	1-A307 5/8 in. dia. bolt	24 in. o.c. and within 1 ft. of wall ends, using 3 x 3 x ¼ in. steel plate washers
Sheathing:		
Plywood	8d	6 in. edge / 12 in. field (2 rows for end stud)
Gypsum wall board	13 ga x 1½ in. (3/8 in. head)	7 in. edge / 10 in. field

Wall Orientation and Attachment to Test Frame

Tests were performed with the shear walls in a horizontal position as shown in Figures 2 - 3. The wall was raised approximately 16 inches above the ground to allow sufficient clearance for instruments and the load cell to be attached to the wall. The bottom plate was secured to a fixed steel structural tube at 24 in. on center. Oversize of bolt holes was limited to 1/32 in. to minimize slip.

Bolts attaching the bottom plate were located a minimum of 12 inches away from the studs adjacent to openings or end of wall. This resulted in the bottom plate lifting when the stud next to an opening was in tension. In turn, the nails attaching the sheathing to the bottom plate had to transfer this tension, and at large displacements were damaged significantly more than nails near tie-down anchors.

Test Equipment and Instrumentation

A hydraulic actuator, with a range of ±6 inches and capacity of 55,000 lbs, was attached to the top left corner of each shear wall (for the configurations shown in Table 1) via a steel tube that was used to distribute the loading to the wall double top plate. The steel tube and the double top plate were attached using 5/8 in. diameter bolts 24 in. on center, beginning one foot from the end of the wall. Eight casters were attached to the structural tube to allow horizontal motion, as shown in Figure 2(b). The casters were fixed parallel to loading. A test was conducted to determine the amount of friction created by the wheels. Even though the magnitude of the friction was negligible (0.5 - 2%) when compared to the capacity of the walls, all recorded loads were corrected for this bias.

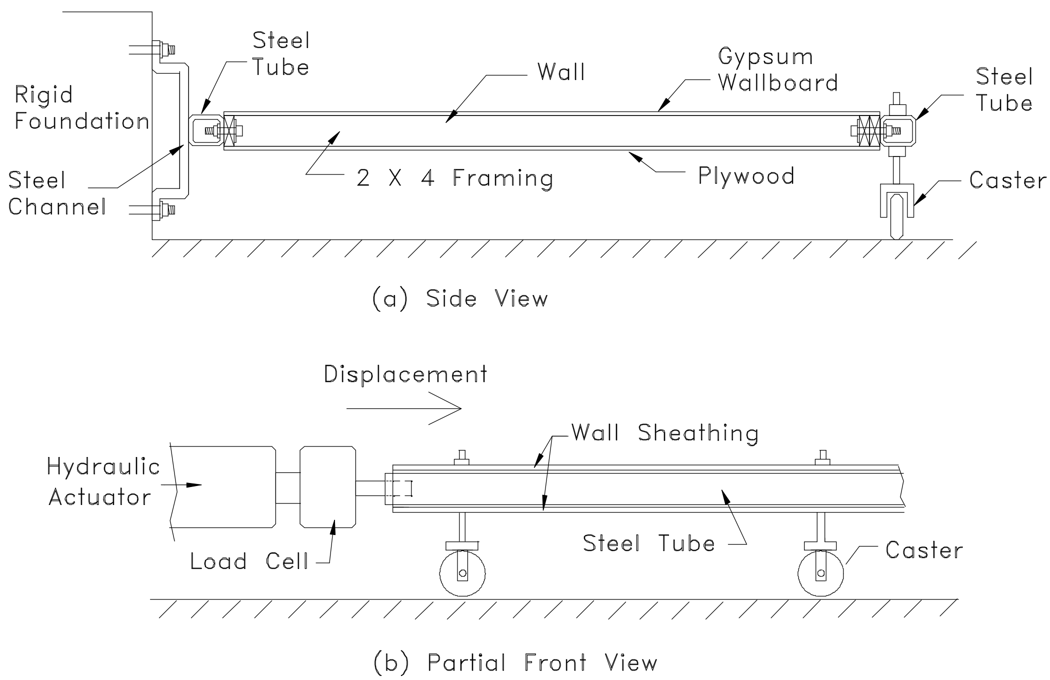


Figure 2: Wall orientation

Figure 3 shows the location of the six LVDT's that were attached to the frame of each wall to measure wall displacements.

LVDT #1 was located adjacent to where the load was applied, measuring the displacement of the top of the wall relative to a fixed reference point.

LVDT #2 and LVDT #3 measured the compression and uplift displacement of the end studs relative to the foundation. These sensors determined the amount of crushing in the sill plate, or uplift of the end stud, depending on which corner of the wall was in compression or tension, respectively. All data recorded was corrected to compensate for amplifications caused by the geometry of the LVDT fixtures. This ensured the actual compression and uplift displacements of the end studs were measured.

LVDT #4 measured horizontal displacement of the bottom plate relative to a fixed point. This measurement allows rigid body translation of the wall to be subtracted from the global displacement to obtain interstory drift. Interstory drift is calculated as LVDT #1 - LVDT #4.

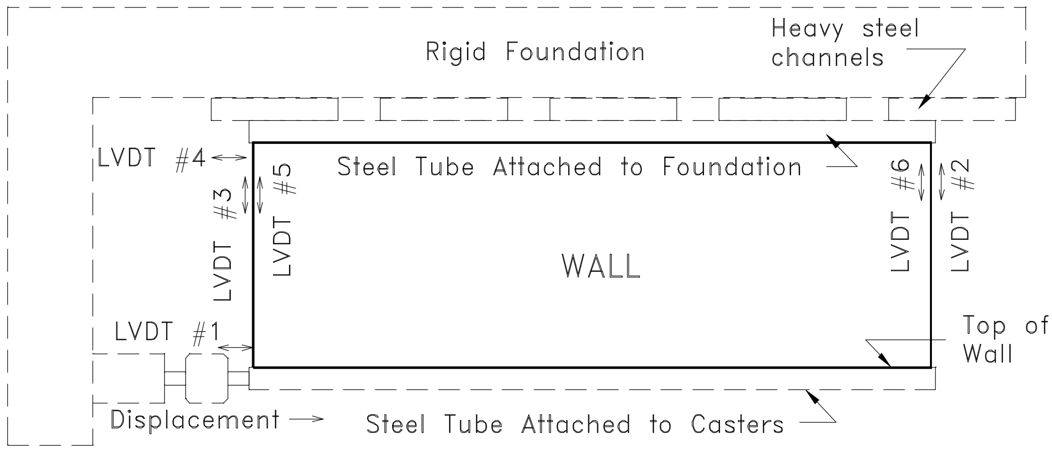


Figure 3: Sensor locations on plan view of wall specimen

LVDT #5 and LVDT #6 were attached to the end studs and tie-down anchors. These sensors measured the slip, if any, of the tie-down anchors relative to the end stud.

The hydraulic actuator contained two internal sensors recording load resisted by the wall and relative displacement of the load cell.

Loading

Monotonic tests were one-directional, displacing the top of the wall six inches over a ten minute period. Data from the 6 LVDT's and load cell were collected 10 times per second. Each of the five wall configurations was tested once.

PROPERTY DEFINITIONS

Maximum capacity, F_{max}, as well as its corresponding displacement, Δ_{max}, were determined from the load-displacement curves of each wall. Capacity at failure, $F_{failure}$, was determined as the load carried by the wall just prior to a sudden significant decrease in strength. Corresponding displacement, $\Delta_{failure}$, is the maximum interstory drift that the wall can sustain and continue resisting significant (i.e. displacement where catastrophic failure occurred) load. F_{max}, $\Delta_{max,}$ $F_{failure}$, and $\Delta_{failure}$, are illustrated in Figure 4.

An equivalent energy elastic-plastic curve (Figure 4), used for comparing monotonic to cyclic results, was determined for each wall. This artificial curve depicts how an ideal perfectly elastic - plastic wall would perform and dissipate an equivalent amount of energy. The displacement at yield, Δ_{yield}, is defined as the displacement where the elastic and plastic lines of this curve intersect. The elastic portion has a slope equal to the elastic stiffness, which is taken as the secant stiffness at 40% of capacity. The horizontal line is located so that the area under the equivalent elastic-plastic curve and load-displacement curve are equal. This definition is also used in the cyclic tests, and is similar to that used in the sequential phased displacement test developed by the Joint Technical Coordinating Committee on Masonry Research (TCCMAR) for the United States - Japan Coordinated Earthquake Research Program and defined by Porter (1987).

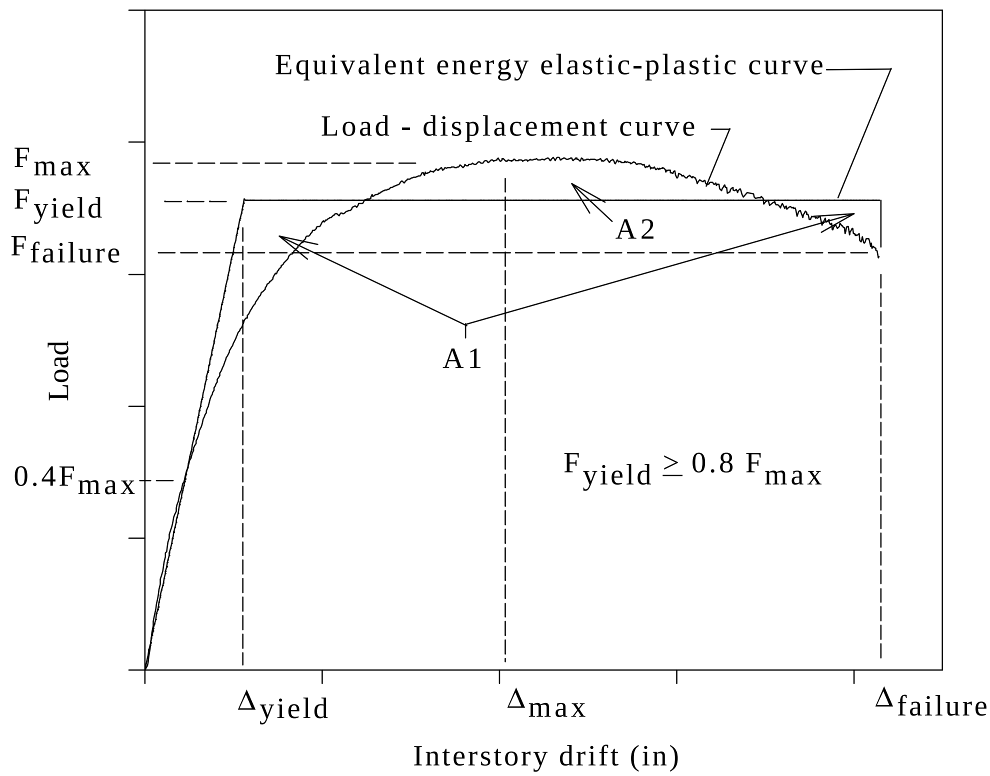


Figure 4: Typical load-displacement and equivalent energy elastic-plastic curve

Ductility is determined using the equivalent energy elastic - plastic curve, and is defined as:

$$D = \frac{\Delta_{failure}}{\Delta_{yield}} \tag{5}$$

where $\Delta_{failure}$ and Δ_{yield} are defined in Figure 4.

It should be noted that alternative definitions have been proposed by organizations such as the Structural Engineers Association of Southern California, and some proposals do not promote the use of ductility ratio. However, it is an indication of how structural systems fail (brittle or ductile), and is therefore a useful parameter to be considered when determining how to convert test results to design values.

Actual shear capacity ratio is defined as load resistance of a wall divided by the load resistance of the fully sheathed wall at the same interstory drift (or at capacity).

MONOTONIC TEST RESULTS

Load-deflection curves for the five walls are shown in Figure 5 and Table 4 contains load resistance at 0.32 in. interstory drift, 0.96 in. interstory drift, 1.6 in. interstory drift, and capacity. Actual and predicted shear capacity ratios are also presented in Table 4. Predicted shear capacity ratios, F, are determined from Equation (1).

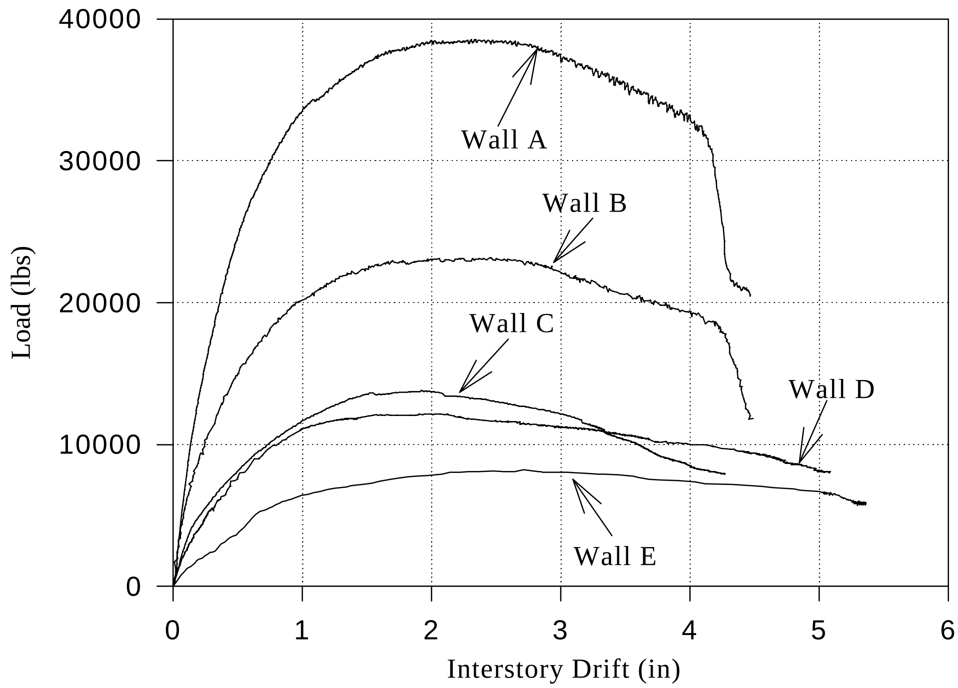


Figure 5: Load vs. interstory drift curves

Table 4: Load-displacement data

	Wall Specimen				
	A	B	C	D	E
Predicted shear capacity ratio, (F)	1	0.51	0.29	0.24	0.13
Displacement (in) @ Maximum Load	2.0	2.2	1.9	1.6	2.7
Maximum Load (kips)	38.8	23.1	13.8	12.1	8.2
Actual shear capacity ratio, (F)	1.0	0.60	0.36	0.31	0.21
Actual / Predicted shear capacity ratio	1.0	1.17	1.22	1.32	1.68
Load (kips) @ 1.6 in. interstory drift	37.7	22.6	13.5	12.1	7.4
Actual shear capacity ratio, (F)	1.0	0.60	0.36	0.32	0.20
Actual / Predicted shear capacity ratio	1.0	1.18	1.24	1.33	1.54
Load (kips) @ 0.96 in. interstory drift	33.6	20.0	11.4	11.0	6.3
Actual shear capacity ratio, (F)	1.0	0.60	0.34	0.33	0.19
Actual / Predicted shear capacity ratio	1.0	1.18	1.17	1.38	1.46
Load (kips) @ 0.32 in. interstory drift	18.5	11.4	6.3	5.5	2.5
Actual shear capacity ratio, (F)	1.0	0.62	0.34	0.30	0.14
Actual / Predicted shear capacity ratio	1.0	1.22	1.17	1.25	1.08

Strength

As shown in Table 4, wall capacity ranged from 8.2 to 38.8 kips, with Wall A having the highest capacity and Wall E having the lowest capacity. Interstory drift corresponding to the capacity ranged from 1.6 in. to 2.7 in. Wall E, which contained large openings and had the lowest stiffness, reached capacity at an interstory drift of 2.7 in.

The ratio of actual to predicted shear capacity ratio, F, is presented in Table 4, where a ratio greater than 1.0 indicates a conservative prediction by Equation (1). Table 4 shows that at interstory drifts of 0.32 in., 0.96 in., 1.6 in. and at capacity, Equation (1) was conservative. The ratio of actual to predicted strength of walls with openings is 1.17 to 1.68 at capacity, 1.18 to 1.54 at 1.6 in. drift, 1.18 to 1.46 at 0.96 in. drift, and 1.08 to 1.25 at 0.32 in. drift. With the exception of 0.32 in. interstory drift, as the sheathing area ratio decreases (i.e. amount of openings increase), the more conservative predictions become.

In Figure 6, actual monotonic capacities and shear capacity ratios are presented with predicted capacity and shear capacity ratio, F, determined from Equation (1). As shown in Figure 6, Equation (1) conservatively estimates monotonic capacity for all wall configurations tested in this investigation.

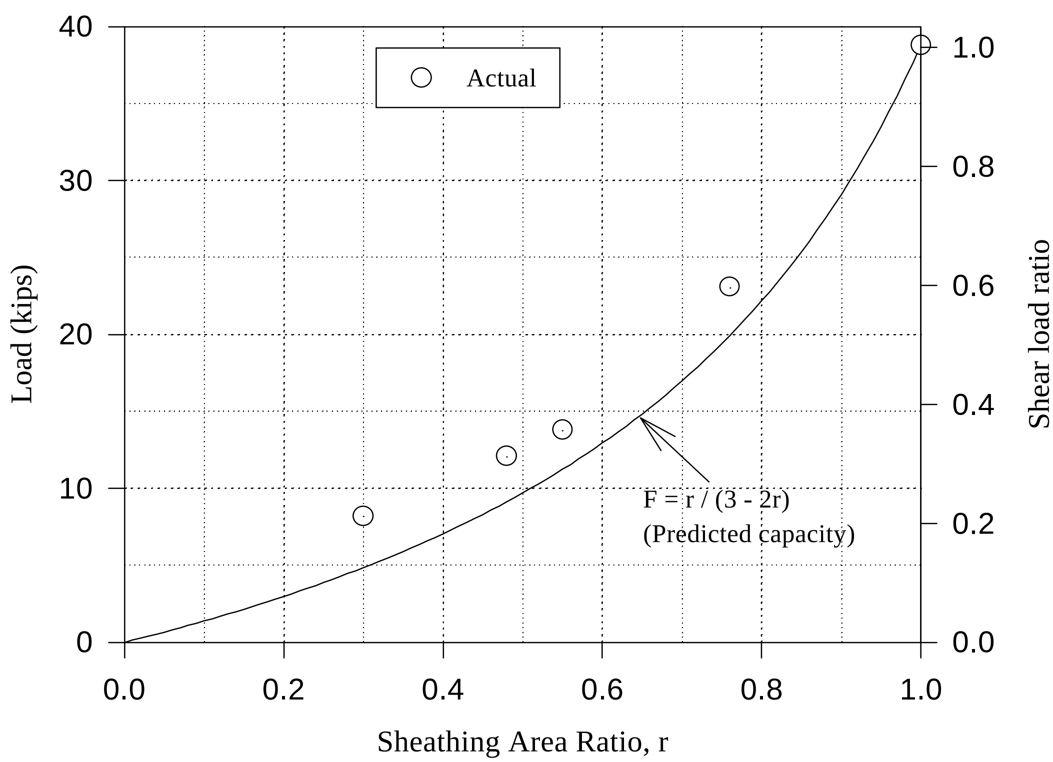


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Figure 6: Actual and predicted capacity and shear capacity ratios as function of sheathing area ratio, r

Yield Load

The yield load values determined for each wall specimen are shown in Table 5. As expected, the trend in these values follows the trend of the capacities with Wall A having the highest yield load, 35.6 kips and Wall E having the lowest yield load, 7.5 kips.

Table 5: Equivalent elastic-plastic curve parameters

	Wall Specimen				
	A	B	C	D	E
F_{yield} (kips)	35.6	20.9	11.8	10.6	7.5
Elastic Stiffness (kips/ in)	63.6	43.5	21.5	18.6	7.7
Δ_{yield} (in)	0.56	0.48	0.55	0.57	0.98
$\Delta_{failure}$ (in)	4.14	4.18	4.00	5.04	5.05
Ductility	7.4	8.7	7.3	8.8	5.2

Stiffness

Elastic stiffness is given in Table 5 and ranged from 7.8 kips/in to 63.7 kips/in. As expected, the more openings present in the wall, the lower the stiffness becomes.

Stiffness can be estimated using Equation (1). Equation (1) underestimates the stiffness by the same amount as strength. Equation (1) provides conservative estimates of stiffness when used to calculate estimated drifts.

Ductility

Table 5 shows the ductility for each wall configuration. Ductility of the five wall configurations ranged between 5.2 and 8.8. Walls A, B and C had similar $\Delta_{failure}$ (4.00 in. - 4.18 in.) and Walls D and E had similar $\Delta_{failure}$ (5.04 in. - 5.05 in.). Drift at yield, for Walls A, B, C and D, ranged from 0.48 in. to 0.57 in.. Drift at yield for Wall E was 0.98 in., which is 72% higher than the next highest Δ_{yield}. Due to the relatively high Δ_{yield} of Wall E, ductility of this wall was lower than the other four. Ductility of Walls A - D ranged from 7.3 to 8.8.

End Stud

Measured displacements of the end studs and tie-downs are shown in Table 6. Due to computer error, the data pertaining to the end stud behavior and the slip of the tie-downs for Wall E was not recorded correctly. However, the other four walls provided quantitative information, and the deflection pattern observed during the test of Wall E was similar. Therefore, the behavior of the studs and anchor slip for this wall should be similar to the rest of the specimens.

Separation of the tension side from the bottom plate ranged from 0.08 in. to 0.18 in. and movement of the compression end studs relative to the bottom plate ranged from 0.06 in. to 0.13 in. Movement of the compression end studs includes closure of gaps as well as crushing of the bottom plate. No significant crushing of the bottom plate was observed. Tension and compression end stud displacements are shown in Table 6.

Table 6: End stud displacement and tie-down slip at wall capacity

LVDT Number and Location	Wall Specimen			
	A	B	C	D
No. 2 -Compression End Stud (in)	0.13	0.06	0.09	0.09
No. 3 - Tension End Stud (in)	0.16	0.08	0.18	0.09
No. 5 - Tie-down Slip, (in) Tension End Stud	0.05	0.04	0.02	0.02
No. 6 - Tie-down Slip, (in) Compression End Stud	0.00	0.01	0.00	0.00

<u>Tie-down Anchors</u>

As shown in Table 6, the maximum slip relative to the stud for anchors in tension, recorded near peak load, was less than 0.05 in., which is negligible.

For Wall C, an instrumented bolt that was calibrated for tension loads was used to connect the tie-down anchor to the foundation instead of the standard 5/8-inch bolt used in the other specimens. Figure 7 shows the measured tension load resisting overturning moment with respect to the interstory drift (racking displacement). Figure 7 also shows lateral load applied to the wall with respect to drift. As can be seen, the peak load in the tie-down was measured at higher dirfts than where the peak racking load for the wall occurred. This is most likely due to the high bending forces placed on the bolt after the wall reached the capacity. However, at the drift associated with the capacity of the wall, the tension force in the tie-down was 8,830 lbs. Due to the non-uniform distribution of load along the full shear wall specimen, the assumtion of overturning force being equal to the unit shear times the wall height will not provide a good estimation of the expected load for walls with openings. Therefore, it is recommended that the shear load distribution of shear walls be investigated further.

<u>Gypsum Wallboard Sheathing</u>

Little or no apparent damage to the gypsum wallboard sheathing was observed at interstory drifts less than 0.50 in. As interstory drift progressed to approximately 1 inch, cracking of tape joints around openings and failure of tape joints between full panels occurred. At large displacements, perimeter nails began to pull through the panel or tear out panel edges. At larger deformations, perimeter nails tore through the edge of the panel or pulled through the panel. Field nails experienced some head pull through and experienced some lateral displacement creating a slot indicating the path of the nail.

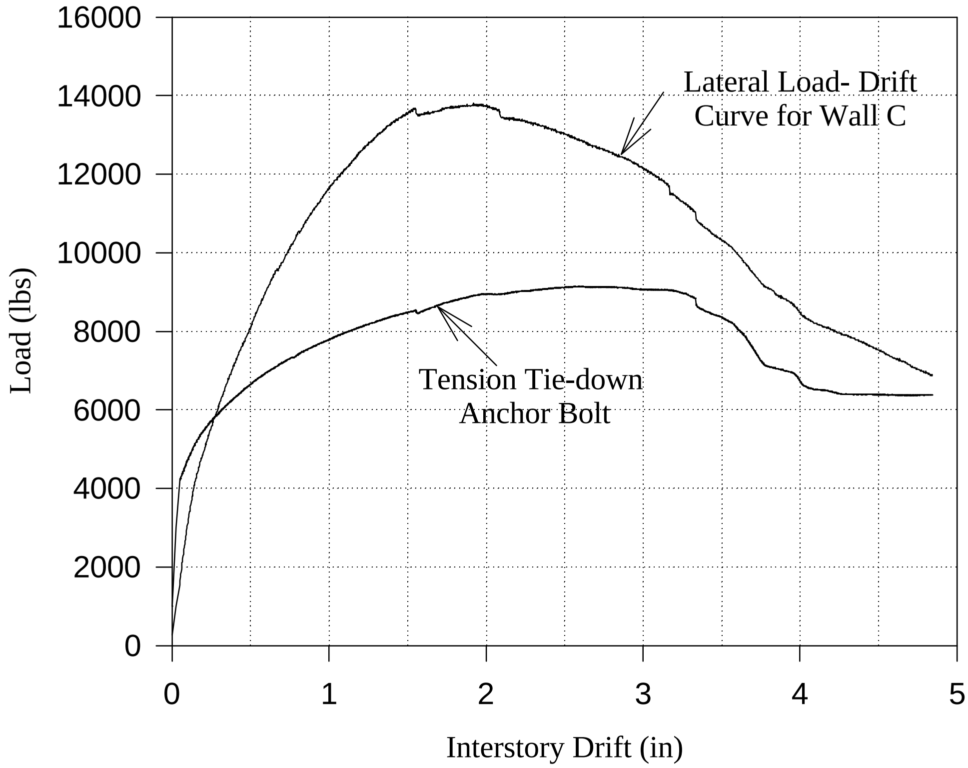

Figure 7: Tension Force in Tie-Down Anchor and Lateral Load Applied to Wall With Respect to Interstory Drift. (Wall C only)

Plywood Sheathing

Racking of plywood panels was observed for full-height panels, while plywood above and below openings acted as a rigid body. Panels below openings experienced some racking. At large displacements, failure of the nails around the perimeter of the sheathing panels consisted of head pull through or combined bending and withdrawal. After peak capacity, nails usually tore through the edges of the plywood.

General Observations

Failure modes for wall(s) with and without openings had many similarities. Typical failures at large displacements (significantly larger displacements than at capacity) for all walls often included buckling of the plywood panel adjacent to the load cell, and nail head pull-through at the bottom edge of the plywood panel, and nail tear through or head pull through of the gypsum wallboard panel edges. Walls with and without openings also had some similar performance characteristics including relatively elastic performance until an interstory drift of approximately 0.50 in. and the ability to support relatively high loads at displacements well beyond maximum capacity. Reduced strength and stiffness performance of shear walls with openings based on an equivalent length of full-height sheathing can be attributed to lack of uplift restraint provided adjacent to openings.

Unlike Wall A which is fully restrained against overturning and represents a traditionally engineered shear wall, the strength and stiffness performance of Walls B-E results from the combined shear resistance offered by full-height sheathed sections that are either fully or partially restrained against overturning. Full-height sheathed sections adjacent to restrained tension end studs are considered to be fully restrained against overturning. For Walls B-E, fabricated in accordance with the Perforated Shear Wall Method, the tie-down on the tension end stud provides the necessary overturning restraint. Other full-height sheathed sections are not fully restrained against overturning because the tension stud is not properly anchored to provide full overturning restraint. However, partial restraint is provided by sheathing nail attachment to the bottom plate and in some cases by partial panels located above and below openings. Shear capacity of the partially sheathed sections is significantly reduced from the fully restrained section and results in an overall reduction in strength and stiffness performance when based on an equivalent length of restrained full-height sheathing.

These tests were performed without an applied dead load in order to test the most conservative condition. If dead load had been present, the studs next to the openings that had no overturning restraint (i.e., no tie-down connectors) would not have lifted from the test frame as much. This would have reduced the damage to the nails attaching the sheathing to the bottom plate in these regions. The result would have been an improved overall performance. This is especially clear when one considers that studs next to openings have the highest axial load due to applied dead load.

CONCLUSIONS

The monotonic strength and stiffness performance of long shear walls with openings of varying dimension are covered in this report. Their performance is consistent with predictions using Sugiyama's empirical equation for shear capacity ratio that forms the basis of the adjustment factors used in the perforated shear wall design method that appears in the *Standard Building Code* and the *Wood Frame Construction Manual for One- and Two- Family Dwellings*. Results indicate that strength predictions are conservative throughout the range of interstory drifts up to capacity.

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