The racking performance of shear walls with various aspect ratios. Part 1. Monotonic tests of fully anchored walls

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Abstract

Current design values for light-frame timber shear walls are based on results of standard monotonic tests of 2.4-m (8-ft.) square walls restrained against overturning. The shear resistance of walls is calculated in terms of load per unit length, assuming that shear forces are distributed uniformly throughout the wall of any size. In past earthquakes, structural failures occurred near large openings where the lateral forces were transmitted through narrow wall segments. To improve our understanding of shear wall racking performance and to facilitate further development of seismic design methodology, a comprehensive study has been conducted that combines experimental and numerical analyses of shear walls of various configurations. This study included static monotonic and cyclic tests of full-size shear walls with height-to-length ratios of 4:1, 2:1, 1:1, and 2:3. Discussed in this paper is the static monotonic response of shear walls with overturning restraint representative of segmented wall construction practices. These walls were attached to the foundation by means of hold-down anchors and shear bolts. Test results revealed that the performance of segmented walls did not depend on the aspect ratio with the exception of narrow (4:1) walls, which exhibited 50 percent lower stiffness per unit length relative to the other walls tested. Differences in failure patterns and reduction in deformation capacity were observed when low-density studs were used at the wall ends. Traditional methods of analyzing segmented shear walls with hold-down anchors were shown to be sufficiently conservative.

Walls as components of the lateral force resisting system of a building are defined as *shear walls*. When resisting wind or an earthquake, shear walls act as vertical cantilevers transferring the lateral forces from the upper parts of the building to the foundation. In light-frame buildings, shear walls typically consist of lumber framing and exterior and interior panel sheathing attached with dowel-type fasteners (nails, staples, or screws). Exterior walls often represent a combination of fully-sheathed segments of various lengths interrupted by windows and doors of various sizes. In shear wall analysis, the vertical framing members at the ends of the wall segment are called *chords* and the horizontal framing members are called *struts*. Struts collect the horizontal forces from the upper parts of the building through framing fasteners and transfer the load to the sheathing through the sheathing fasteners. Sheathing panels provide racking resistance, and transfer the load to the chords through the sheathing fasteners. Chords resist the overturning moment created by the shear forces. *Segmented wall* design practice usually requires that the chords be attached to the lower structures (foundation or story below) through *hold-down anchors* and *anchor bolts* to restrain shear walls from overturning. Bolts attaching the struts to the upper and

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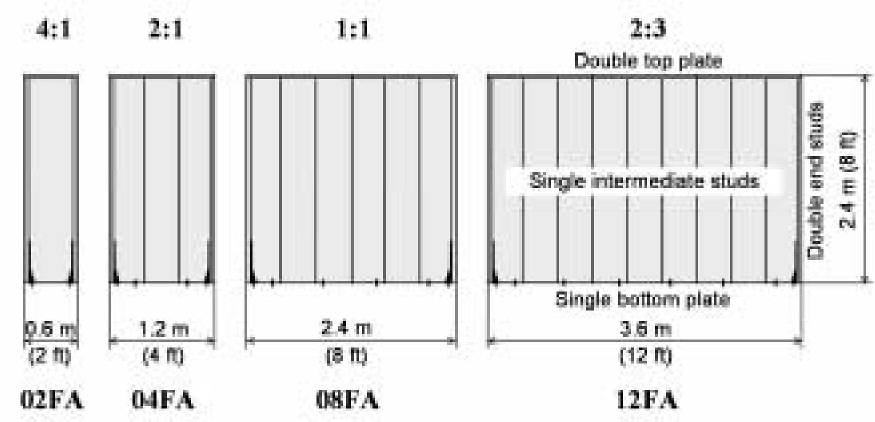

Figure 1. — Geometry and notation of wall specimens.

lower structures to prevent wall sliding are called *shear bolts*. As opposed to the segmented wall construction, conventionally built walls are secured to underlying structures by nails or shear bolts only; therefore, the overturning is resisted only by the weight of the building and the sheathing nails along the bottom of the wall.

The height-to-length ratio of a shear wall is called the *aspect ratio*. Shear wall design traditionally uses the shear resistance calculated in terms of load per unit length without regard to the aspect ratio, assuming that the lateral force is distributed uniformly throughout the total length of all shear panels without openings (Breyer et al. 1999). However, there is an opinion that the shear distribution is not uniform (Diekmann 1997). Current design values for shear walls were proposed by the American Plywood Association (APA) based on static monotonic tests on 2.4-m (8-ft.) square walls fully restrained against overturning (Tissel 1990). Tests were performed using static *monotonic* (non-reversed) loading applied in several stages at a uniform rate of displacement according to ASTM standards E 72 (ASTM 1995a) and E 564 (ASTM 1995b).

Very few investigations in shear wall performance have focused on aspect ratio effects. White and Dolan (1994) investigated the effects of aspect ratio on shear wall response using numerical methods, and validated the model for shear walls with a 1:1 aspect ratio. Different materials, fabrication techniques, and test procedures used in most previous studies prohibit direct comparison of the results. Some research results supported the common assumption that the stiffness and strength of long walls were in linear proportion to their length (Patton-Mallory et al. 1984), while other observations did not confirm this (Wolfe 1983). Stiffness and strength of narrow walls were found greatly reduced, because any hold-down movement was magnified by the high aspect ratio (Commins and Gregg 1994). Evaluation of single-family residential buildings in the 1994 earthquakes in Northridge, California (Andreason and Rose 1994) revealed that damage occurred at the narrow wall segments near large openings such as wide windows and garage doors and along the top of the foundation. A thorough wall bracing, together with intensive fastening of sheathing to framing and framing to foundation, was recommended for narrow walls to develop required shear resistance (Tissel and Rose 1994).

This information suggests that wall aspect ratio, hold-down restraint, and quality of sheathing attachment determine the stiffness and strength of shear walls. In the transition to performance-based design philosophy, all these factors may be important to consider when establishing multiple performance levels for the structures. In light of the ongoing discussions, it is necessary to investigate strength and stiffness of shear walls in a wide range of aspect ratios using uniform manufacturing and testing procedures. Information obtained in such a study can improve our understanding of the racking performance of shear walls and will contribute to further development of seismic design methodology.

This study is part of a comprehensive research program conducted at Virginia Tech that combines experimental and numerical analyses of shear walls of various configurations, including segmented and conventional walls (Salenikovich 2000). Presented in Part 1 of this study are results of monotonic tests on shear walls with hold-down anchorage representative of segmented wall design practices. The objective was to analyze the effects of aspect ratio on shear wall strength and deformation characteristics under static monotonic load. Observations of failure patterns and measurements of sheathing and framing displacements and forces in anchor bolts were conducted to describe the shear wall racking performance. Cyclic test results are presented in Part 2 of the study (Salenikovich and Dolan 2003). The tests on conventional shear walls will be evaluated in subsequent articles.

Experimental

Specimens

All shear wall specimens were 2.4-m (8 ft.) tall. Four aspect ratios were considered (**Fig. 1**):

- 4:1 – the size often accommodating garage doors and wide "view" windows.
- 2:1 – the minimum width of the traditional wall allowed in U.S. model codes to resist high seismic loads. It is the typical width of fully sheathed wall segments filling the space between windows and doors in residential buildings.
- 1:1 – the ASTM standard specimen size serving as a reference point for comparison with other tests.
- 2:3 – the size for investigating if longer walls have the same performance characteristics as square walls.

Throughout the study, materials and framing techniques were kept constant. Acronymic notation of individual test specimens was used to indicate the wall length in feet (02, 04, 08, or 12), the type of overturning restraint (FA = full anchorage), the load regime (m = monotonic), and replication number (1, 2, etc.). ASTM E 564 standard practice requires testing a minimum of two shear wall specimens of a given configuration. A third specimen is tested if the difference between the strength or shear stiffness of the two specimens exceeds 15 percent.

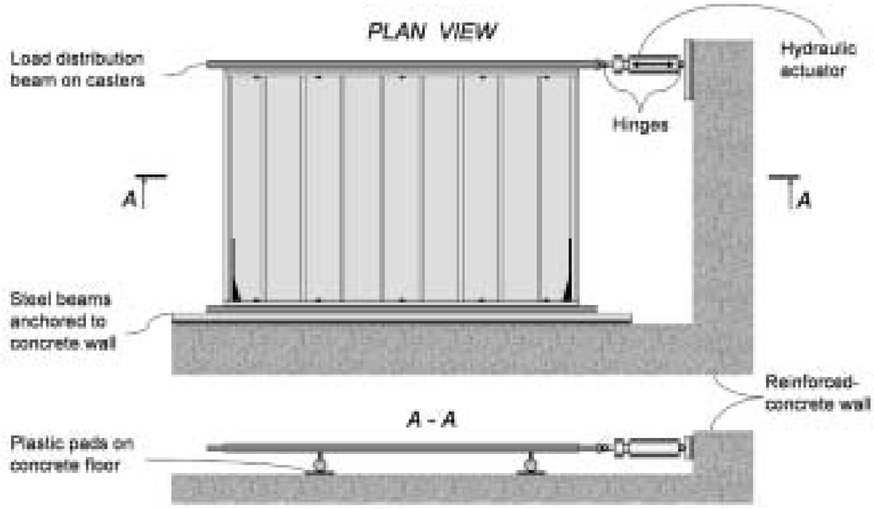

Figure 2. — Test setup.

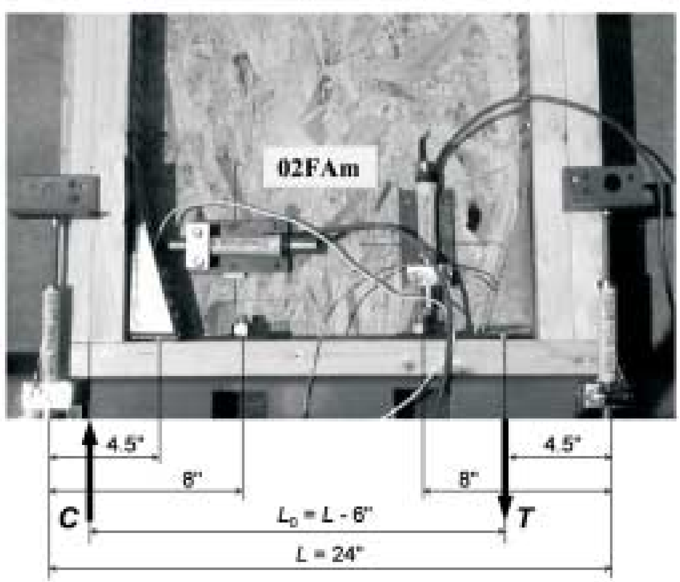

Figure 3. — Anchorage and effective length of 0.6-m (2-ft.) wall.

Prior to wall assembly, modulus of elasticity and density of each framing member were measured using a Metriguard® Model 340 transverse vibration E-computer (Metriguard 1994); moisture content was measured using a Delmhorst® Model J-3 moisture meter. The wall frame was assembled of 38- by 89-mm (2- by 4-in.-nominal) spruce-pine-fir (SPF) stud grade members spaced 0.41 m (16 in.) on centers, except for the walls with the aspect ratio of 4:1 with studs at the wall ends only (**Fig. 1**). The chords consisted of two studs fastened by two 16d (∅4.1- by 89-mm) common nails every 0.60 m (2 ft.). All studs were attached to the single bottom plate and the double top plate, with two 16d common nails at each end. A single layer of OSB sheathing, 11 mm (7/16 in.) thick, was attached to one wall side by power-driven 8d (∴3.3- by 63.5-mm) common SENCO® nails at 0.15 m (6 in.) on centers along the edges and 0.30 m (12 in.) on centers along intermediate studs. The long dimension of the sheathing was oriented parallel to the studs and fastened to the framing with a 19-mm (3/4-in.) edge distance along the top and bottom plates, and 10 mm (3/8 in.) along the vertical edges. As a means of overturning restraint, Simpson Strong-Tie® HTT22 connectors were attached on the inside of the chords by thirty-two 16d (∴3.8- by 82.6-mm) sinker nails.

Test setup

The specimens were stored in the laboratory ambient conditions for at least 2 weeks after fabrication to allow for wood relaxation around the nails. Each specimen was tested in a horizontal position (**Fig. 2**). No dead load was applied in the plane of the wall, which conservatively represented a wall parallel to floor joists. The wall was supported by two 76- by 127-mm (3- by 5-in.) steel beams attached to the wall top and bottom plates with ∴15.9-mm (5/8-in.) bolts spaced 0.6 m (24 in.) on centers. These bolts were secured by nuts with the use of 64- by 64-mm- (2.5- by 2.5-in.-) wide and 6-mm- (0.25-in.-) thick steel plate washers. In 0.6-m (2-ft.) walls, the distance between the shear bolts was reduced as shown in **Figure 3**.

To eliminate interference of the support with the sheathing displacements, the narrow face of the support beams was oriented toward the plates. To reduce the amount of the wall slip along the support during the test, the oversize of holes for the shear bolts was minimized. Holes in the supporting beams were only 0.8 mm (1/32 in.) larger than the bolt diameter; holes in the top and bottom plates were drilled without oversize. At the ends, the wall was anchored to the supporting beam by ∴15.9-mm (5/8-in.) bolts through the hold-down connectors. Anchor bolts were instrumented with strain gages to measure the tension forces transferred to the chords through the hold-down anchors. The bolts were tightened to approximately 18 kN (4 kips) of initial tension. The holes for the anchor bolts were oversized by 13 mm (1/2 in.) to minimize the base shear effects on the tension force measurements in the instrumented bolts.

The steel beam at the bottom plate was secured to the reinforced-concrete reaction wall. The steel beam at the top plate distributed the racking load from a programmable hydraulic actuator. The actuator, with a displacement range of ± 152 mm (6 in.) and a capacity of 245 kN (55 kips), was secured between the support and the distribution beam by means of the hinged connections shown in **Figure 2**. If these hinges were omitted, the separation of the wall framing

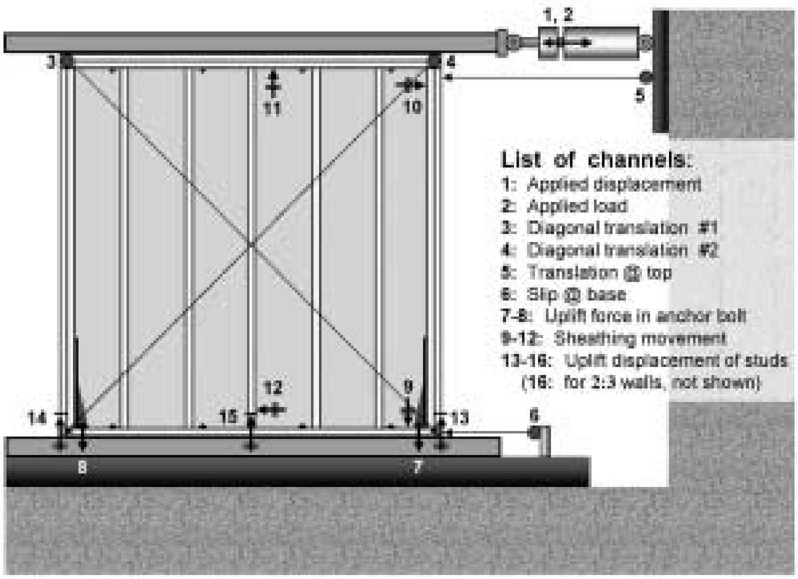

Figure 4. — Instrumentation of shear wall test specimen.

during the test would be restrained by the weight of the equipment, and the load measurement would be biased by the induced moment. Two casters attached to the load distribution beam allowed free movement of the specimen top, parallel to the direction of the applied load. The casters rolled along the greased surface of plastic pads laid on the concrete floor to reduce friction induced by the wall weight. The load was applied at a constant rate of 15 mm/min. (0.6 in./min.) in a single stroke. Each test was stopped when the specimen fully exhausted its ability to resist load.

Instrumentation and measurements

A data-acquisition system was equipped with 16 channels. The monitored responses are indicated in **Figure 4**. All data were recorded at a frequency 15 times per second. The hydraulic actuator contained the internal linear variable differential transducer, LVDT (channel 1), and the load cell (channel 2) that supplied information on the applied displacement and force that was used for the load-deflection analysis of the tests.

Resistance potentiometers (pots) 3 and 4 measured diagonal elongation of the wall between the top and bottom plates. The diagonal measurements were taken to obtain information on shear deformation of the wall assuming the specimen distorted as a parallelogram (ASTM E 564). This assumption was valid only until separation of the studs from top and/or bottom plates started; therefore, the diagonal measurements were of limited use. Pot 5 (attached to a rigid foundation) measured lateral translation of the top of the right end stud. The difference between readings of channels 1 and 5 illustrated the separation of the stud from the top plate and slip between the top plate and load distribution beam during the test. Pot 6 recorded the horizontal slip of the wall relative to the supporting steel beam. Channels 7 and 8 represented anchor bolts instrumented with strain gages to measure tension forces in the anchors.

LVDTs (channels 13 and 14) aligned with the wall edges were mounted on the supporting steel beam to measure uplift displacement of the end studs. Walls with aspect ratios of 1:1 and 2:3 accommodated additional LVDTs (15) and (15 and 16), respectively, to measure uplift of intermediate studs. These observations helped in estimating the wall rotation. To measure the displacement of sheathing relative to the frame, one sheathing panel in each wall accommodated four LVDTs (9 to 12) near the panel corners. The LVDT probes rested against polished steel plates attached to the studs to reduce friction when the probe moved along the stud. It was assumed that channels 9 and 11 measured only vertical translation components and channels 10 and 12 measured the horizontal components.

Load-deflection parameters

In traditional shear wall analysis (Breyer et al. 1999), the unit shears and overturning moments are estimated assuming that the distance between the chords equals the overall wall length (i.e., the total width of sheathing). The thickness of the end studs and the location of the anchor bolts are neglected. If the anchors are located inside the wall, this approach overestimates the actual distance between the vertical reactions, and, therefore, underestimates the forces acting in the wall (Commins and Gregg 1994). To eliminate this error in our analysis, the *effective wall length* (L_0) was introduced, measuring the distance between the vertical reaction forces as shown in **Figure 3**. It was assumed that the compression reaction went through the overturning point at the centerline of the compression chord, and the tensile reaction went through the center of the anchor bolt on the tension side of the wall. Since the anchor bolts were located inside the wall, L_0 was shorter than the overall length (L) by some 0.15 m (6 in.). Although for long walls this difference is insignificant, for narrow walls it is a considerable change, increasing the effective aspect ratios in 1.2-m (4-ft.) and 0.6-m (2-ft.) walls by 14 percent and 33 percent, respectively. To illustrate the racking resistance of shear walls with the effective aspect ratios, the racking loads (F) were normalized by the effective wall length, and the subsequent analysis was based on *unit shear load* (v) (kN/m (kip/ft.)):

$$v = F/L_0 \qquad [\mathbf{1}]$$

Similarly, tension forces in anchor bolts were predicted as follows:

$$T = F \cdot h/L_0 \qquad [\mathbf{2}]$$

where:

h = 2.44 m (96 in.), the height of shear wall

Usually, shear wall deflections are characterized by *story drift* after deducting slippage of the wall between the supporting structures. In this study, measurements on channel 6 showed that the slip displacements were negligible, less than 0.25 mm (0.01 in), assuming that the steel beam did not slip relative to the concrete wall. Therefore, the load-deflection curves and parameters, such as shown in **Figure 5**, were obtained using data from channels 1 and 2.

The maximum load point determined the wall *maximum shear strength* (v_{peak}) and *deflection capacity* ($\div_{peak}$). The *failure point* ($v_{failure}$, $\div_{failure}$) was considered to occur at 0.8 v_{peak} (i.e., when a 20% decrease of v_{peak} occurred). The area under the load-deflection curve

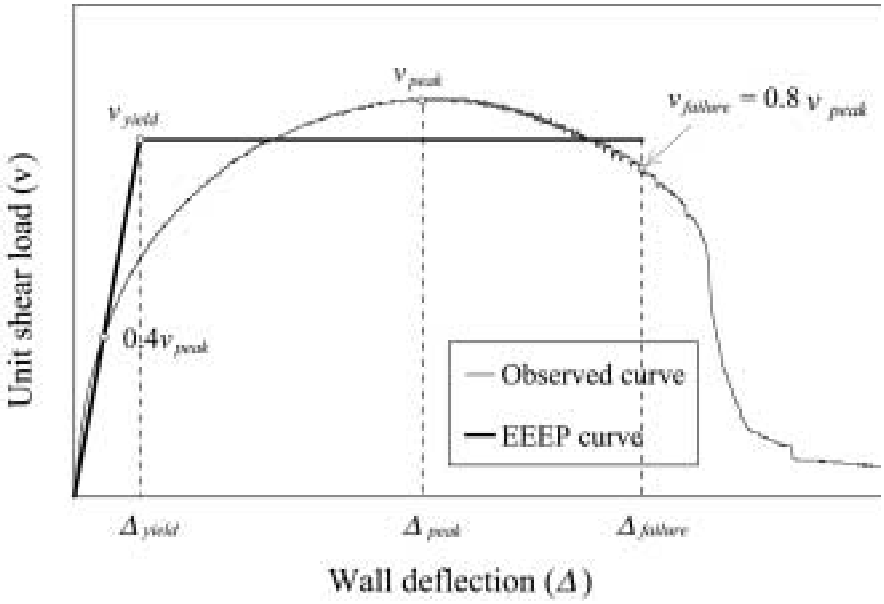

Figure 5. — Performance parameters of shear walls.

Table 1. — Performance parameters of fully anchored shear walls under monotonic load. [a]

	Units	12FAm1	12FAm2	08FAm1	08FAm2	04FAm1	04FAm2	02FAm1	02FAm2
v_{peak}	kN/m	10.33	9.33	10.82	10.40	9.76	10.11	10.17	9.51
	kip/ft.	0.708	0.639	0.741	0.713	0.669	0.693	0.697	0.652
$\div_{peak}$	mm	79.9	53.5	81.2	65.6	49.0	65.2	150.8	112.5
	in.	3.15	2.11	3.20	2.58	1.93	2.57	5.94	4.43
v_{yield}	kN/m	9.15	8.36	9.47	9.28	8.48	9.06	8.56	8.29
	kip/ft.	0.627	0.573	0.649	0.636	0.581	0.621	0.587	0.568
$\div_{yield}$	mm	13.7	12.5	14.2	12.4	10.3	13.2	20.8	24.6
	in.	0.54	0.49	0.56	0.49	0.41	0.52	0.82	0.97
$\div_{e}$	mm	6.2	5.6	6.5	5.6	4.8	5.9	9.5	11.1
	in.	0.24	0.22	0.25	0.22	0.19	0.23	0.38	0.44
$\div_{failure}$	mm	116.8	87.2	107.2	107.2	62.4	105.4	152.4[b]	152.4[b]
	in.	4.60	3.43	4.22	4.22	2.46	4.15	6.0	6.0
G	kN/mm	1.63	1.63	1.63	1.83	2.00	1.68	1.04	0.83
	kip/in.	9.32	9.31	9.30	10.43	11.42	9.58	5.94	4.76
$w_{failure}$	kN m/m	1.01	0.68	0.95	0.94	0.49	0.90	1.26[a]	1.19[a]
	kip ft./ft.	0.226	0.152	0.213	0.211	0.109	0.201	0.283	0.268

[a]See Figure 1 for shear wall notation.
[b]The test was stopped before failure was observed.

limited by the failure point approximated *unit work to failure* ($w_{failure}$) (i.e., the energy dissipated by the wall of unit length). Using these data, the *equivalent energy elastic-plastic* (EEEP) curve was derived as shown in **Figure 5**. The initial slope of the EEEP curve, drawn through 0.4 v_{peak} on the load-deflection curve, determined the unit *secant elastic stiffness* ($k_e = 0.4v_{peak}/\div_e$, where $\div_e$ = deflection at $0.4v_{peak}$).[1] To find the yield load (v_{yield}), the following equation was derived by equating the areas under the observed and EEEP curves:

$$v_{yield} = \left(\Delta_{failure} - \sqrt{(\Delta_{failure})^2 - \frac{2w_{failure}}{k_e}} \right) \cdot k_e \quad [3]$$

where the expression in parentheses determined the *yield deflection* ($\div_{yield}$)

[1]The value of $0.4v_{peak}$ is a value that is used in several national and international test standards for determining the initial stiffness of non-linear responses.

The bilinear EEEP curves depict how an ideal perfectly elastic-plastic wall would perform, dissipating an equivalent amount of energy, and allow comparison of the nonlinear performance of different walls on the equivalent energy basis. The *elastic shear modulus*, *G*, was obtained as follows:

$$G = k_e \cdot h \quad [4]$$

General observations

All walls exhibited a significant amount of racking. **Figure 6** shows a graph of sheathing displacements relative to the framing near the corners of the first panel in the 12FAm1 wall. Similar displacements were observed in 08FAm walls. At peak loads, the horizontal displacements reached 5 mm (0.2 in.) and vertical displacements reached 10 mm (0.4 in.). Bearing and friction forces between adjacent edges of sheathing panels might have provided for a relatively uniform distribution of the sheathing displacements relative to the framing in these walls (There was less than a 50% difference between corresponding displacements at the top and the bottom). In walls with one sheathing panel (04FAm and 02FAm), displacements at the top were less than a half of those at the bottom, and horizontal displacements were less than a half of vertical displacements. The displacements might be more uniform if the chords were anchored at the top as well as at the bottom.

Major performance characteristics of fully-anchored shear walls under static monotonic loads are shown in **Table 1**. Eight walls of different sizes developed the average v_{peak} of 10.1 kN/m (0.69 kip/ft.) with a remarkably low coefficient of variation (5%). These data indicate that narrow walls can develop the same unit shears as long walls if calculations are based on a proper effective length. Note however, if the loads were normalized by the overall wall length (width of shear panel), the shear forces in the narrow walls would be significantly underestimated.

Typical load-deflection curves of the tested walls are presented in **Figure 7**. These graphs do not include walls 04FAm1 and 12FAm2, which are discussed in the next section. Graphs 02FAm and 08FAm represent the average curves from two matching tests. Essentially, walls with aspect ratios Ω2:1

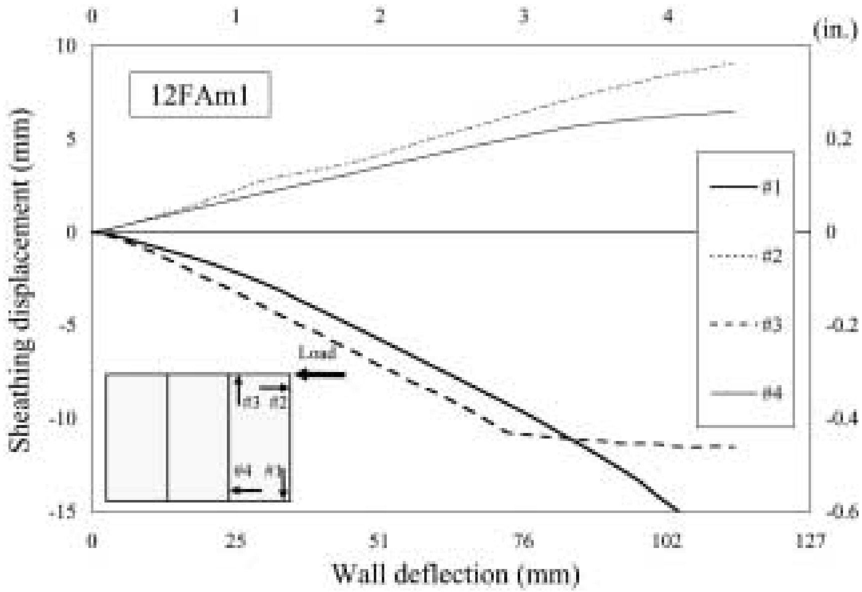

Figure 6. — Typical sheathing displacements in a long fully anchored wall.

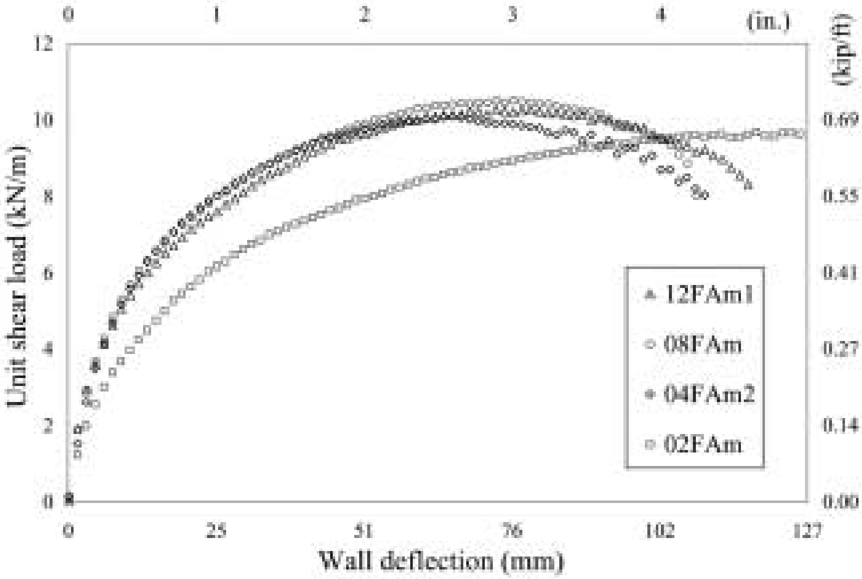

Figure 7. — Load-deflection curves of fully anchored walls during monotonic tests.

developed identical load-deformation patterns, reaching the maximum resistance at deflections beyond 64 mm (2.5 in.), and then gradually degrading. A 20 percent decrease in resistance occurred past 105-mm (4.1-in.) deflections, approximately seven times $\div_{yield}$.

Narrow (4:1) walls were approximately half as stiff relative to the longer walls, which explains their poor service record during earthquakes. At a deflection of 64 mm (2.5 in.), they resisted only 8.6 kN/m (0.59 kip/ft.), approximately 15 percent less than the other walls. Nevertheless, these narrow walls developed deflections exceeding 152 mm (6 in.), more than 10 times $\div_{yield}$, without noticeable strength degradation, because there was small displacement demand on sheathing-to-framing connections. The large drifts were due to a greater effect of rigid body rotation: the uplift displacements of the end studs contributed horizontal deflections in proportion to the aspect ratio.

Figure 8 shows typical vertical displacements of the end studs. Since the end studs were anchored to the foundation, uplift displacements did not exceed 5 mm (0.2 in.) at peak load. Analyzing the wall distortion pattern, it can be concluded that approximately two-thirds of the measured vertical displacement was caused by the stud rotation about the hold-down anchor; the nailslip and deformation of the hold-down anchor contributed the remaining one-third of the uplift. At typical failure, the right end stud separated from the top plate and the sheathing panels gradually unzipped along the top and/or bottom plates with the sheathing nails tearing through the panel edges. Usually, the sheathing unzipped along one of the studs and bottom plate with the nails pulling heads through the sheathing as shown in **Figure 6**.

Specific gravity effects

Walls 04FAm1 and 12FAm2 are discussed separately because their performance deviated from the other walls. The response curves are shown in comparison with the matching walls in **Figure 10**. Although the elastic stiffness and the load capacity of the matching walls differed less than 10 percent, there was at least a 35 percent reduction in deflections at the peak and failure loads. These walls dissipated approximately 40 percent less energy per unit length than the other walls. In other words, the ductility and toughness of these two walls were significantly reduced when compared to the matching walls.

Figure 11 illustrates the predominant failure mode of wall 04FAm1: the sheathing nails along the right end stud were pulled out. Similar failure was experienced by wall 12FAm2. Information on the specific gravity and location of each framing member recorded during wall manufacture suggested that these failure modes and the lower performance were associated with the lower density of the right end studs (**Table 2**). The right end studs in walls 04FAm1 and 12FAm2 had significantly lower specific gravity (0.42 and 0.37, respectively) than the rest of the walls. It is, therefore, rational that the entire row of nails along the low-density studs pulled out and started the early failure mechanism.

Figure 12 shows the sheathing displacements of wall 04FAm1. The vertical displacements at the bottom were three to four times larger than at the top. A similar graph of vertical displacements was recorded for wall 12FAm2. This information indicates that most of the work was done by the nails at the bottom plate. The lack of connection resistance along the end stud due to low wood density overloaded the nails at the

Table 2. — Specific gravity and strength parameters of shear walls under monotonic load.

	12FAm1	12FAm2	08FAm1	08FAm2	04FAm1	04FAm2	02FAm1	02FAm2
$SG_{average}$ [a]	0.49	0.46	0.53	0.53	0.46	0.46	0.47	0.47
$SG_{right\ end\ stud}$ [b]	0.53	0.37	0.56	0.52	0.42	0.49	0.44	0.50
F_{peak} (kips)[c]	8.14	7.35	5.56	5.34	2.34	2.43	1.04	0.98
$v_{peak}/v_{seismic}$	2.96	2.67	3.10	2.98	2.80	2.90	2.91	2.73
$F_{peak}/F_{seismic}$	2.84	2.56	2.90	2.79	2.45	2.53	2.18	2.04
T_{peak} (kips)	5.48	4.68	4.23	4.95	4.65	5.15	7.23	4.02

[a]Specific gravity of all framing members based on the ovendry volume.
[b]Specific gravity of the right end stud based on the ovendry volume.
[c]1 kip = 1000 lbf = 4.45 kN.

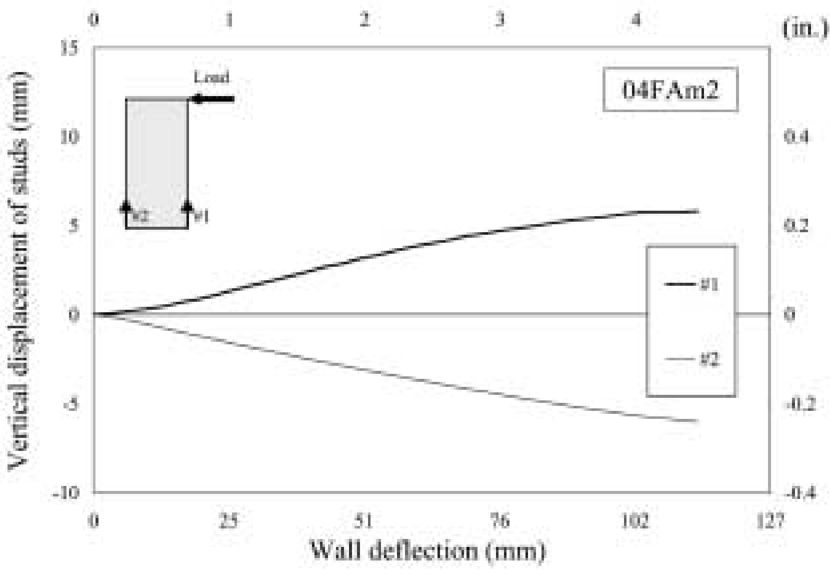

Figure 8. — Vertical displacements of studs.

Figure 9. — Typical view of the wall bottom after failure.

bottom plate and caused the early strength degradation of the entire shear wall.

Design implications

According to the International Building Code (ICC 2000), the allowable shear for walls of the tested configuration (11-mm [7/16-in.] vertical sheathing attached with 8d nails with 150-mm [6-in.] spacing at panel edges, and with stud spacing 410 mm [16-in.] on centers) with framing of Douglas-fir-larch or southern pine equals 3.79 kN/m (0.260 kip/ft.). Allowable shears are adjusted for specific gravity of the framing lumber using the *specific gravity adjustment factor* = [1 4 (0.5 4 *SG*)] Ω1, where *SG* is specific gravity for the species of lumber in the National Design Specification for Wood Construction (AF&PA 1997). For SPF species, SG = 0.42. Therefore, in seismic design, the allowable shear for the tested walls equals $v_{seismic}$ = 3.79 Δ 0.92 = 3.49 kN/m (0.239 kip/ft.). In wind design, a 40 percent increase of design capacities is permitted (ICC 2000): v_{wind} = 3.49 Δ 1.40 = 4.89 kN/m (0.335 kip/ft.). Corresponding design load capacities ($F_{seismic}$ and F_{wind}) can be estimated by multiplying the allowable shears and the overall wall length (L).

To compare the design values with test results, **Table 2** shows the ultimate load (F_{peak}) resisted by each shear wall tested and *strength ratios*: $v_{peak}/v_{seismic}$ *and* $F_{peak}/F_{seismic}$. The corresponding strength ratios for wind design (not shown in the table) are 40 percent lower. The comparison shows that design values for shear walls with the aspect ratios Ω 1:1 are sufficiently conservative. Strength ratios exceeded 2.8 for seismic design (2.0 for wind design) with the exception of wall 12FAm2, which had a very low-density right end stud. Walls with the aspect ratio 2:1 developed similar unit shears as the longer walls; however, their strength ratios based on F_{peak} were less conservative, because the design capacity included the overall length. To equalize the safety levels of narrow walls and long walls the load capacities should be calculated using the effective length (L_0). Alternatively, appropriate adjustment factors should be recommended for narrow (2:1) walls, depending on the anchorage conditions.

Although shear walls with the aspect ratio 4:1 are not permitted in engineered design (ICC 2000), **Table 2** shows that

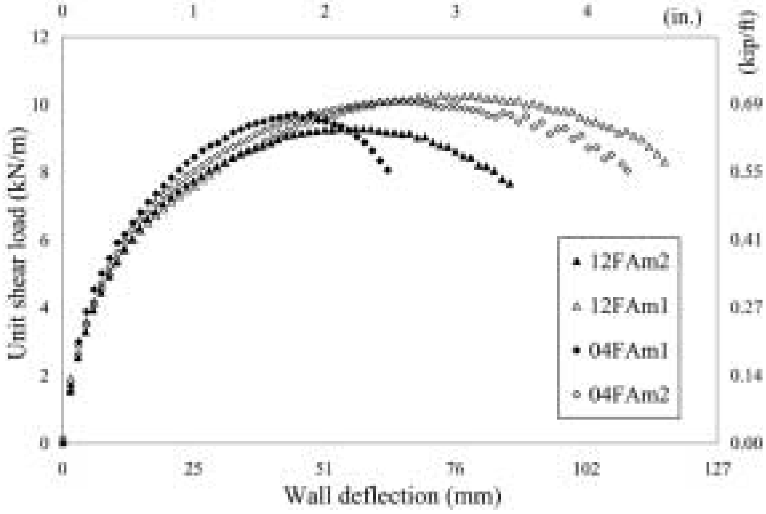

Figure 10. — Load-deflection curves of walls 12FAm2 and 04FAm1.

Figure 11. — Typical failure mode of walls with low-density end studs.

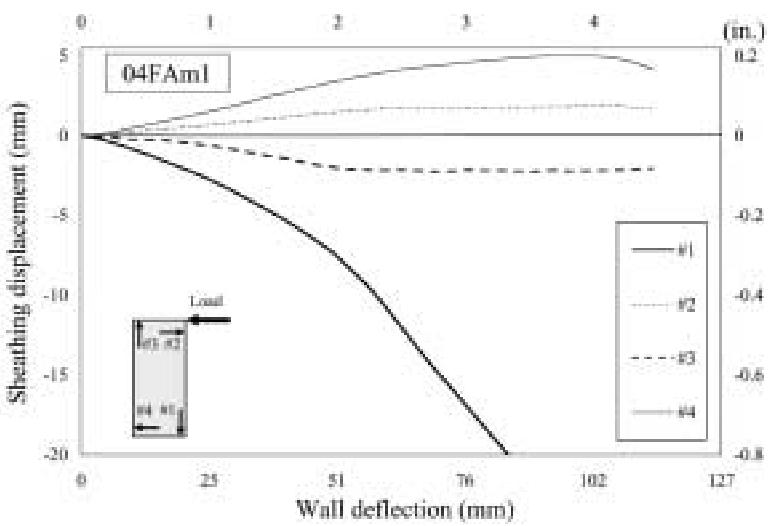

Figure 12. — Sheathing displacements of walls with low-density end studs.

their shear capacity could be predicted quite safely using current allowable values and the effective wall length. However, a reduction factor between 0.5 and 0.85 would be recommended to account for the lower stiffness of these walls as follows from previous discussion of **Figure 7**. The amount of reduction would depend on location and stiffness of hold-down anchorage.

Table 2 shows the ultimate forces (T_{peak}) measured in the anchor bolt on the tension side of each tested wall. On the average, T_{peak} measured 22.7 kN (5.05 kips), while the allowable tension load for the HTT22 hold-down anchor is 23.4 kN (5.26 kips) (Simpson 1995). None of the hold-down anchors or anchor bolts had any visible sign of damage after the tests. These data indicate that the anchors had sufficient overstrength to provide a desirable failure mechanism (i.e., yielding of sheathing-to-framing connections). **Figure 13** shows an example graph of measured and predicted tension forces in anchor bolts as a function of shear wall deformation. The forces predicted by Eq. [**2**] showed good correlation with the measurements. These observations provide experimental evidence that the use of the effective wall length in shear wall analysis offers accurate force predictions.

Conclusions

Testing walls in a wide range of aspect ratios allowed direct comparisons among the walls of different sizes. It was found that the maximum shear strength of fully anchored walls did not depend on the aspect ratio, and could be accurately predicted assuming a proper effective wall length. Walls with aspect ratios Ω2:1 were equally stiff, on a unit length basis, while narrow (4:1) walls were half as stiff, because their deflections were magnified by rigid body rotation in proportion with the aspect ratio. The shear strength of narrow walls did not degrade at high deflections due to small displacement demand on sheathing-to-framing connections.

Typically, uniform wood density allowed better distribution of sheathing displacements relative to the perimeter framing and the development of full load capacity of the sheathing-to-framing connections. However, when sheathing nails withdrew from the wood because of low wood density, the shear wall ductility and energy dissipation was reduced up to 50 percent. The designer

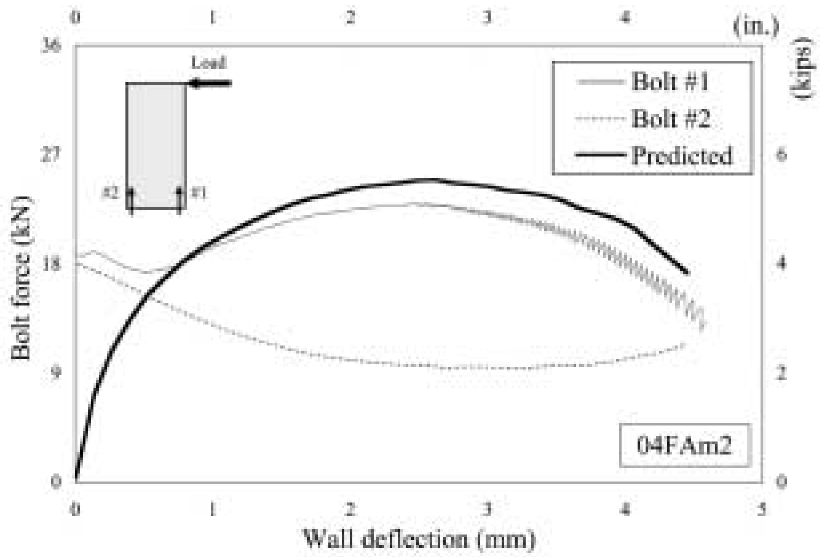

Figure 13. — Tension forces in anchor bolts.

should use higher grade lumber or dense wood species for framing members in narrow shear wall segments when they are critical components in the lateral force resisting system (such as ground-floor shear walls with large openings) to ensure acceptable performance.

Comparison of monotonic test results with published design values proved the traditional design practice for segmented shear walls to be sufficiently conservative. However, using the effective wall length in shear wall analysis would enhance the accuracy and would equalize the safety levels of walls with various aspect ratios.

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