MONOTONIC AND CYCLIC LOAD TESTING OF PARTIALLY AND FULLY ANCHORED WOOD-FRAME SHEAR WALLS

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Abstract. The objectives of this study were to evaluate the performance of wood-frame shear walls under monotonic and cyclic loads by: 1) determining variability of shear wall performance; 2) comparing performance of walls under each loading protocol; 3) evaluating effects of anchorage on wall performance; and 4) evaluating performance of walls qualitatively and quantitatively with respect to code-defined performance measures. Sets of tests consisting of eight partially and two fully anchored walls were conducted using both the ASTM E564 monotonic protocol and CUREE cyclic-test protocol for ordinary ground motions for a total of 20 walls. Statistical comparisons of parameter variance and mean values were made between partially anchored walls tested under different protocols and performance comparisons were made between partially and fully anchored walls. Cyclic tests on partially anchored walls generally exhibited a coefficient of variation that was lower than for monotonic tests. Failure mode of fully anchored walls was different than that for partially anchored walls because hold-downs changed the load path. Comparison of test results with ASCE 41 m-factors suggests that ductility of partially anchored walls is below the acceptance criteria for shear walls with structural panel sheathing.

Keywords: Seismic performance, wood-frame, shear wall, cyclic testing, CUREE protocol, monotonic protocol, prescriptive design, code performance.

INTRODUCTION

Historically, light-frame residential and commercial wood structures have performed quite well during earthquakes. However, assessments performed after several natural disasters demonstrated that damage to residential wood structures, including residences designed in accordance with today's building codes, can be very significant. In 1998, there were 14 tropical storms and 10 hurricanes in the US that caused $3.6 billion in damage and 32 deaths. Damage to wood-frame construction after the Northridge earthquake dominated in all three basic categories of earthquake loss: 1) casualties—24 of the 25 fatalities in the Northridge earthquake were caused by building damage that occurred in wood-frame structures; 2) property loss—one-half or more of the $40 billion in property damage was associated with wood-frame construction; and 3) functionality—48,000 housing

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units, almost all of them in wood-frame buildings, were rendered uninhabitable by the earthquake (Seible et al 1999).

Some of the losses may be from gaps in knowledge that require testing to be more representative of conditions in actual construction under more realistic loadings (Zacher 1999). Dolan (2000) presented a history of the design values for shear walls. All shear-wall design values in the United States are based on monotonic tests (ASTM 2000) of 2.4 × 2.4 m walls. In these standard tests, not only are the loads not representative of the short duration, random, and reversing loads experienced in an earthquake or hurricane, but the shear walls in the tests are not comparable to those used in residential and commercial construction. Because design values are based on static monotonic tests, data from these tests do not represent a dynamic event. The overall goal of this project is to address this problem by testing shear walls under actual earthquake records and compare the results with walls tested under standard protocols. This article presents the results of the monotonic and cyclic testing program. The full results of the dynamic testing conducted can be found in Seaders (2004) and White (2005).

There are currently two available design approaches for a wood-frame residential structure to resist wind and seismic lateral loads. The most common is a prescriptive approach governed by the International Residential Code (IRC) (ICC 2006b). Buildings with structural elements that do not meet the prescriptive guidelines of the IRC must instead be analyzed under the governing engineered code requirements. In the Pacific Northwest, this is currently the International Building Code (IBC) (ICC 2006a).

A primary difference between these two approaches is the IRC assumes shear walls resist overturning moments based only on the overturning resistance resulting from dead load and adjacent perpendicular walls. As a result, prescriptive shear-wall elements (brace panels) are not required to have hold-downs installed unless they are between 810 and 1220 mm wide. Many times, the same walls designed under the IBC would require hold-downs to be installed to resist tension forces in the shear-wall chords.

Most of the literature (Pardoen et al 2000; Gatto and Uang 2002) describes tests of walls with hold-downs (fully anchored) to resist tension forces in the wall chords, thus simulating an engineered design. This project focuses on partially anchored (no hold-downs), prescriptive shear walls that are more typical in residential construction.

The literature also does not fully address how shear-wall performance is affected by material and/or construction variability. The test standards for monotonic testing, ASTM E564-00 (ASTM 2000), and cyclic testing, ASTM E2126-02a (ASTM 2003), used to establish acceptable design values require only two tests unless the peak load values are more than 15% different for monotonic tests or 10% for cyclic tests, in which case a third test is required. This standard has been followed in most studies. Most researchers have used only two or three walls for any given wall treatment. Pardoen et al (2000) tested three walls per configuration under the city of Los Angeles–University of California Irvine shear-wall test program. The CUREE-Caltech wood-frame project also tested two shear walls for each configuration (Gatto and Uang 2002). Although this may be acceptable in many instances, it leaves open the question of how much variability should be expected based on materials and construction. Folz and Filiatrault (2001) emphasized the need to quantify the variability in the response of shear walls under load.

Several researchers have compared the performance of wood shear walls under various loading protocols. Dinehart and Shenton (1998) compared the sequential phased displacement (SPD) protocol with monotonic tests and found the SPD gave a 12% lower ultimate load and much lower (42%) displacement at ultimate load. They also found the SPD caused more nails to fracture or pull out compared with monotonic tests in which nails tended to pull away from the framing with the sheathing, causing more framing damage. As a result of a 30% reduction in load between the first and fourth cycles of repeated cycles with equal peak displacement, they recommended that a 25% reduction in allowable unit shear based on monotonic tests be adopted. He et al (1998) tested five walls using three different cyclic loading protocols (FCC, CEN–long, and CEN–short). They found the CEN–long protocol gave results most similar to those under simulated earthquake loading but that improvements were still required to properly model response of a wall under actual earthquake loading. Karacabeyli and Ceccotti (1998) compared five different cyclic protocols (SPD, CEN–long, CEN–short, FCC, and ISO) and compared them with monotonic and pseudodynamic tests. They found that different protocols produced different failure modes. Specifically, the SPD and FCC protocols produced nail-fatigue failure from the higher energy demands. When comparing the cyclic protocols with a pseudodynamic test, they concluded the allowable unit shear could conservatively be based on monotonic curve results, contrary to Dinehart and Shenton (1998). Most recently, Cobeen et al (2004) concluded there was no evidence to support a reduction in allowable unit shear values based on the results of the CUREE Caltech Wood Frame Project (Seible et al 1999).

The limitation of these studies (Dinehart and Shenton 1998; He et al 1998; Karacabeyli and Ceccotti 1998) for this project is that they did not incorporate the CUREE cyclic protocol (Krawinkler et al 2001), a widely used standard for cyclic testing. Also, larger sample sizes (greater than 2 – 3 walls per treatment) are needed to establish a difference in average values for ultimate load or displacement at ultimate load with a meaningful level of confidence. Thus, the question of whether the monotonic curve can be used to conservatively establish allowable unit shear values or whether the cyclic backbone curve provides a better approximation of shear-wall earthquake response remains unanswered. The project investigators hope to contribute to this question by comparing earthquake shake-table testing with monotonic and cyclic testing conducted in this project.

This article presents the first part of a two-phase research project to investigate the performance of walls under monotonic, cyclic, and various earthquake loading protocols. The overall objectives for the project are as follows:

1. To examine the behavior of shear walls under standard static test (ASTM E564) and cyclic test (CUREE) protocols for later comparison with the behavior of shear walls under various actual dynamic loading records; and
2. To understand the behavior (load-deflection response, strength, failure mode, ductility, energy dissipation characteristics, and so on) of shear walls under various actual dynamic loading records: a) subduction zone, long-duration earthquakes for Oregon/Washington; and b) earthquakes, including sequences, from specific sites in California.

This article presents the results of the monotonic and cyclic testing conducted in Phase I of the project. Specific objectives for this article are to:

1. Estimate the variability of shear-wall performance under monotonic and cyclic tests;
2. Evaluate the effects of anchorage on wall performance;
3. Compare the performance of walls under monotonic and cyclic loading protocols; and
4. Evaluate the performance of the walls qualitatively and quantitatively with respect to code-defined performance measures.

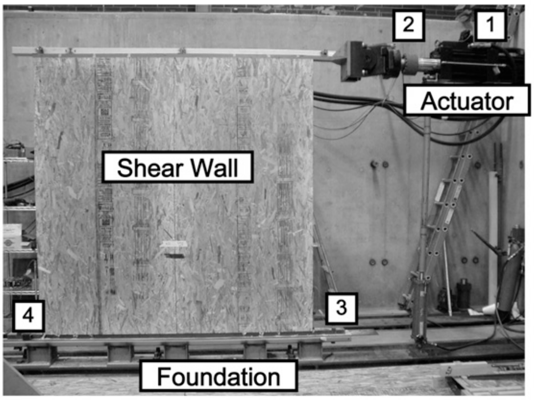
Results of preliminary earthquake testing in Phase I are given in Seaders (2004), and the remainder of the earthquake testing and results are given in White (2005).

MATERIALS AND METHODS

Load Frame and Test Equipment

All tests were conducted at Oregon State University. The loading frame used for monotonic and cyclic testing is shown in Fig 1.

Specimens were bolted to a fabricated steel beam solidly attached to the strong floor to



1. Internal LVDT – Displacement at top of wall
2. Load Cell – Applied load at top of wall
3. LVDT – Uplift (inboard end)
4. LVDT – Uplift (outboard end)

Figure 1. Monotonic and cyclic testing frame.

simulate a fixed foundation. Specimens were loaded using a 490 kN servo-controlled hydraulic actuator with a 250 mm total stroke. The hydraulic actuator was attached to the strong wall and supported by a 100 mm hydraulic cylinder. The cylinder was charged with an oil-over-air accumulator with a pressure of approximately 690 kPa. This allowed the actuator to raise and lower freely during the test without creating additional vertical loading on the wall. A steel C-channel was attached to the load cell and actuator and was laterally braced to the strong wall. The C-channel was connected to the top plate of the wall using four evenly spaced 12 mm A307 bolts installed through both top plate members; 13 mm holes were drilled in the top plates after the walls were positioned, ensuring a tight, nonslip bolted connection. The cyclic driving function was generated by an Analogic 2020 Polynomial Waveform Synthesizer. Data were collected using a personal computer with an AMD 550 MHz processor running National Instruments LabVIEW 6.1.

Wall Specimens

Wall specimens were designed as International Residential Code (ICC 2006b) prescribed brace-panel construction. Specimens were 2440 × 2440 mm as shown in Fig 2. Walls were constructed using 38 × 89 mm #2 & Better kiln-dry Douglas-fir framing. Studs spaced at 610 mm were nailed to a top plate and sill plate with 2 – 16d (3.3 × 83 mm) nails per stud, and a second top plate was nailed to the first using 1 – 16d nail at 610 mm on center. Two 32/16 APA rated 1220 × 2440 × 11 mm structural OSB panels were installed vertically with nailing as shown in Fig 2. All nails were full round-head, smooth-shank, strip-cartridge SENCO nails driven using a SENCO SN 65 pneumatically driven framing nailer. For a test specimen with the most realistic shear-wall performance, 12 mm regular gypsum wallboard (GWB) was installed opposite the structural panel sheathing.

International Residential Code (ICC 2006b) brace panel construction using structural panel sheathing (Method 3) requires 12 mm anchor bolts at least every 1800 mm, but does not require hold-downs. Thus, for the basic test specimen, 12 mm A307 anchor bolts were installed at 300 mm from each end of the wall. Walls having only anchor bolts installed without hold-downs are referred to as partially anchored. A modified wall design was included in the test matrix to account for the effects of full anchorage and for comparison with strength values published in the literature. The modified design was identical to the basic design except that SIMPSON Strong-Tie PHD-2A hold-downs were installed at the ends of the wall.

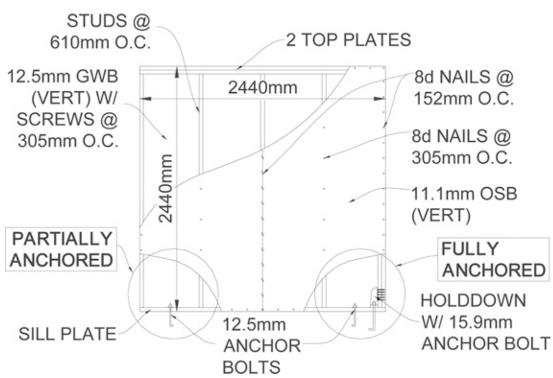


Figure 2. Test specimen schematic.

Hold-downs also necessitated installing an additional stud at each end of the wall. Walls with both anchor bolts and hold-downs installed are referred to as fully anchored.

Loading Protocols

Monotonic tests were based on the ASTM E564-00 (ASTM 2000) test protocol, which requires that ultimate load be reached in less than 5 min. Partially anchored walls were tested at a loading rate of 0.5 mm/s and fully anchored walls were tested at 0.76 mm/s. This corresponded to a time to failure of approximately 8 min for partially and 12 min for fully anchored walls.

Cyclic tests were conducted according to the CUREE protocol for ordinary ground motions developed as part of the CUREE-Caltech Wood frame project (Krawinkler et al 2001). The protocol consists of primary cycles at increasing displacement amplitudes followed by 2 – 5 trailing cycles at 75% of the primary cycle amplitude. Tests were conducted continuously up to 1.5 times the reference displacement, Δ_{ref}. Subsequent groups of cycles were separated by a short 15-s pause to allow the test to be stopped in a controlled manner in the event of wall collapse. All tests were conducted at 0.1 Hz.

Partially anchored walls were tested to a maximum displacement of 3.0 Δ_{ref} using 49 fully reversed cycles. The reference displacement was 19 mm. Reference displacement for fully anchored walls was 60 mm, and tests were limited to 2.0 Δ_{ref} by the stroke length of the hydraulic actuator (±130 mm). Fully anchored walls were tested to a maximum displacement of 2.0 Δ_{ref} using 43 fully reversed cycles.

Test Matrix

Two wall treatments (fully and partially anchored) were tested with two different loading conditions (monotonic and cyclic). Eight monotonic tests and eight cyclic tests were conducted on partially anchored walls to provide large enough sample sizes to estimate variability. However, one of the partially anchored monotonic tests was discarded as a result of operator error that caused additional vertical loading on the wall. Two walls were tested for each of the fully anchored monotonic and cyclic tests. Two partially anchored walls were also tested monotonically with dead load applied. Table 1 shows a complete listing of the testing conducted and the labeling used for each wall treatment.

Data Analysis

Backbone analysis. An analysis of the backbone curve (envelope curve) provides a useful tool for comparing results from monotonic and cyclic tests. The backbone curve for cyclic tests is derived from the hysteresis curves by drawing a line between the consecutive points of peak load of each primary cycle. Figure 3 shows an example of points of interest used to derive backbone parameters. The equivalent energy-elastic-plastic (EEEP) curve is an elastic-perfectly-plastic curve that is defined by the initial stiffness (G_e), area under the curve equal

Table 1. *Test matrix and labeling.*

Treatment	Protocol	
	Monotonic (ASTM E564)	Cyclic (CUREE)
Partially anchored	PA-MT (N = 7)	PA-CT (N = 8)
Fully anchored	FA-MT (N = 2)	FA-CT (N = 2)
Partially anchored with dead load	DL-MT (N = 2)	—

N = number of tests.

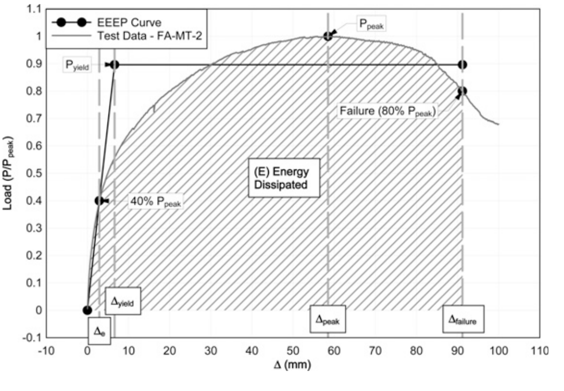


Figure 3. Explanation of parameters from backbone analysis.

to the energy dissipated (E), and the calculated yield load (P_{yield}) as defined in ASTM E2126-02a (ASTM 2003) in which:

$$P_{yield} = \left[\Delta_{failure} - \sqrt{(\Delta_{failure})^2 - \frac{2 \cdot E}{G_e}}\right] \cdot G_e \quad (1)$$

with $\Delta_{failure}$ as the deflection at failure and the initial stiffness (G_e) as the secant stiffness at 40% of peak load.

Qualitative analysis. Failures in the wall were primarily at connections. Thus, failure modes observed were classified as illustrated in Fig 4: edge breakout, nail pull-through, nail withdrawal, and sill-plate splitting.

Gypsum wallboard screws also exhibited a brittle-fracture failure mode not illustrated in Fig 4. Three possible failure modes existed for the GWB screws: edge breakout, localized gypsum crushing (similar to nail pull-through), and brittle fracture. At the end of each test, overall condition of the test specimen was recorded along with the locations and types of connection failures and condition of sheathing and framing members.

m-Factor analysis. ASCE 41 (ASCE 2007) defines a ductility parameter for each type of structural component called an m-factor. The m-factor is somewhat related to the R-factor used in the IBC, except it applies to individual elements instead of an entire system, and it modifies the load-resistance balance by increasing the structural element strength instead of decreasing the applied loads. ASCE 41 defines the acceptance criteria for linear analysis procedures for deformation-controlled (ie ductile) structures as:

$$m \cdot \kappa \cdot Q_{CE} \geq Q_{UD} \quad (2)$$

where m is the component-demand modifier to account for expected ductility associated with the selected structural performance level, Q_{CE} is the expected strength of the component or element at the deformation level under consideration, Q_{UD} is the total load resulting from earthquake and gravity forces, and κ is the knowledge factor to account for uncertainty in strength of existing structures.

The m-factor for linear analysis procedures is determined with an idealized load-displacement curve superposed on actual test data. The idealized load-displacement curve is found by drawing the elastic portion of the curve through the point at 60% of peak load and then drawing the remaining linear segments to minimize (visual approximation) the area between the idealized curve and actual load-displacement data to create equal areas under idealized and actual load-displacement curves.

The acceptance criteria m-factor for a target performance level is calculated as 0.75 times the ratio of the drift for the desired performance level to the drift at the yield point on the idealized curve. For example, the drift that defines the m-factor for collapse prevention (CP) corresponds to the failure point on the idealized curve. Thus, the m-factor would be calculated as 0.75 times the ratio of the drift at the failure point to the drift at yield. Life safety (LS) drift is taken as 75% of the CP drift, and immediate

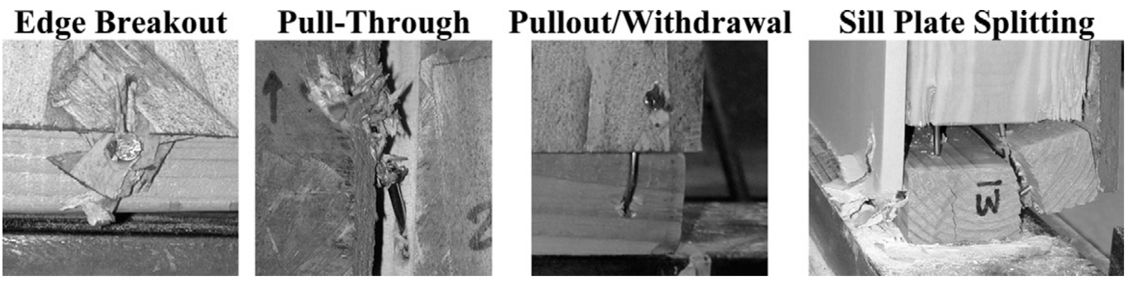


Figure 4. Failure modes observed.

occupancy drift is taken as 67% of the LS drift. Derivation of these parameters is illustrated in Figs 2 – 3 of ASCE 41 (ASCE 2007).

RESULTS AND DISCUSSION

Variability Analysis and Protocol Comparisons

Table 2 shows a comparison of the average values for the monotonic and cyclic testing and *p* values for the variance tests (F-test) and mean tests (t-tests) performed on the data. The *p* value listed indicates the probability that the null hypothesis, H_0, should be accepted (H_0 is the prediction that the variances or mean values are equal). The *p* values were calculated for both the mean test (t-test) assuming equal variance and the mean test (t-test) assuming unequal variance. The *p* value from the appropriate mean test was used based on the results of the variance test (F-test) at a level of significance of 0.05.

As shown in Table 2, COV of each parameter, except Δ_{peak} (displacement at peak load) and u_{peak} (uplift between foundation and stud at end of wall at peak load), was lower for the cyclic tests than for monotonic tests. An F-test for equal variance was performed on each backbone parameter. Variance for E, Δ_e, Δ_{yield}, and G_e showed a difference at a level of significance of 0.05 (shown in bold). Although not statistically significant, COV for P_{peak} was lower for cyclic tests compared with monotonic tests. This may be the result of the nature of the cyclic test protocol with trailing cycles at 75% of the peak displacement of the preceding primary cycle.

The incremental loading and trailing cycles of the CUREE protocol effectively permit the wall to "relax" after it has experienced damage. This relaxation allows the localized internal stresses surrounding the nails and other connections to be relieved before localized failure occurs. Therefore, in the subsequent loading cycle, stresses can be redistributed more equally throughout the wall connections. The monotonic loading protocol does not have this benefit because the unidirectional gradually increasing loading causes localized internal stresses to build up until an individual element fails. When a connection fails, stresses are immediately redistributed to the others. This sudden redistribution may have a chain reaction effect because other connections already near failure are immediately overstressed. As a result, a sudden decrease in load-carrying capacity is observed in the load-deflection curve (Fig 5) in monotonic tests. It should also be noted that results may or may not be the same for fully anchored walls because more connections are engaged during loading. Behavior of partially anchored walls is controlled by the sheathing-to-sill-plate nail connections. Because of the small number of these connections, one failure causes a significant increase in load on the remaining fasteners. In contrast, the fully anchored wall engages nearly all of the fasteners on the edges of the

Table 2. *Statistical hypothesis testing for partially anchored monotonic and cyclic tests.*

		Monotonic (N = 7)		Cyclic (N = 8)		*p* values			
							t-test mean test (H_0: $\mu_1 = \mu_2$)		
Parameter	Units	Mean μ_1	COV	Mean μ_2	COV	F-test variance test	Equal variance	Unequal variance	Ratio cyclic/monotonic (μ_2/μ_1)
P_{peak}	kN	9.65	14.9%	8.58	9.6%	0.17	**0.048**	0.058	0.89
Δ_{peak}	mm	23.4	15.6%	20.8	17.2%	0.951	**0.099**	0.099	0.89
u_{peak}	mm	16.9	18.5%	18.1	30.7%	0.184	**0.315**	0.309	1.07
E	J	238	25.1%	183	13.8%	**0.04**	0.017	**0.027**	0.77
Δ_e	mm	3.2	39.3%	2.9	17.0%	**0.028**	0.295	**0.307**	0.92
P_{yield}	kN	8.27	15.6%	7.01	9.9%	0.127	**0.016**	0.023	0.85
Δ_{yield}	mm	6.9	38.6%	6	16.9%	**0.023**	0.206	**0.223**	0.88
$\Delta_{failure}$	mm	32	18.4%	29.1	10.4%	0.105	**0.121**	0.135	0.91
G_e	kN/mm	1.33	32.7%	1.21	13.4%	**0.02**	0.246	**0.261**	0.91

Note: Performance parameters for monotonic tests were calculated directly from the load-deflection curves; parameters for cyclic tests from the backbone curve. Bold values indicate significance at the 0.05 level.

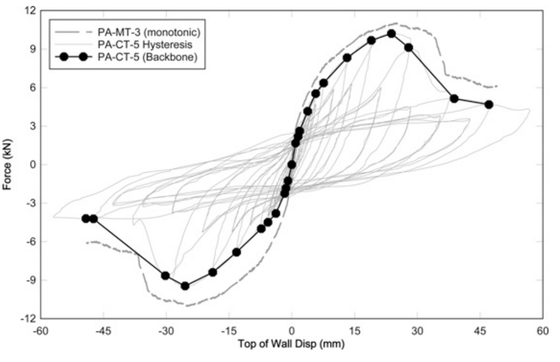


Figure 5. Backbone curves of partially anchored monotonic and cyclic tests.

sheathing panels. Thus, because of the large number of fasteners carrying load, failure of a single connection has a much smaller impact on the overall performance of the wall.

Because Dinehart and Shenton (1998) found peak load and displacement at peak load for cyclic tests to be equal or smaller than the values obtained from monotonic tests, comparisons were made using a one-tailed t-test at a level of significance of 0.05. The t-test, assuming equal variance, was used to calculate *p* values unless the F-test (for equal variance) indicated that the variance was different at a level of significance of 0.05 (ie F-test *p* value less than or equal to 0.05). In Table 2, *p* values for both tests are listed, but the bold values were used for comparisons. Average values of P_{peak}, E, and P_{yield} were different at a level of significance of 0.05. Average value of P_{peak} from the cyclic tests was 11% lower than from monotonic tests. Average value of E from the cyclic tests was 23% lower than from monotonic tests, and P_{yield} was 15% lower from the monotonic to cyclic tests. These trends are also graphically shown in the representative monotonic and cyclic curves in Fig 5. This is similar to what Dinehart and Shenton (1998) found in comparing the SPD cyclic loading protocol with static testing. They found a 12% reduction in ultimate load-carrying capacity and a 42% reduction in displacement at peak load between monotonic and cyclic tests. The difference between the 11% reduction in displacement at peak load (Δ_{peak}) found in this investigation and 42% reduction found by Dinehart and Shenton (1998) may be from differences between the CUREE and SPD loading protocols. Langlois et al (2004) concluded the performances in monotonic and cyclic tests were equal; however, all of their tests were on fully anchored walls. Comparison with dynamic tests must be made to investigate how well these tests represent the performance of walls under actual dynamic loading. One possible reason the values of P_{peak}, E, and P_{yield} may be lower under cyclic compared with monotonic tests is that the energy demand of cyclic tests is higher. Discussion of Dinehart and Shenton (1998) by Karacabeyli et al (1999) suggested that this may be the reason that the SPD protocol produced lower P_{peak} values. The same may be true in this case because the total energy dissipated (area enclosed by all hysteresis loops) in the cyclic tests to $\Delta_{failure}$ is approximately 1200 – 1300 J compared with 238 J in the monotonic tests.

One observation from Fig 5 is that positive and negative backbone curves for the cyclic tests are asymmetrical. The first explanation is when the wall experiences damage in one direction, its load-carrying capacity is reduced slightly as it is racked in the opposite direction. The second possibility is that some additional vertical loading was caused by the test equipment if the rate of uplift was faster than the hydraulic support cylinder (Fig 1) could compensate. The hydraulic cylinder that supported the hydraulic actuator (Fig 1) allowed it to freely move vertically; however, the rate at which it was able to move was limited by the flow characteristics of the supply hose and accumulator. Thus, as long as the rate of uplift was low, there was no significant vertical loading on the wall. Figure 6 shows a comparison of the monotonic and cyclic curves for fully anchored walls. The hysteresis curve for a fully anchored wall also shows some asymmetry between the positive and negative backbone curves. However, it is unlikely that test equipment caused significant effects on the fully anchored tests because uplift was very low.

Anchorage Effects

The addition of hold-downs to the wall produced a dramatic change in overall behavior and performance of the shear wall. Relevant backbone parameters from the fully anchored monotonic and cyclic tests are shown in Table 3.

By comparing Tables 2 and 3, it is apparent the addition of hold-downs produced a large increase in load-carrying capacity, deformation capacity, and energy dissipation characteristics of the shear-wall specimens. Average value of P_{peak} increased from 9.65 – 24.34 kN and 8.58 – 22.47 kN for monotonic and cyclic tests, respectively. This represents an approximate 2.5-factor increase between fully and partially anchored walls. Similarly, values for Δ_{peak} and P_{yield} increased by factors ranging from 2.1 – 2.8. The most dramatic increase between partially and fully anchored walls was in the energy dissipation parameter, which increased by factors of 8.66 and 6.39 for monotonic and cyclic tests, respectively. The value of u_{peak} was the only parameter to show a decrease between the partially and fully anchored walls, as expected.

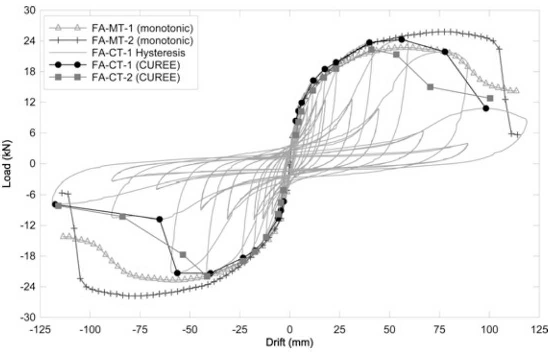


Figure 6. Backbone curves of fully anchored monotonic and cyclic tests.

Average value of uplift displacement at peak load (u_{peak}) decreased from 16.9 – 10.4 mm and 18.1 – 6.6 mm for monotonic and cyclic tests, respectively. Thus, installation of hold-downs predictably reduces the uplift at peak load. For both fully and partially anchored walls, uplift is proportional to the lateral displacement (Δ_{peak}). Thus, if "uplift" is evaluated as a ratio of uplift to lateral displacement, this difference is even more significant. Average uplift rate for partially anchored walls was 0.72 mm/mm for vertical/lateral displacement and 0.87 mm/mm for monotonic and cyclic tests, respectively. Average uplift rate for fully anchored walls was 0.16 and 0.15 mm/mm for monotonic and cyclic tests, respectively. This corresponds to a decrease in uplift rate between the partially and fully anchored walls by approximately a factor of 5.

Although no statistical comparisons can be made between tests because there are only two fully anchored walls for each wall treatment, trends similar to the partially anchored tests can be seen in the data. Average value of P_{peak} for the cyclic tests was 7.7% lower than for the monotonic tests. Surprisingly, the differences between many of the other backbone parameters are much greater. Average value for Δ_{peak} is 33% lower for cyclic than monotonic tests. This is much larger than the 11% difference observed with the partially

Table 3. *Fully anchored monotonic and cyclic backbone parameters.*

		Monotonic			Cyclic			
	Test#	FA-MT-1	FA-MT-2	Mean	FA-CT-1	FA-CT-2	Mean	Ratio cyclic/monotonic
P_{peak}	kN	22.75	25.92	24.34	22.81	22.13	22.47	0.923
$\Delta peak$	mm	58.5	74	66.3	47.9	41.1	44.5	0.671
u_{peak}	mm	8.4	12.4	10.4	7.1	6.2	6.6	0.635
E	J	1792	2333	2062	1331	1010	1170	0.567
Δe	mm	2.9	4	3.5	4	5.1	4.5	1.286
P_{yield}	kN	20.39	22.98	21.68	19.58	19.01	19.29	0.890
$\Delta yield$	mm	6.6	9	7.8	8.6	10.9	9.7	1.244
$\Delta failure$	mm	91.2	106	98.6	71.1	58.5	64.8	0.657
G_e	kN/mm	3.09	2.56	2.83	2.28	1.75	2.02	0.714

anchored walls and is very similar to results found by Dinehart and Shenton (1998). Similarly, there was a large decrease in E of 43% between the monotonic and cyclic tests.

Effect of Dead Load Application

Table 4 shows a comparison of the partially anchored walls tested monotonically with dead load (P_{DL}) applied to the average values obtained from the partially and fully anchored monotonic tests. As expected, the partially anchored walls with dead load had higher P_{peak} (41 – 88%) and Δ_{peak} (22 – 74%) values than similar walls without dead load applied. Greater increases resulted from larger applied dead loads with the fully anchored configuration serving as an upper bound for these increases.

Failure Mode Comparisons

Failure modes of partially anchored walls were almost identical for both test protocols. In every case, failures were confined to the connections between the sill plate and bottom edge of OSB and GWB panels. In several cases, sill-plate splitting occurred; however, in most cases, nail and screw connections failed first. The failure mode was so consistent for partially anchored walls with different test protocols because the connections along the bottom of the wall had a much higher demand placed on them than any of the other connections. The higher demand is because all uplift forces must be transferred through these connections into the sill plate and then through the anchor bolt into the foundation.

Table 4. *Comparison of monotonic tests with dead load applied.*

Anchorage No. of tests	Units	PA 7	DL 1	DL 1	FA 2
P_{DL}	kN	—	10.7	17.8	—
P_{peak}	kN	9.65	13.62	18.11	24.34
Δ_{peak}	mm	23.4	28.5	40.7	66.3
E	J	238	476	778	2062
G_e	kN/mm	1.33	1.11	1.10	2.83

Note: P_{DL} is the total applied dead load

Fully anchored walls exhibited a much different behavior. Here, uplift forces are transferred from the sheathing into the studs and then directly into the foundation through the hold-down and anchor bolt. Thus, the sheathing-to-sill-plate connections carry an equal portion of the shear loading as other sheathing to stud connections in the wall and are not required to carry uplift forces. This allows the sheathing connections to resist shear forces as intended and the GWB connections to act as a group. GWB connections, individually very weak, can have appreciable strength when combined.

Thus, there are noticeably different failure modes for fully anchored walls subjected to each test protocol. The monotonic test protocol primarily exhibited nail pull-through and some edge breakout in the nailed sheathing connections. GWB connections exhibited crushing around the screws with little damage to the screws themselves. Failure modes observed in cyclic testing exhibited more nail withdrawal than monotonic tests and occasional fractures in GWB screws.

Before each test, careful inspection of each wall was made and each fastener that was overdriven or too close to the panel edge was noted in the pretest report. Comparing posttest damage with pretest inspections, it was apparent that fastener edge distance controlled whether a connection would fail by edge breakout or a combination of withdrawal and pull-through. Because of limited space requirements at adjoining panel edges and difficulty in placing nails accurately with a pneumatic nailer, it was not uncommon to have several fasteners with less than the required 9.5 mm edge distance. Sheathing connections typically failed by edge breakout. Thus, special attention should be given to shear walls when adjoining sheathing panel edges are joined on 38 mm thick framing members.

Code Comparisons

An m-factor analysis was performed for each monotonic and cyclic test. Average values of the m-factor from testing of partially and fully

Table 5. *m-Factors for monotonic and cyclic tests.*

Acceptance criteria (ASCE 41 Table 8-3)		N = 7 PA-MT	N = 8 PA-CT	N = 2 FA-MT	N = 2 FA-CT
IO	1.7	1.21	1.19	2.28	1.58
LS	3.8	1.80	1.78	3.41	2.36
CP	4.5	2.40	2.37	4.54	3.15

IO, immediate occupancy; LS, life safety; CP, collapse prevention.

anchored walls are shown in Table 5. Fully anchored monotonic tests had an average m-factor (ductility) greater than that provided for wood shear walls with wood structural panel sheathing in ASCE 41 (ASCE 2007) for linear procedures at the immediate occupancy and collapse prevention performance levels. Partially anchored walls clearly did not meet the acceptance criteria.

From Table 5, it appears that there is almost no difference between the ductility of partially anchored walls tested using the monotonic and cyclic protocols. The monotonic tests of fully anchored walls, however, appear to have a noticeably higher ductility than cyclic tests. This suggests the acceptance criteria should be revised to reflect differences in ductility between fully and partially anchored walls and that the acceptance criteria for fully anchored walls may be revised to better reflect shear-wall performance based on cyclic testing. These conclusions, however, are only based on the preliminary testing in this project and more testing is needed to make a final recommendation for design. Moreover, the following aspects deserve further study: 1) effects of different cyclic test protocols on these observations/conclusions; 2) interactions between wall aspect ratio and the degree of anchorage; and 3) effect of end walls to partially offset the lack of hold-downs in partially anchored systems.

CONCLUSIONS

1. Cyclic tests on partially anchored walls generally exhibited a coefficient of variation lower than monotonic tests. Comparisons of variance using an F-test at an alpha level of 0.05 also indicated that variances of energy dissipation, displacement at 40% of peak load (Δ_e), yield displacement from EEEP curve (Δ_{yield}), and initial stiffness (G_e) were significantly different. The lower variances may be the result of a redistribution of loads during the cyclic protocol that results in more consistent properties related to failure such as peak load and energy dissipation than for monotonic tests.
2. Comparisons of average values between monotonic and cyclic tests of partially anchored walls using a one-tailed t-test show that backbone parameters for peak load, yield load, and energy dissipation are significantly different at an alpha level of 0.05.
3. Performance parameters for fully anchored walls exhibited increases over partially anchored walls by a factor of about 2.5 for peak load and displacement at peak load and a factor of almost 9 for energy dissipation. Other backbone parameters also exhibited increases.
4. Failure mode of fully anchored walls was different than for partially anchored walls because hold-downs changed the load path. Partially anchored walls failed only in the sheathing-to-sill-plate nail connections and in the sill plate itself, irrespective of loading protocol. No other fasteners in the wall experienced any visible damage or displacement.
5. Sheathing and gypsum wallboard fasteners of fully anchored walls experienced different failure characteristics when subjected to various loading protocols. Monotonic tests caused primarily nail pull-through-type failures in the sheathing connections and crushing of the gypsum in the screwed GWB connections. Fully reversed cycling of CUREE tests caused some nails to withdraw and GWB screws to fracture.
6. Partially anchored walls with dead load applied experienced increases in load-carrying capacity that were approximately proportional to the magnitude of the dead load resisting moment applied. Fully anchored walls represent an upper bound for the performance of walls with dead load.

7. Comparison of test results with ASCE 41 (ASCE 2007) m-factors shows the ductility of partially anchored walls is below acceptance criteria for shear walls with structural panel sheathing.

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