Monotonic and Cyclic Tests of Shear Walls with Gypsum Wallboard, Fiberboard, and Hardboard Sheathing

Wood Materials and Engineering Laboratory
Washington State University
P. O. Box 641806
Pullman, WA 99164-1806

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By:

J.D. Dolan
Associate Professor of Wood Engineering

and

Adam Toothman
Research Assistant

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INTRODUCTION

This report presents the procedures and results of an experimental study in which shear walls and bracing panels sheathed with gypsum wallboard (GWB), fiberboard, hardboard, and OSB were tested monotonically and cyclically to failure. There were a total of 45 walls tested which included walls sheathed on one side only and walls sheathed on both sides with different combinations of sheathing materials. Specimen materials, fabrication details, and testing procedures are explained first. Test results are then presented with a discussion of general trends and patterns observed in the data. Finally, comparisons of test results are made.

OBJECTIVES

The report provides comparative data for a wide variety of wall specimens subjected to monotonic and cyclic shear loading. The study plan included a variety of wall specimens permitting evaluation of walls with and without full end restraint (e.g., shear wall and braced wall panel), the influence of monotonic and cyclic shear loading, and the performance of walls sheathed with dissimilar materials.

EXPERIMENTAL PROCEDURES

A total of 45 wall tests were conducted in accordance with procedures for monotonic and cyclic loading (see Table 1). All wall specimens were 4ft by 8ft (2:1 aspect ratio) as shown in Figure 1 The 4 ft x 8 ft (2:1 aspect ratio) size represents the maximum shear wall aspect ratio permitted to resist earthquake loads without further reduction in design values (IBC 2000). It was shown by Salenikovich (2000) that all walls 1.2m (4ft) and longer (2:1 aspect ratio and less) developed the same unit strength.

Table 1. Wall Specimens Included in Project

Wall Sheathing	Test Number	
	Hold-down	No Hold-down
Monotonic Wall Tests – ASTM E564		
OSB	1, 1a, 2, 2a	N/A
Hardboard	3, 3a, 4	N/A
Fiberboard	5, 6	N/A
GWB	7, 8	N/A
OSB/GWB	25, 26	N/A
Hardboard/GWB	27, 28	N/A
Fiberboard/GWB	29, 30	N/A
Cyclic Wall Tests – ASTM E2126		
OSB	10, 10a	17, 18
Hardboard	11, 12	19, 20
Fiberboard	13, 14a	21, 22
GWB	15, 16	23, 24
OSB/GWB	31, 32	37, 38
Hardboard/GWB	33, 34	39, 40
Fiberboard/GWB	35, 36	41, 42

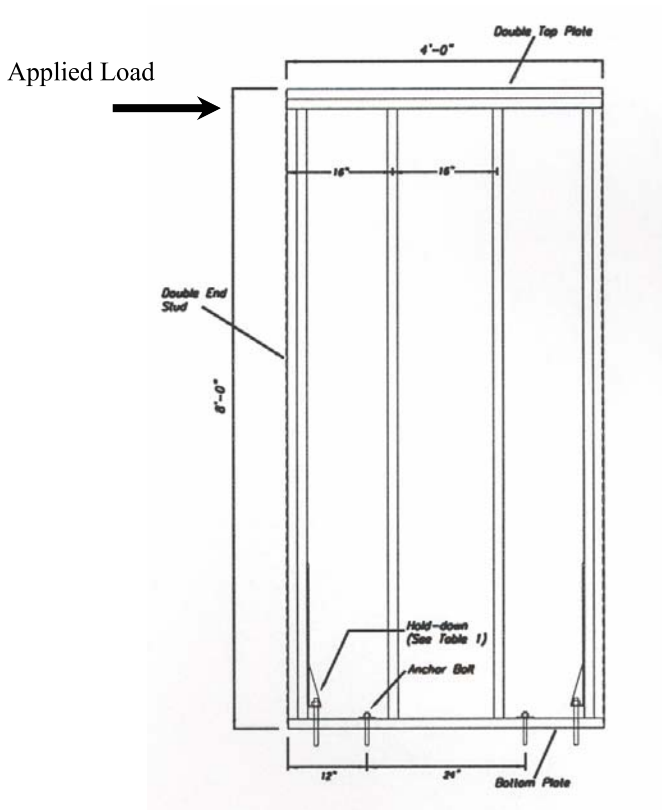

Figure 1: Shear Wall Framing Layout and Dimensions

Materials

With the exception of the fiberboard and hardboard, all of the materials were purchased from the local building materials supply store. The fiberboard and hardboard sheathing was supplied by the American Hardboard Association. The description of sheathing materials and fastening schedule are presented in Table 2. The description of framing materials, framing nail connections, and the hold-down connections are presented in Table 3.

Table 2: Sheathing description and nailing schedule

Sheathing		Nail Type	Nail Spacing (o.c)	
Material	**Thickness**		**Edge**	**Field**
OSB	11 mm (7/16") per US VPA DOC PS-2	**8d common** (Ø3.33mm x 63.5mm long) (Ø 0.131" x 2 ½" long)	152mm (6")	305mm (12")
Hardboard	9 mm (3/8") per ANSI/AHA 135.4 and 135.6	**6d box** (Ø 2.5 x 51mm long x 6.8mm Ø head) (Ø 0.099" x 2" long x 0.266" Ø head)	102mm (4")	203mm (8")
Fiberboard	12 mm (½") per ASTM C209	**11ga. Galv. roofing nail** (Ø 3 x 38mm long x 9.5mm Ø head) (Ø 0.12" x 1 ½"long x 3/8" Ø head)	102mm (4")	152mm (6")
Gypsum Wallboard (GWB)	12 mm (½") per ASTM C36	**11ga. Galv. roofing nail** (Ø 3 x 38mm long x 9.5mm Ø head) (0.12" Ø x 1 ½"long x 3/8" Ø head)	178mm (7")	406mm (16")

Table 3: Framing, Framing Connections, Hold-down Anchor Details

Component	Fabrication Details
Framing: Framing Connections: Plate to Stud Plate to Plate Stud to Stud	Stud, Spruce-Pine-Fir, (Stud or Better) 38 x 89 mm (2 x 4 in.) (nom.) @ 406mm (16 in.) o.c. Moisture content ranged from 12-15%. (2) 16d common per foot (2) 16d common per foot (2) 16d common each end
Hold-down	USP HTT22, nailed to end studs with (32) 16d (3.8 x 82.6mm) sinker nails, 15.9mm (5/8") diameter bolt to connect to foundation (See Figure 4)
Anchor Bolts:	15.9mm (5/8 in.) diameter A307 bolt with 63.5 x 63.5 x 6.35mm (2.5 in. square x 1/4 in.) steel plate washers.

Fabrication of the Walls

Wall specimens were constructed and stored inside the laboratory until it was time to test. A steel construction frame was used to ensure a square wall that could easily fabricated. A picture of the Test frame is shown in Figures 2 and 3.

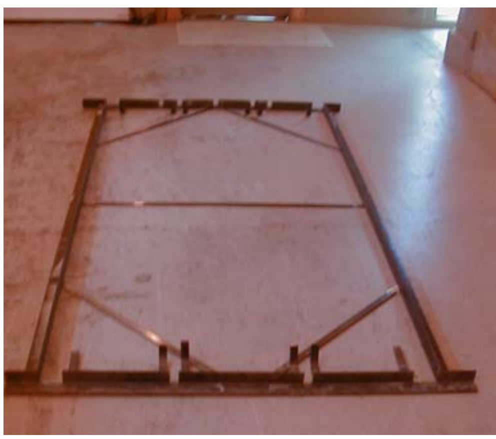

Figure 2: Construction Frame

Figure 3: Frame with Wall in place

Figure 4: Hold-down connection

All of the sheathing nail positions were pre-marked at 19 mm (¾”) from the panel edge. An edge distance of 9 mm (3/8”) is often used when two sheathing panels are attached to one stud, but the walls investigated for this test only had one sheathing panel enabling a 19 mm (¾”) edge distance at the perimeter of the panels. All nails were hand driven with the exception of the 8d common nails used to attach the OSB sheathing. The common nails were full headed, pneumatically driven, SENCO nails that were purchased from a local hardware store.

Nails were driven flush with the sheathing material as specified. For walls sheathed on one side, sheathing was placed on the bottom side of the framing to allow for easy installation of the anchor bolts, and to allow the sheathing to fall off during the tests if it became separated from the framing. For walls sheathed on both sides, GWB was placed on top and nailed to the framing after the wall was bolted into place in the test frame.

Test Setup

Walls were tested in a horizontal position as shown in Figure 5. No dead load was applied. The bottom plate of the wall was attached to a 76x127-mm (3x5-in.) steel beam by 16 mm (5/8”) anchor bolts spaced at 610 mm (24-in.) on center. For the walls with hold-downs, 16 mm (5/8”) bolts were used to attach the hold-down to the steel beam. All of the holes in the bottom plate were drilled 1.5 mm (1/16”) larger than the bolt diameter. In order to avoid interference of the sheathing with the support, the narrow face of the beam was in contact with the bottom plate of the wall. This allowed the sheathing to move over top of the steel beam without meeting any resistance. Another 76x127-mm (3x5-in.) steel beam was attached to the top plate with 16 mm (5/8”) bolts spaced 610-mm (24-in.) o.c. The top steel beam was attached to the programmable hydraulic actuator, which provided the racking force to the wall.

Figure 5: Test Setup with wall in-place

The hydraulic actuator had a displacement range of +/-152 mm (6 in.) and a capacity of 245 KN (55 Kips). It was secured between the support and the distribution beams by hinged connections as shown in Figure 6. The hinges are present to release any moment that could be applied from the displacement of the actuator and specimen. One caster was fixed to the load distribution beam to allow the free movement of the top of the wall in the direction parallel to the applied load. The description of the instruments illustrated in Figure 6 is provided in Table 4.

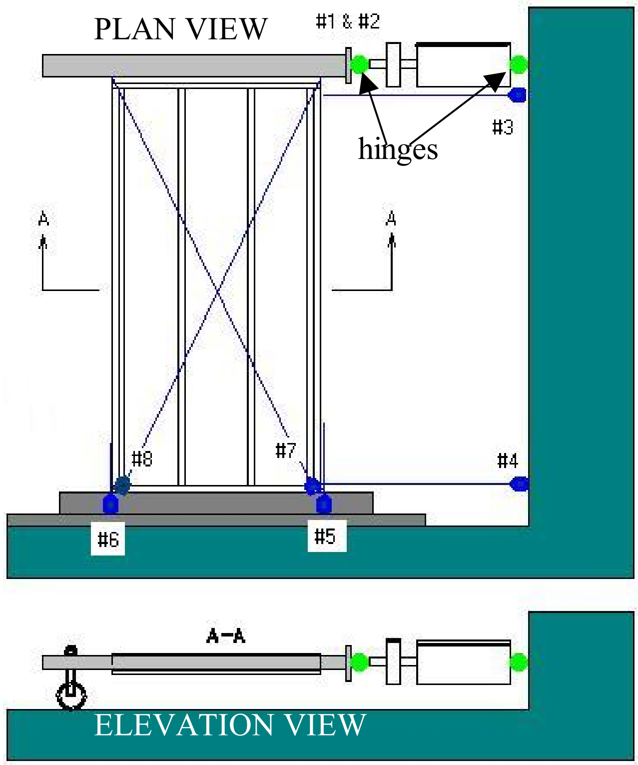

Figure 6: Plan and elevation view with LVDT locations

Table 4: Pot numbers with appropriate measurement

Pot #	Measures:
1	Ram Displacement
2	Records Load
3	Top Displacement (Verifies Pot #1)
4	Bottom Displacement
5	Right Uplift
6	Left Uplift
7	Right Diagonal Displacement
8	Left Diagonal Displacement

Instrumentation and Data Acquisition

The eight channels of the data acquisition system used for this walls. An internal linear variable differential transducer, LVDT, and the 55 kip load cell attached to the actuator were used to take measurements at the top of the wall. Resistance potentiometers (string pots) were used to measure the deflection at critical location of the walls as shown in Figure 6.

The hydraulic actuator contained an LVDT (channel #1) used to measure the horizontal deflection at the top of the wall. The load cell (channel #2) supplied information on the amount of force being resisted by the specimen. The string pot at the top of the wall (Channel #3) was used to verify the displacement from channel #1 and account for any slip between the top of the wall and the load distribution beam. Channel #4 measured the horizontal displacement of the bottom plate. If there was any slip of the bottom plate, it had to be subtracted from the top displacement in order to get the inter-story drift of the wall.

Channels #5 and #6 measured the vertical uplift at the bottom of the wall that occurred while the racking load was applied. Channels #7 and #8 measured the diagonal displacement of the walls between the top and bottom plates to obtain information on shear deformation as discussed in ASTM E564.

Testing Procedures

All of the walls were either tested using ASTM E564 for monotonic loading, or ASTM E2126 for cyclic loading.

Monotonic Testing

Monotonic tests were in accordance with ASTM E564 except that the walls were tested at a constant rate of 17 mm/min (0.67 in./min) until failure or until the actuator reached its maximum displacement of six inches. Unloading phases were omitted as the load was applied continuously until failure. This was done because of the limited information that the unloading phases provide. For the data collection, ten readings per second were recorded during the entire test.

Cyclic Testing

Cyclic tests were in accordance with ASTM E2126 Method A. ASTM E2126 closely follows the SPD procedure developed by TCCMAR (Porter 1987) with a few changes. The SPD procedure is based solely on the First Major Event (FME), while ASTM E2126 takes into account the FME and ductility ratio when determining the initial amplitude of each phase in the protocol. The FME is defined as the displacement at which the structure starts to deform inelastically (anticipated yield displacement).

In general, the SPD protocol is displacement controlled and involves triangular reversed cycles at increasing displacement levels. The displacement increase in ASTM E2126 is based on the FME and ductility ratio. The ductility ratio is the displacement at the failure load divided by the displacement at the yield load.

$$\text{Ductility Ratio} = \mu = \frac{\Delta_{failure}}{\Delta_{yield}}$$

The loading schedule and waveform pattern for a typical wall is shown in Table 5 and Figure 7. FME and the ductility ratio values for each different sheathing material are listed in Table 6. From the loading schedule defined by ASTM E2126, and shown in Table 5, it is possible that one phase will have a lower initial displacement than the previous phase. During phase five, if the ductility factor is not greater than 20, then the amplitude of the initial cycle will not be greater than 100% FME. Anytime this occurred, the phase was not included when forming the envelope curves and only increasing displacement amplitudes were included. This was done because incorporating decreasing displacement phases does not appear to provide any beneficial information into the performance of the wall

Table 5: Amplitude of initial cycle

Pattern	Phase	Amp. of Initial Cycle % FME
1	1	25
	2	50
	3	75
2	4	100
	5	5 μ
	6	10 μ
	7	20 μ
	8	40 μ
	9	60 μ
	10	80 μ
	11	Increase by 20 μ Until failure

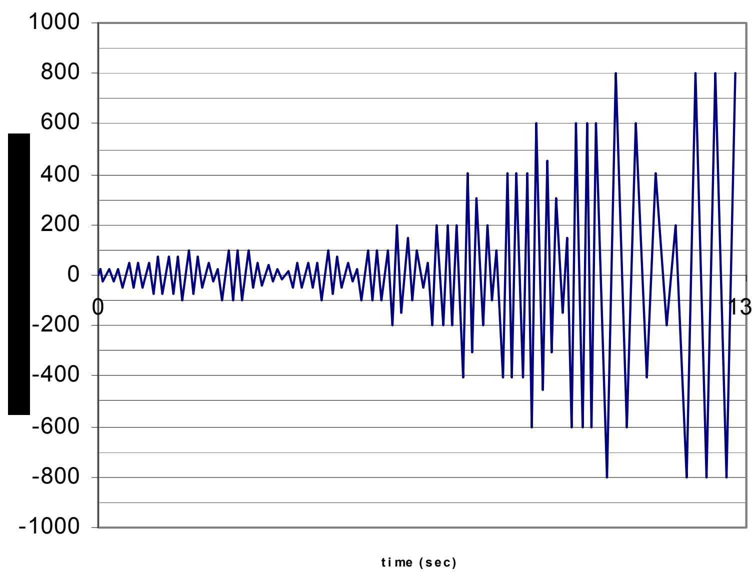

Figure 7: ASTM E2126 SPD Waveform Pattern

Table 6: FME and μ ratios for wall specimens

Sheathing Material	FME	Ductility Ratio, μ
OSB	0.5	10
Hardboard	0.45	9
Fiberboard	0.55	7.5
Gypsum Wallboard	0.55	7.5
OSB/GWB	0.65	7.5
Hardboard/GWB	0.3	15
Fiberboard/GWB	0.4	8.5

The cyclic frequency was held constant at 0.5 Hz. until the displacement amplitude was 80 μ. At this cycle, the frequency was changed to 0.25 Hz. This was done in order to minimize inertial loads on the wall. The FME and ductility ratio values were determined from the monotonic tests, using the concept of an EEEP curve as described in the EEEP Parameters section to follow. Each wall was tested twice, so the average yield load, yield displacement, and ultimate load displacement were used to determine the FME and ductility ratio. It should be noted that these values are an estimate due to the variation that is introduced when computing where the first yield actually took place, and its corresponding displacement. The values for fiberboard and GWB were similar when computed from the monotonic tests, so for comparison purposes, they were both tested under the same loading schedule. Walls with no hold-downs were tested under the same loading procedure as its corresponding anchored wall. This was done for comparison purposes and because all of the monotonic tests were performed with hold-downs.

SHEAR WALL PROPERTY DEFINITIONS

Load-Displacement Curve

For every test, a load-displacement curve was produced from the data obtained from channels #1 and #2. The displacement used to generate the graph is the interstory drift, which is the displacement of the top of the wall (channel #2) minus any displacement of the bottom of the wall (channel #4). From the data collected, it was observed that the bottom plate displacement was negligible (less than 2 mm) so it was not subtracted from the total wall displacement.

The load-displacement graph for monotonic tests (Figure 8) is always positive. Cyclic tests produce a load-displacement graph similar to the one shown in Figure 9. It consists of many positive and negative loops corresponding to its loading protocol. Two distinct curves are generated from this loading. The initial envelope curve consists of a line connecting the peak loads at the initial displacement of each phase of the SPD cyclic loading procedure. This curve represents the maximum load performance of the wall, and can be compared to monotonic load-displacement graphs Comparison of these curves allows evaluation of the strength and stiffness degradation due to fully reversed loading cycles. Since the cyclic tests include positive and negative sides, the average of the absolute values are used in the analysis of the walls. This is doe based on the assumption that a structure must be able to resist fully-reversed seismic excitations of equally large magnitudes. The stabilized envelope curve connects all of the peak loads from the last cycle of each loading phase in the loading procedure

In this study, the absolute values of the negative and positive cycles were averaged to get the initial envelope curve. The initial envelope curve for the two data sets for each sheathing material was then averaged in order to determine the overall load-displacement values. Stabilized envelope curves are determined in a similar manner. It can be seen that each wall has different load and displacement response (i.e., peak and failure loads and displacements.) Therefore, it is important to compare several different parameters when analyzing the relative performance of various wall types.

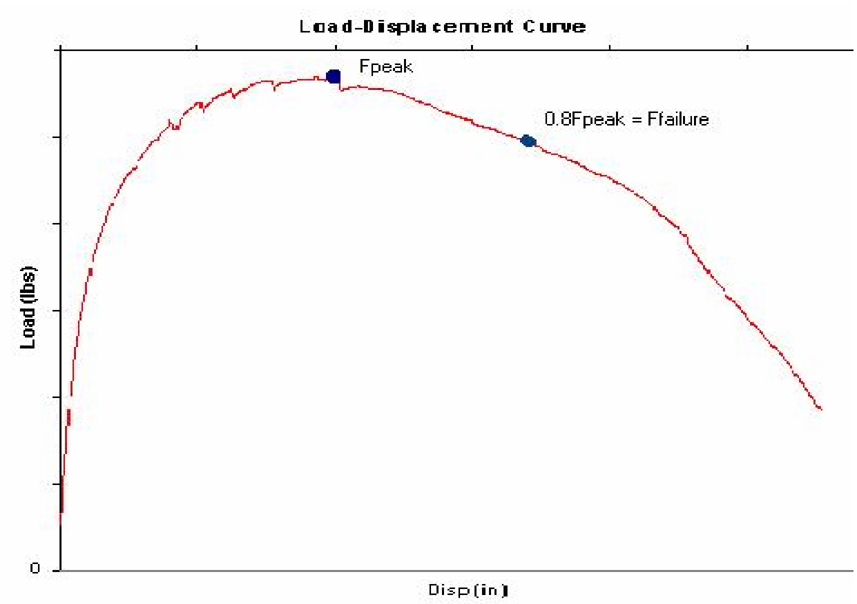

Figure 8: Typical monotonic load-displacement curve

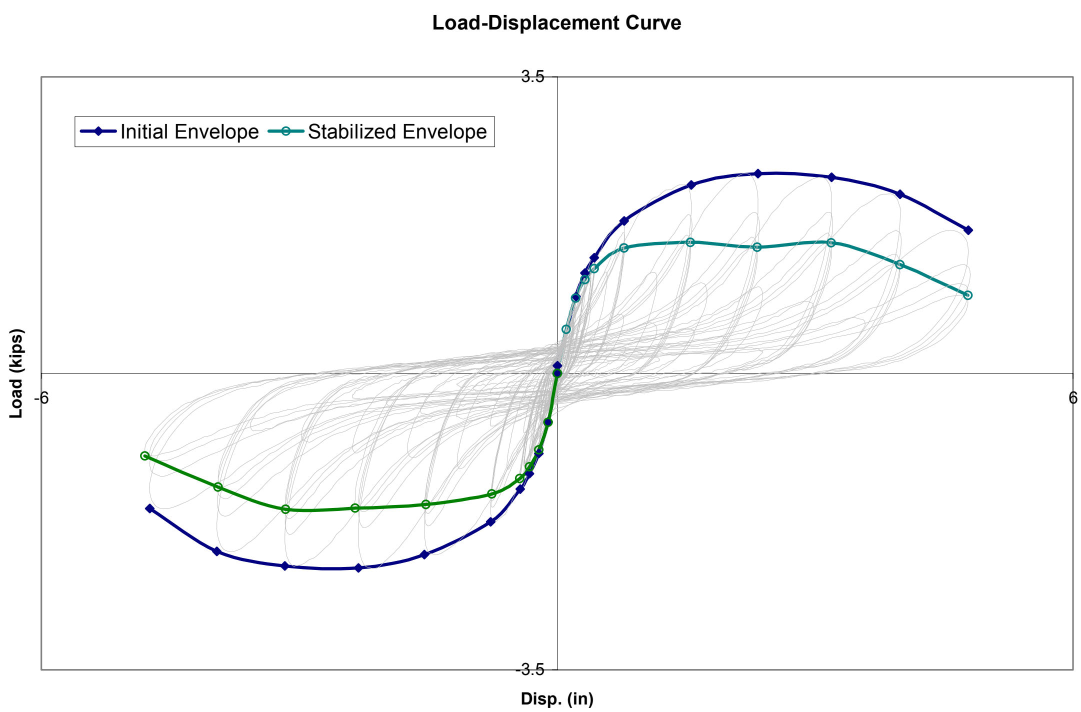

Figure 9: Typical hysteresis loops from wall subjected to cyclic loading.

Peak Load

The peak load, F_{peak}, of monotonic tests is simply the maximum load that the wall could resist during the loading period. During cyclic tests, $F_{peak,initial}$ is recorded as the maximum positive and negative load of the initial cycle of each phase. The absolute value of the maximum positive and negative load is averaged to get F_{peak} of the wall. The displacement of the wall, Δ_{peak}, is also recorded at its corresponding loading. The stabilized capacity, $F_{peak,stabilized}$ is the largest average load resisted during the last cycle of each phase of the SPD loading, and $\Delta_{peak,stabilized}$ represents the corresponding displacement at this capacity.

Failure Load

The walls tested in this study were considered to fail when the resisted load reached $0.8F_{peak}$ (or $0.8F_{peak,initial}$ depending on the test). For light-frame shear walls, the failure is rarely sudden, but instead a gradual decline mirroring its increase in load. Since $0.8F_{peak}$ is an arbitrary value for failure, it should be noted that some variation could result when comparing other parameters based on this value. Failure displacement is also recorded and used to determine the ductility of the wall.

Energy Dissipation

A structure must be able to sustain large deformations, and be able to dissipate large amounts of energy during an earthquake. Due to the uncertainties of assumptions used in mathematical models, experimental testing gives the most accurate and realistic means of predicting the hysteretic behavior of a shear wall. The amount of energy dissipated by a structure is taken directly from the load-displacement curve. From monotonic tests, it is simply the area under the curve measured from the initial displacement until the failure displacement of the wall (which is taken at the load $0.8F_{peak}$ in this study.) For cyclic tests, the total energy dissipated is defined as the area under the envelope curve as shown in Figure 9. It can be seen that the area is significantly smaller than the actual energy dissipated by the structure because the hysteresis loops overlap. The area under the envelope curve is used for comparison purposes, and to account for the variation of the different types of cyclic loading procedures.

EEEP Parameters

Light-frame wood walls do not exhibit a distinct yield load, and the proportional limit cannot be definitely set. Various definitions proposed for the yield load in the past (Foliente 1996). To determine the yield load in this study, the use of an equivalent energy elastic plastic (EEEP) curve is incorporated. An EEEP curve is a perfectly elastic plastic idealization of the actual response of the specimen, and is illustrated in Figure 10. The EEEP curve encompasses the same amount of area as the actual load-displacement curve from the origin to the failure displacement. In this study, this area is taken as a measure of the toughness of the system. Toughness is the energy that is needed fail a specimen. For monotonic tests, the toughness is simply the area under the load-displacement curve. In cyclic testing, the area is calculated from the initial envelope curve of the wall. Therefore, the calculated area is less than the total amount of energy dissipated since the hysteresis loops overlap. The area is calculated using the initial envelope curve to compute parameters used for comparison purposes.

The EEEP curve is a function of the yield load and displacement, the failure displacement, area under the observed load-displacement graph, and the elastic stiffness.

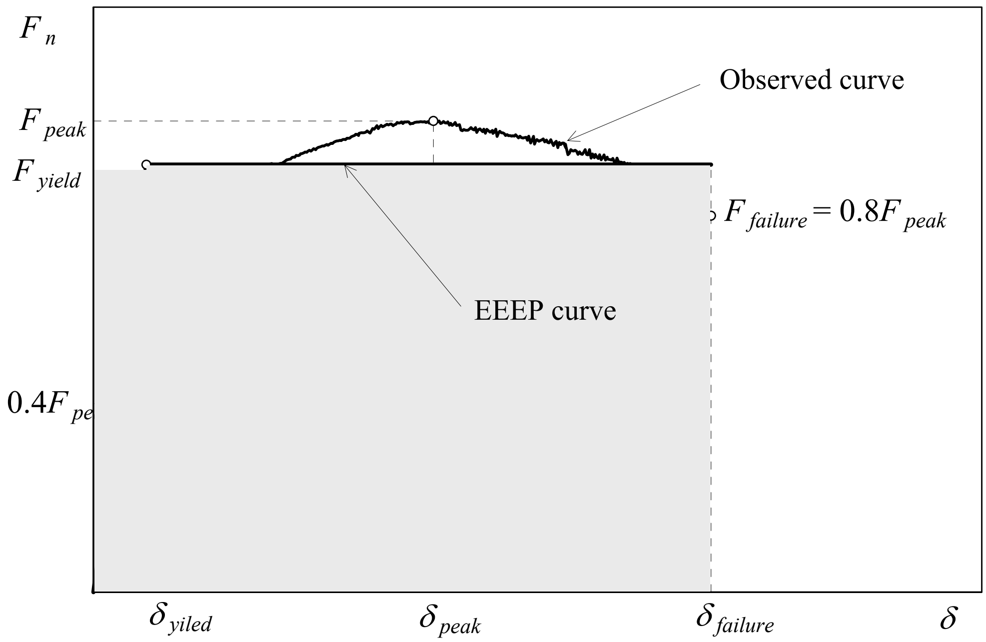

Figure 10: An Illustration of the EEEP curve with defined variables (Salenikovich, 2000)

Elastic Stiffness

The elastic stiffness, k_e, is defined by the slope of the secant passing through the origin and the point on the load-displacement curve (or envelope curve) that is equal to 40 percent of the peak load, F_{peak}. The slope of this line is used to define the elastic portion of the EEEP curve. It is also used to find other parameters such as the yield load, yield displacement, and the ductility ratio.

$$\text{Elastic Stiffness} = k_e = \frac{0.4F_{peak}}{\Delta_{0.4F_{peak}}}$$

The definition of elastic stiffness is based on the ASTM standard for cyclic tests of mechanical connections. The elastic stiffness is a good representation of the stiffness that a wall would exhibit when subjected to low to moderate displacements (Salenikovich 2000).

In this study, there is a wide variation in the elastic stiffness within an individual set of tests. This was due in part to the initial load that was put on the specimen. An initial load was sometime present when the wall was attached to the test frame and the bolt holes did not line up perfectly. This can only be considered to be actual load, even though it appeared more like a bias to the load reading. In these cases, the symmetry of the hysteretic curves (for cyclic tests only) were checked to see if in fact the load reading was a bias. None of the specimens with this issue could be corrected for this reading, and stiffness was the only variable that may have been affected by this. However, indirectly, the yield load is affected since it is defined by the initial stiffness curve. Specimens with higher initial load readings also had higher initial stiffness values.

Yield Load and Yield Displacement

The elastic portion of the EEEP curve is determined by the elastic stiffness. It begins at the origin and ends at the yield load and displacement. The plastic portion of the EEEP curve is a horizontal line equal to the yield load and extends until the failure displacement (Figure 10). In order for the area of the load-displacement curve and the EEEP curve to be equal, the value of F_{yield} is found where the area of the load-displacement curve equals the area of the EEEP curve. Assuming that F_{yield} is a function of the elastic stiffness, the area under the load-displacement graph, and the failure displacement, it can easily be calculated that:

$$F_{yield} = \frac{-\Delta_u \pm \sqrt{\Delta_u^2 - \dfrac{2A}{k_e}}}{\dfrac{-1}{k_e}}$$

where: F_{yield}= Yield Load (kip, kN)
A = the area (kip-in, kN-mm) under the observed load displacement curve from the origin to the failure displacement ($\Delta_{failure}$)
k_e = Elastic Stiffness (kip/in, kN/mm)

Once F_{yield} is determined, the yield displacement can be calculated using the relationship:

$$\text{Yield Displacement} = \Delta_{yield} = \frac{F_{yield}}{k_e}$$

Ductility

Ductility is an important feature of a structural system, which enables it to yield and deform inelastically without failure. The ability to bend, but not break is crucial when subjected to the sudden and powerful motions of earthquakes. There have been several proposed methods to express the ductility of a structure. One accepted measurement of ductility is the ratio of the peak displacement to the yield displacement.

$$\text{Ductility} = D = \frac{\Delta_{peak}}{\Delta_{yield}}$$

This definition only considers the structure's ability to yield until reaching its maximum load.

The most commonly accepted definition is the ASTM E2126 definition, which defines the ductility factor, μ, as the ratio of the failure displacement and the yield displacement.

$$\text{Ductility Factor} = \mu = \frac{\Delta_{failure}}{\Delta_{yield}}$$

This value represents the amount of displacement that a structure can undergo from yielding until failure, and takes into consideration that most ductile structures, such as light-frame shear walls, are able to resist loads far beyond Δ_{peak}. When the structural component has reached its capacity, it transfers additional load onto other components.

The ductility factor introduced above is the ratio of two displacements and is therefore not a measure of the structures ability to withstand large deformations without failing. If a structure undergoes large deformations before failing, but has a large yield displacement, the structure is not necessarily a ductile system. The reverse is also true, so ductility should always be considered together with other performance indicators.

Although ductility is an important characteristic, it should be noted that it is not a material property, and caution should be used when comparing different structural systems. Given the variation that exists when calculating the elastic stiffness and yield load, there is a large margin for error and inconsistent results. Ductility alone does not provide much insight into the performance of the walls.

Earthquake Performance Indicators

Equivalent Viscous Damping

Damping is the process by which free vibration of a structure steadily diminishes in amplitude. The energy of the vibrating system is dissipated by various mechanisms. In a typical structure, some common mechanisms that contribute to energy dissipation is the friction at steel connections, opening and closing of micro-cracks in concrete, and friction of the structure itself with nonstructural elements such as partition walls (Chopra 2001). In the case of shear wall testing, the main source of energy dissipation comes from friction of the fastener with the sheathing and framing.

The most common method for defining equivalent viscous damping is to compare the energy dissipated in one hysteresis loop of the actual system, to an equivalent viscous system. The energy dissipated by the actual system is determined from the area in the hysteresis loop denoted as E_D. The strain energy, or the energy of an equivalent viscous system, U_o, is denoted by the triangle shown in Figure 11. The equivalent viscous damping ratio is determined by:

$$\zeta \approx \frac{1}{4\pi}\frac{1}{\omega/\omega_n}\frac{E_D}{U_o}$$

where: w=testing frequency
w_n= natural frequency of the wall

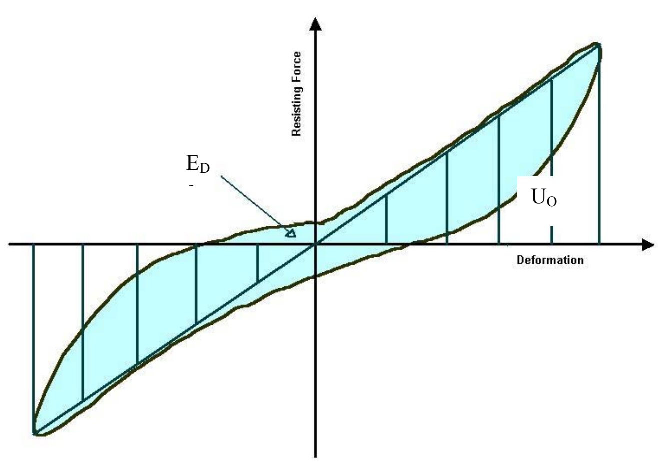

Figure 11: Typical stabilization loop used to determine damping ratio

The frequency used to test the walls in this study was held constant at 0.5 Hz. Although the natural frequency and the testing frequency were not equal, it is still a satisfactory approximation (Chopra 2001), and the damping ratio reduces to the expression:

$$\zeta \approx \frac{1}{4\pi}\frac{E_D}{U_o}$$

Not all of the hysteresis loops were perfectly symmetric. In order to be as accurate as possible, the area of both triangles were averaged to approximate the value of the Strain Energy, U_o. The equivalent viscous damping ratio was computed until failure for all of the walls in this study.

In this study, the equivalent viscous damping ratio, EVDR, is only an approximation since the walls are loaded quasi-statically beyond the elastic limit. Theoretically, the viscous damping should be zero at low loading rates where the inertial forces are zero. In addition, timber structures are predominantly non-linear, and display a complex mix of coulomb damping, internal friction, and rupture of material when loaded beyond the elastic limit (Heine 1997). It is not appropriate to compare wood-frame structures to structures with other materials when considering the equivalent viscous damping beyond the elastic limit (Lowe and Edwards 1984, Polensek 1988, Foliente 1994). Although the EVDR in the elastic range should be zero, there was some energy dissipated by friction of the sheathing-to-framing connection and compression of grain.

Because each sheathing material was tested using a different loading scheme, considering the total energy dissipation from every cycle does not provide complete information. Another way to evaluate the energy dissipating capacity of a sheathing panel subjected to cyclic loading is to plot the amount of hysteretic energy during each cycle with respect to the displacement. This concept is illustrated in Figure 12 and provides information on how well a given wall configuration dissipated energy at various racking displacements.

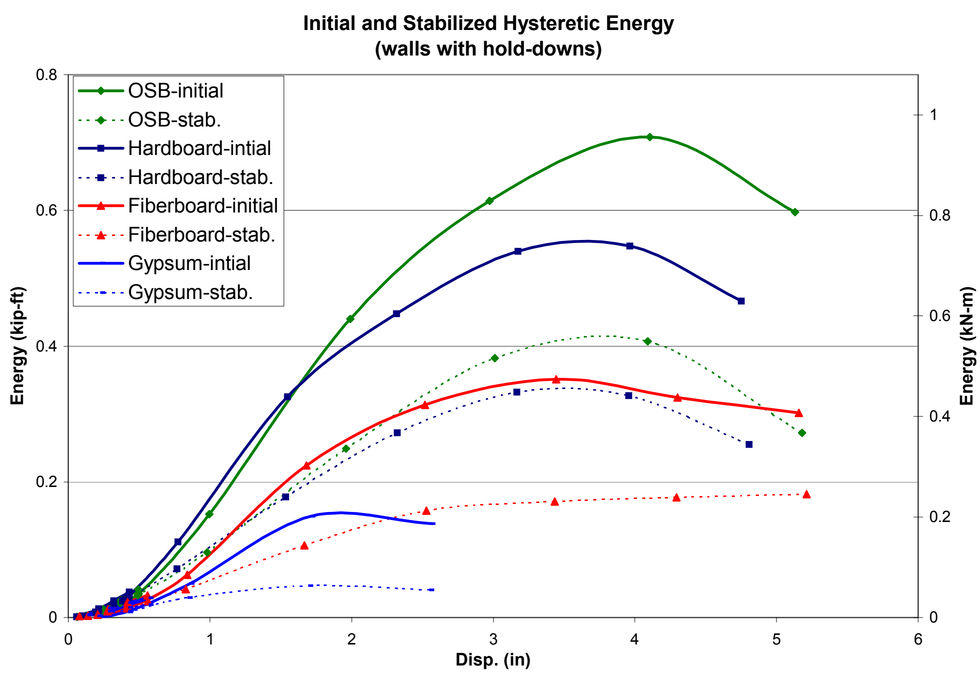

Figure 12: Equivalent Viscous Damping Ratio With Respect to Racking Displacement

Cyclic Stiffness

The average cyclic stiffness, k_c, was determined by calculating the average slope of the maximum positive and negative load values for the initial and final stabilization cycle of every phase in the loading schedule. The cyclic stiffness provides important information into the behavioral changes of the specimen. The stiffness degradation can be directly monitored by computing the cyclic stiffness, which can be used for comparing parametric changes in structural systems.

In this study, the cyclic stiffness is plotted against racking displacement to provide an indication of the stiffness degradation of the wall subjected to cyclic loading. Cyclic stiffness degradation is a result of the sheathing-to-framing connection becoming distorted. In some cases, the nails tore through the sheathing, while other times the nails pulled out of the framing. Regardless of the nail behavior, the continuously reversed cycles caused stiffness to decrease with each loading phase. This occurred because after every cycle, the nail would tear a larger hole, or pull out of the framing a little more. Therefore, the next cycle would have to be displaced that additional amount before the wall could resist more load, resulting in a reduction in the stiffness of the wall. Initial stiffness was large, but the stiffness degradation was exponential and eventually approached zero. It can easily be seen that the stiffness degradation does not correspond the strength degradation.

Wall Behavior and Mode of Failure

It is generally accepted that the sheathing nail load-slip behavior is the single most influential factor in the performance of shear walls (Soltis and Mtenga 1985; Stewart 1987; Dolan 1989). Walls in this study typically failed when the sheathing nails either pulled out of the framing or tore through the sheathing. When this happened, the sheathing panels were no longer effectively attached to the framing and the wall was unable to resist any further shear forces. Since different nails and nail spacing were used for the sheathing panels, the interaction of the nails with the sheathing and framing produced different wall behavior and modes of failure.

RESULTS AND DISCUSSION

Monotonic Test Results

Average test results for monotonic tests are provided in Table 7 and the average load-deflection curves for the four sheathing materials considered are presented in Figure 13. Note that Tests 1 and 2 were performed when a steel guide piece was used to resist uplift of the top of the wall. However, it was observed that the distribution beam was hitting the side of the steel guide piece, which was effectively adding gravity loads to the wall after peak load had been achieved. This prohibited the wall from failing, so when average values are calculated in the following sections, Tests 1 and 2 are not included when considering any parameters after the peak load. Complete results for each monotonic test can be found in Appendix A. In Appendix A, a description of the test observations is given for each specimen, along with the mode of failure. A listing of every shear wall parameter is given in tabular form. Graphs of the load-displacement curve, end-stud uplift and diagonal movements are provided in Appendix A.

Table 7. Average Results from Monotonic Wall Tests.

Wall Type	Peak Load		Δ_{Peak}		Failure Load		$\Delta_{Failure}$		Yield Load		Δ_{Yield}		Ductility Ratio	Energy Dissipation	
	(kN)	(kip)	(mm)	(in.)	(kN)	(kip)	(mm)	(in.)	(kN)	(kip)	(mm)	(in.)	μ	(kN-m)	(kip-ft)
Single Sided Walls															
OSB	11.16	2.51	97.5	3.84	8.94	2.01	142	5.6	9.96	2.24	17.5	0.69	7.15	1.31	0.97
Hardboard	9.26	2.08	57.4	2.26	7.38	1.66	117	4.61	8.4	1.89	11.6	0.46	8.7	0.95	0.7
Fiberboard	6.75	1.52	86.4	3.4	5.38	1.21	117	4.6	6.0	1.36	15.9	0.63	7.3	0.66	0.49
GWB	4.46	1.0	56.9	2.24	3.56	0.08	104	4.1	4.1	0.92	10.5	0.42	10	0.4	0.3
Double Sided Walls															
OSB/GWB	13.52	3.04	82.0	3.23	11.59	2.61	128	5.1	12.30	2.77	13.34	0.52	10.3	1.49	1.1
Hardboard/ GWB	13.0	2.92	50.3	1.98	10.32	2.32	114	4.5	11.80	2.66	6.28	0.25	18.5	1.25	0.92
Fiberboard/ GWB	10.3	2.31	66.3	2.61	8.00	1.80	93	3.7	9.17	2.06	9.42	0.38	11.0	0.81	0.6

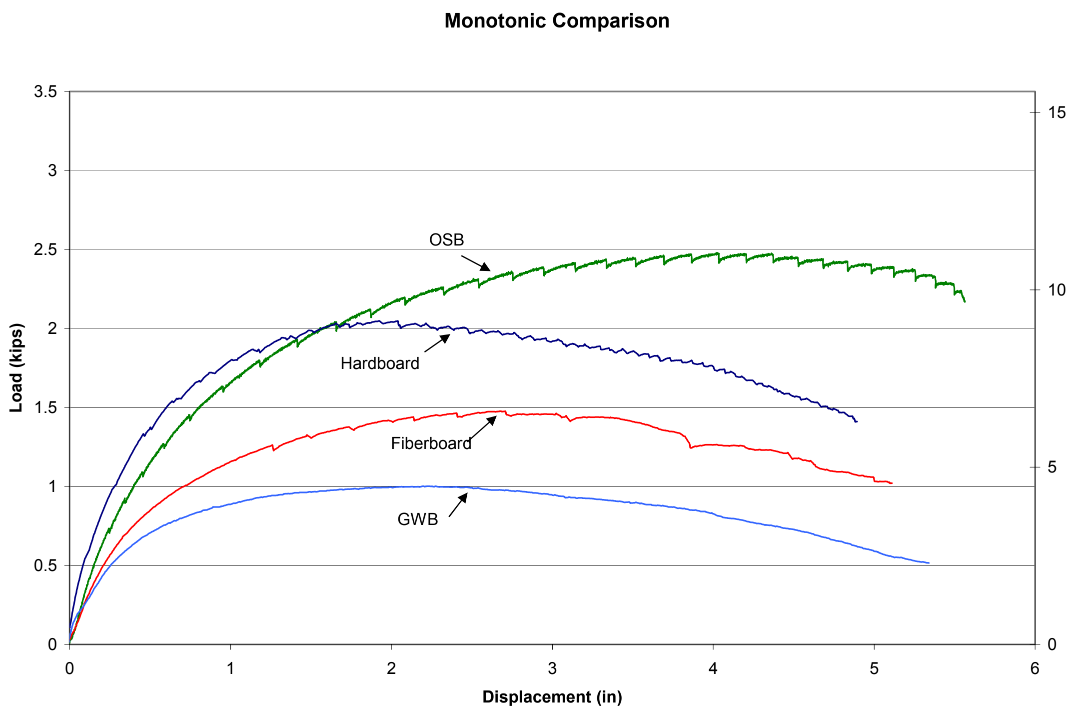

Figure 13: Load-Displacement graph of monotonic tests sheathed on one side

Peak Load

<u>Single sided walls</u>

Figure 13 provides an average load-displacement curve for OSB, hardboard, fiberboard, and GWB walls tested. The OSB wall achieved the maximum load at an average of 11.16 kN (2.51 kips), followed by the hardboard wall at 9.26 kN (2.08 kips), the fiberboard wall at 6.75 kN (1.52 kips), and the GWB wall at a load of 4.46 kN (1.0 kips).

Double sided walls

The addition of GWB to single sided OSB, hardboard, and fiberboard walls resulted in increased peak load as shown in Figure 14. If one assumes that the peak loads of the single sided wall tests are directly additive with the results of the walls with GWB, an estimation of the expected peak load of the combined sheathing materials is obtained. Ratios of actual peak load to predicted peak load are shown in Table 8, and suggest that GWB can be considered additive with other sheathing materials tested, but a small reduction of 5-10 percent in the resistance value of the gypsum sheathing may be justifiable. Variability in tested OSB wall strength, gypsum wall strength, or OSB/GWB wall strength may explain the relatively low ratio for the OSB/GWB wall type.

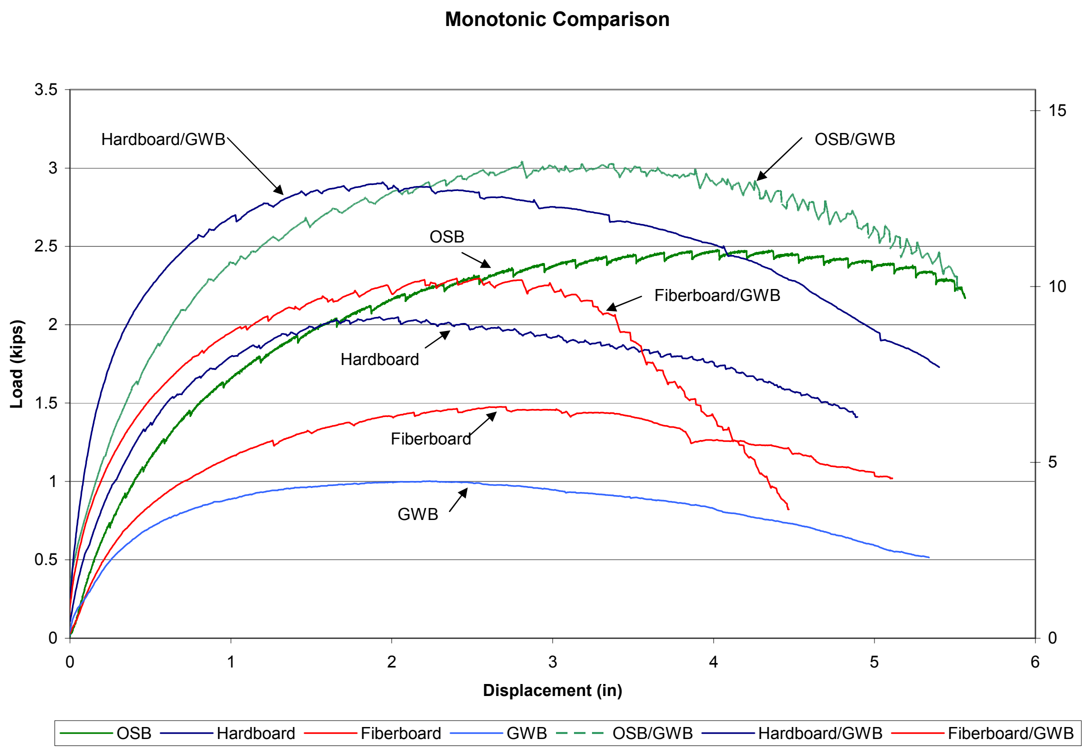

Figure 14: Contribution of Gypsum subjected to monotonic loading

Table 8: Comparison of Wall Peak Load for Walls With and Without GWB (Monotonic Tests)

Wall Type	Predicted Load (kip) (1)	Actual Load (kip) (2)	Ratio (2)/(1)
OSB/GWB	3.51	3.04	0.87
Hardboard/GWB	3.08	2.92	0.95
Fiberboard/GWB	2.52	2.31	0.92

Yield Load and Displacement

Single sided walls

As shown in Table 7, the yield load and associated displacement values for the walls with different sheathing materials followed the same pattern as the peak load. This is not surprising since the yield point is an indirect function of the peak load and failure displacement because it is calculated based on the energy under the envelope curve. The walls with OSB sheathing had the highest values of yield load and associated displacement. While the GWB had the lowest values of yield load and associated displacement.

Double sided walls

The combination of materials changed the yield parameters significantly. In all cases the yield load increased, as would be expected. The trend in yield load also followed a similar trend to the one-sided wall results in that the OSB/GWB had the highest value, followed by the hardboard and fiberboard values in that order. However, the yield displacement value for the hardboard is lower than the value for the fiberboard. This is a possible indication of where the addition of GWB has the largest effect on performance and the displacement information should be investigated in future studies of combined sheathing material effects on wall performance.

Elastic Stiffness

Single sided walls

Average elastic stiffness values for the walls tested are shown in Table 9. The GWB wall and the fiberboard wall had an elastic stiffness of 0.39 kN/mm (2.22 kip/in). The hardboard sheathed walls had the highest elastic stiffness, followed reasonably closely by the OSB sheathed walls. There was a significant difference between these two types of walls and fiberboard and GWB sheathed walls, indicating the soft materials did not resist the nail bearing as well.

Table 9: Elastic Stiffness Values for Monotonic Wall Tests

Wall Type	Elastic Stiffness	
	(kN/mm)	(kip/in)
Single Sided Walls		
OSB	0.57	3.25
Hardboard	0.72	4.11
Fiberboard	0.38	2.16
GWB	0.39	2.19
Double Sided Walls		
OSB/GWB	0.92	5.33
Hardboard/GWB	1.88	10.64
Fiberboard/GWB	0.97	5.42

Double sided walls

Sheathing the walls with different materials (i.e., adding GWB to the opposite side of the wall) significantly increased the elastic stiffness of all of the walls, with the initial stiffness of the hardboard sheathed walls more than doubling. The stiffness of the OSB walls almost doubled, while the stiffness of the fiberboard wall almost tripling. This parameter was also observed to be the most affected by finish materials on structural walls by the CUREE-Caltech Woodframe project (Cobeen et al, 2003).

Failure Load and Displacement

Single sided walls

As shown in Table 7, OSB sheathed walls experienced the largest force and displacement at failure when subjected to monotonic loading. Although the walls sheathed with hardboard had more strength than fiberboard sheathed walls, the displacement at failure was nearly the same. The GWB wall was the weakest assembly tested and also failed at the lowest displacement.

Double sided walls

The failure displacement of the double sided walls decreased by an average of 13.5 mm (0.53 in.) The displacement capacity of the fiberboard/GWB walls decreased by 25 mm (1.0 in.) when compared to walls with fiberboard alone. The more rapid reduction in strength after peak load is reached in the fiberboard walls with GWB can be seen in Figure 14.

Ductility Ratios

Single sided walls

Ductility ratios varied from 10.9 to 5.1. GWB walls were the most ductile with a ductility ratio of 10. However, GWB did not resist much load, so the yield load and displacement were small. Since the GWB was able to maintain its minimal load through a large displacement, the ductility ratio was large. Hardboard walls exhibited a ductility ratio of 8.7, while OSB walls had a ductility ratio of 7.2. Elastic stiffness of the hardboard walls was greater than that of OSB walls so the yield displacement was smaller than for OSB walls. Even though the average failure displacement of the OSB walls was 25 mm (1.0 in) larger than the fiberboard wall, the ductility ratio of the fiberboard wall was larger since the yield displacement was significantly smaller.

Double sided walls

The ductility of GWB by itself was large when compared to the other sheathing materials. When included with another sheathing materials, the ductility ratio increased by a substantial amount. This can be attributed to the increase in elastic stiffness, which decreased the yield displacement. When GWB was included, the failure displacement actually decreased on average by 13.5 mm (0.53 in.) Although more tests are needed to give an accurate measurement of the ductility, Table 7 shows that the addition of GWB tends to increase the ductility of the system

Energy Dissipation

Single sided walls

The energy dissipated by the OSB and GWB sheathed walls yielded reasonably consistent results. While the response of the hardboard and fiberboard sheathed walls was erratic after reaching peak load, and the amount of energy dissipated was inconsistent. On average, the OSB wall was able to dissipate 1.31 kN-m (0.97 kip-ft) of energy. GWB sheathed walls dissipated 0.41 kN-m (0.3 kip-ft) of energy. From the hardboard tests (Walls 3 and 4), the energy dissipated was calculated to be 1.18 kN-m (0.87 kip-ft) and 0.53 kN-m (0.39 kip-ft), respectively. Due to the large discrepancy of these two values for hardboard sheathing, Test 3a was conducted and the energy dissipated was calculated to be 1.14 kN-m (0.84 kip-ft). Combining the three results together, the coefficient of variation was 38.2%. Wall 4 behaved as the other two walls until it reached its peak load. After reaching peak load, Wall 4 drastically reduced in strength, which resulted in minimal energy dissipation after peak load. Wall 4 failed at 74 mm (2.9 in), while Walls 3 and 3a failed at 140 mm (5.5 in). Variability of the sheathing panel, framing members, or specimen fabrication may explain Wall 4's sudden failure.

A similar occurrence was observed for the fiberboard sheathed walls (Walls 5 and 6). Walls 5 and 6 dissipated 0.81 kN-m (0.6 kip-ft) and 0.52 kN-m (0.38 kip-ft) of energy. Although both walls nearly reached the same maximum load and elastic stiffness, Wall 6 reached its maximum load at 66.3 mm (2.61 in), while Wall 5 reached its maximum load at 107 mm (4.2 in). Wall 6 failed at 94 mm (3.7 in), while Wall 5 failed at 140 mm (5.5 in). This allowed Wall 5 to dissipate a much greater amount of energy because it could deform without failing. The reason for this discrepancy is unknown, but it may be explained by variability of sheathing materials, framing members, or specimen fabrication.

Double sided walls

It can be deduced from the results in Table 7 that there was some increase in energy dissipated when GWB was included in the specimens. The GWB walls way themselves were able to dissipate an average of 0.4 kN-m (0.3 kip-ft) of energy, while only an average of 0.21 kN-m (0.15 kip-ft) increase in energy dissipation was achieved when GWB was used with another sheathing material. However, it can be concluded that the addition of GWB does increase the energy dissipation capacity of the wall system.

Wall Behavior and Mode of Failure

Single sided walls

OSB Walls – A majority of the damage was observed on the bottom plate in the form of sheathing nails pulling out of the framing and/or nail tearing through the sheathing edges. Similar damage was also observed at the other panel edges. At large deformations, the tension side, end stud pulled away from the top plate as shown in Figure 15. Three typical types of damage are illustrated in Figures 15-17.

Figure 15: End stud separation at the top of the wall specimen (OSB)

Figure 16: Nail pullout and tear-through on bottom plate (OSB)

Figure 17: Unzipping of nails on bottom plate (OSB)

Hardboard Walls – Sheathing nails always pulled out of the framing. The nails did not damage the hardboard sheathing as can be seen in Figures 18 and 19. The smaller diameter and shorter nail, as well as the increased density of the hardboard sheathing contributes to this pull out failure mechanism in lieu of tear out of failures observed for the OSB walls. Sheathing nails in the bottom plate pulled out first as shown in Figures 18 and 19, followed by the nail pullout along the end studs. When the sheathing nails pulled out of the framing, this allowed the tension end stud to pull away form the top plate as shown in Figure 20. This is similar to one of the failure mechanisms for OSB walls shown in Figure 15.

Figure 18: Nail pullout along bottom plate (hardboard)

Figure 19: Nail pullout in hardboard panels

Figure 20: End stud separation at top of wall (hardboard)

Fiberboard Walls – Fiberboard nailing generally tore through the sheathing edge as shown in Figure 21. However, due to the large head diameter of the nail, the nails were also observed to pull out of the framing in some cases. The fiberboard nails pulled out of the framing or tore though the sheathing around the entire perimeter of the wall as shown in Figures 21 and 22. Some of the sheathing nails completely pulled out of the framing and were lying on the ground after the test. There was some minor separation of the end stud and the top plate, but not as much as for the OSB and hardboard sheathed walls.

GWB Walls – GWB nailing generally tore through the panel edges. Nail tear through would start along the bottom plate and continue around the perimeter of the wall as shown in Figures 23 and 24. It was observed that the sheathing would completely fall off of the frame during the test.

Figure 21: Nail tear-through (fiberboard)

Figure 22: Sheathing-to-framing separation (fiberboard)

Figure 23: Nail tear-through in the gypsum panels

Figure 24: Nail tear-through on the bottom plate (gypsum)

Double sided walls

For the walls sheathed on both sides, the general observations from the tests were similar to when the walls were tested with sheathing material on one side only. Illustrations of the failure mechanisms for the double sided walls are provided in Figures 25-27. The GWB was always the fist to fail in the wall specimens.

Figure 25: Racking displacement of sheathing panels

Figure 26: Nail pullout and tear-through of fiberboard and gypsum

Figure 27: Fiberboard and Gypsum interaction

Cyclic Test Results

Average test results are provided in Table 10. Individual specimen results for the cyclic tests with hold-down connection are presented in Appendix B, while the individual specimen results for the cyclic tests without hold-down connections are presented in Appendix C. In Appendices B and C, a description of the test observations is given for each specimen, along with the mode of failure. A listing of every desired shear wall parameter is given in tabular form. Load-displacement curves are also provided in Appendices B and C. A typical cyclic load-displacement curve is illustrated in Figure 28.

Table 10. Average Results from Cyclic Wall Tests (wall with hold-down connections).

Wall Type and Cycle	Peak Load		Δ_{Peak}		Failure Load		$\Delta_{Failure}$		Yield Load		Δ_{Yield}		Ductility Ratio	Energy Dissipation	
	(kN)	(kip)	(mm)	(in.)	(kN)	(kip)	(mm)	(in.)	(kN)	(kip)	(mm)	(in.)	μ	(kN-m)	(kip-ft)
Single Sided Walls															
OSB															
Initial	9.83	2.21	76	2.98	7.87	1.77	116	4.58	8.72	1.96	16	0.63	7.2	10.57	7.80
Stabilized	6.60	1.48	62	2.45	5.28	1.19	100	3.92	6.02	1.35	9	0.37	11.1	11.24	8.29
Hardboard															
Initial	10.0	2.25	59	2.32	8.00	1.80	106	4.17	8.95	2.01	10	0.40	10.6	9.59	7.08
Stabilized	7.06	1.59	54	2.13	5.65	1.27	95	3.75	6.40	1.44	5	0.20	19.0	11.45	8.44
Fiberboard															
Initial	6.49	1.46	64	2.52	5.19	1.17	109	4.29	5.70	1.28	13	0.51	8.4	5.87	4.33
Stabilized	4.48	1.01	59	2.32	3.58	0.81	93	3.65	3.97	0.89	7	0.29	13.3	6.32	4.66
GWB															
Initial	3.74	0.84	44	1.71	2.99	0.67	65	2.55	3.20	0.71	7	0.29	9.3	1.53	1.13
Stabilized	2.39	0.54	12	0.47	1.91	0.43	12	0.48	1.76	0.39	1.9	0.08	20.0	0.30	0.22
Double Sided Walls															
OSB/GWB															
Initial	12.33	2.77	53	2.09	9.87	2.22	103	4.07	10.89	2.44	10	0.39	10.3	12.42	9.16
Stabilized	8.47	1.91	53	2.08	6.78	1.52	87	3.46	7.56	1.70	3	0.12	29.0	11.50	8.48
Hardboard/GWB															
Initial	12.77	2.87	60	2.36	10.22	2.30	107	4.19	11.51	2.59	14	0.56	7.6	10.41	7.68
Stabilized	8.27	1.86	55	2.17	6.62	1.49	97	3.84	7.48	1.68	6	0.23	16.2	11.32	8.35
Fiberboard/GWB															
Initial	9.14	2.06	50	1.96	7.31	1.64	96	3.78	8.11	1.82	9	0.36	10.6	7.86	5.80
Stabilized	6.48	1.46	37	1.45	5.19	1.17	89	3.51	5.82	1.31	4	0.16	22.2	9.17	6.76

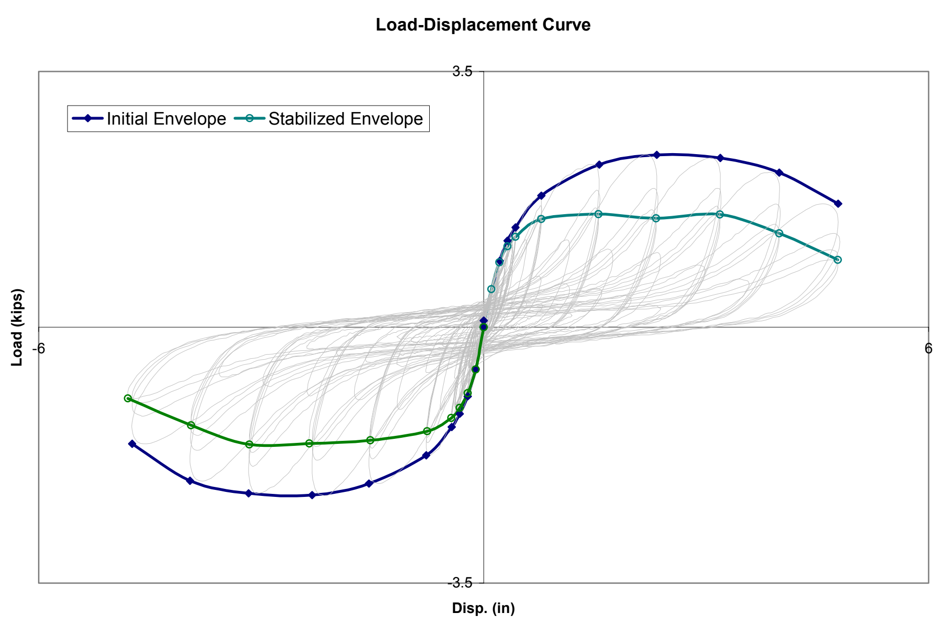

Figure 28: Typical load-displacement curve subjected to cyclic loading

Peak Load (walls with hold-down connections)

Single sided walls

The average initial cycle envelope curves for the walls tested cyclically with hold-down connections are shown in Figure 29. As can be seen, hardboard sheathed walls had an average peak load of 10.0 kN (2.25 kips), OSB sheathed walls had an average peak load of 9.83 kN (2.21 kips) fiberboard sheathed walls had an average peak load of 6.49 kN (1.46 kips), and GWB had an average peak load of 3.74 kN (0.84 kips). Stabilized envelop curves for the wall specimens are presented in Figure 30. These curves were constructed using the same procedures and the initial envelop curve. As shown, the maximum degredation when subjected to continuous cycles at the same displacement was 30 – 40%.

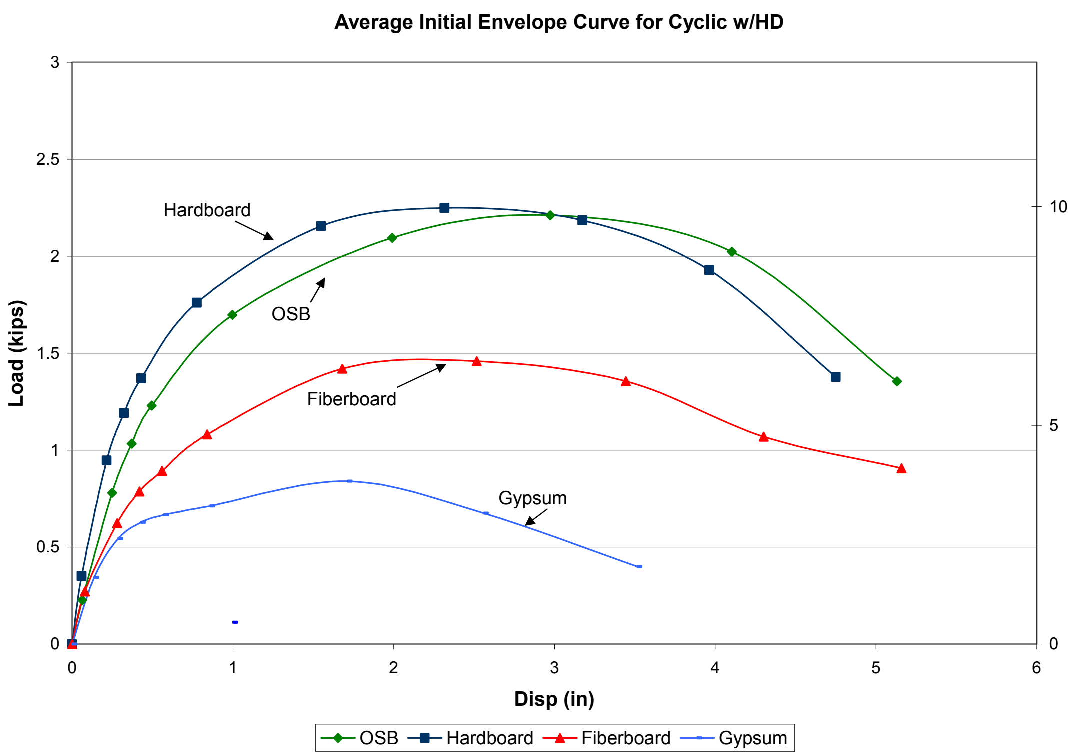

Figure 29: Average Initial Envelope Curve

Double sided walls

The peak load of double sided walls was compared to walls sheathed on one side to show the contribution of GWB to the maximum strength of the wall. The average initial envelop curves for OSB, hardboard, and fiberboard are presented in Figure 31 along with the average envelop curves for GWB alone and for the combination of each of the three sheathing materials with GWB. Where GWB was added, the peak load was not linearly additive to the other sheathing materials, but it still contributed to increased overall peak load. Predicted peak loads were estimated using the assumption of linear addition for peak loads. The resulting predictions along with the actual average test values are presented in Table 11. The ratio of the two peak load values provides an indication of the accuracy of the assumption of linear addition is. As can be seen, the GWB is approximately 90% effective when applied in combination with the other three sheathing materials.

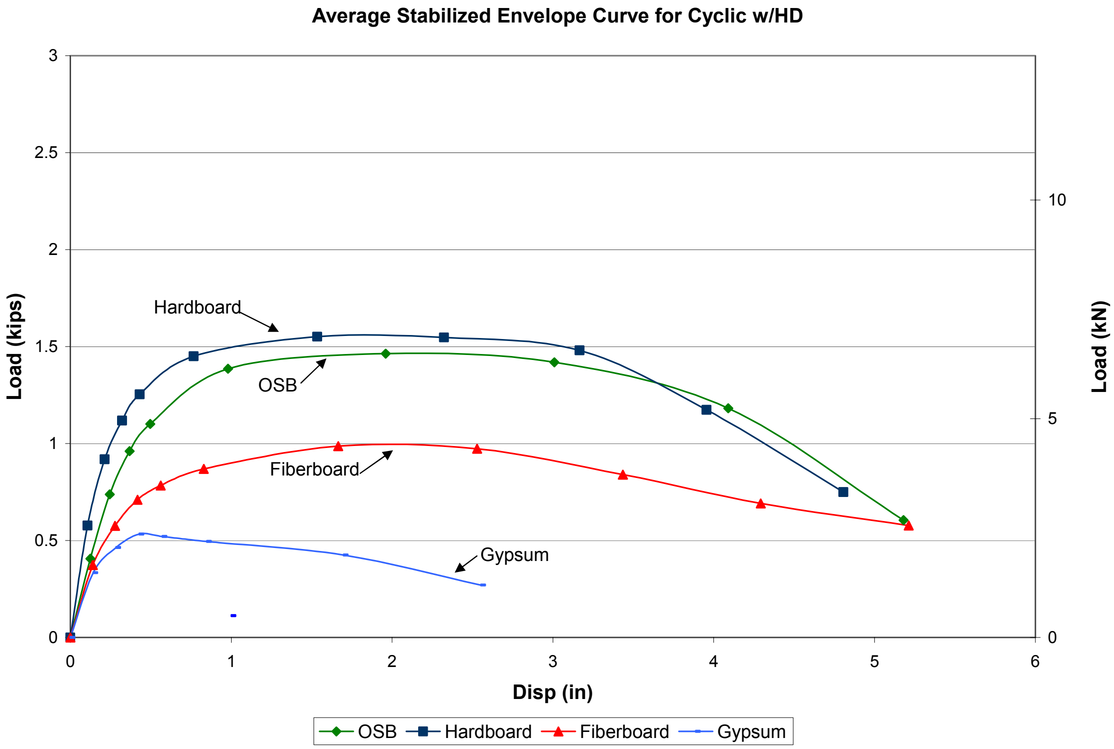

Figure 30: Average Stabilized Envelope Curve

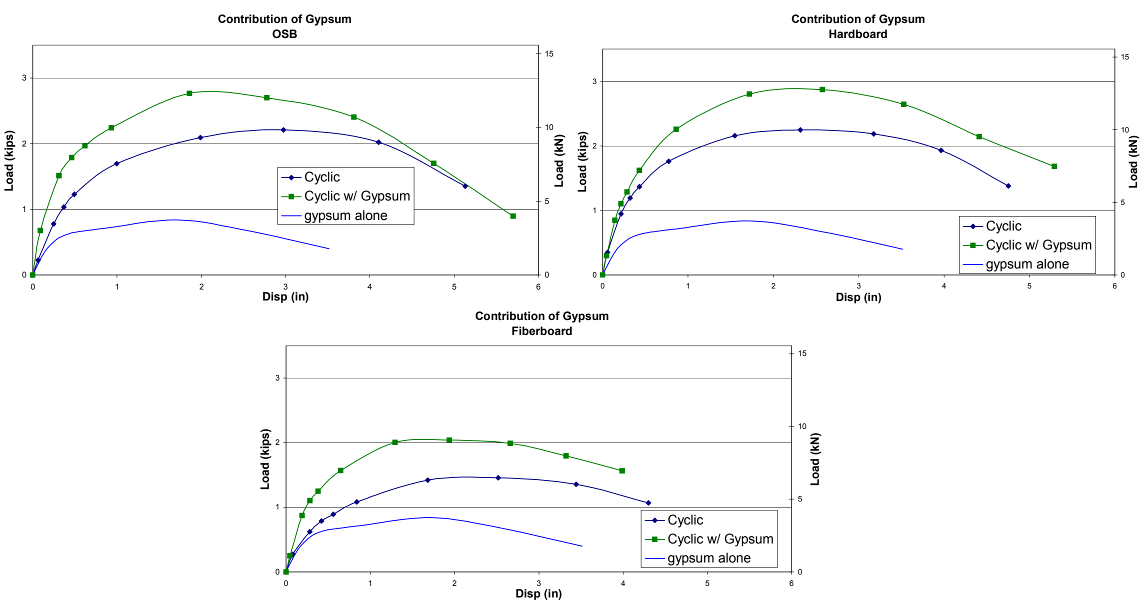

Figure 31: Contribution of gypsum to cyclic tests with hold-downs

Table 11: Comparison of walls with and without GWB (cyclic tests with hold-down connections.)

Wall Type	Predicted Load (kip) (1)	Actual Load (kip) (2)	Ratio (2)/(1)
OSB/GWB	3.05	2.77	0.91
Hardboard/GWB	3.09	2.87	0.93
Fiberboard/GWB	2.30	2.06	0.90

Elastic Stiffness (walls with hold-down connections)

Single sided walls

The elastic stiffness of the initial envelope curves followed the same pattern as peak load. Hardboard sheathed walls had the highest stiffness and gypsum walls had the lowest stiffness. Hardboard sheathed walls had an elastic stiffness of 0.92 kN/mm *(5.27 kip.in). OSB sheathed walls had an average elastic stiffness of 0.66 kN/mm (3.76 kip/in). Fiberboard and GWB had an elastic stiffness of 0.51 kN/mm (2.91 kip/in) and 0.45 kN/mm (2.56 kip/in), respectively.

Double sided walls

The elastic stiffness of the initial and stabilized curves were similar because the load-displacement curve at 40% of maximum load was the same. At this point, the wall has not yielded, so there was very little strength degradation during the stabilized cycles at low load levels.

Yield Load and Displacement (walls with hold-down connections)

Single sided walls

As shown in Table 10, the yield loads can be grouped into three groups, with the highest yield load being attributed to OSB and hardboard sheathed walls. The middle yield load is attributed to the fiberboard and the low yield load is associated with the GWB sheathed wall. The yield displacements can be grouped somewhat the same way with the exception of the hardboard being closer to the fiberboard than the OSB displacement.

Double sided walls

When GWB is combined with the other three sheathing materials, the yield loads all increase, however, the grouping according to value and order is the same. The yield displacement was not affected much by the addition of GWB to the three wall assemblies. The order of magnitude for the yield displacements changes to the hardboard/GWB sheathed wall providing the highest value, followed by the OSB/GWB and the fiberboard sheathed wall. The fiberboard sheathed wall had almost the same yield displacement as the OSB sheathed wall.

Failure Load and Displacement (walls with hold-down connections)

Single sided walls

OSB and hardboard sheathed walls were able to sustain the largest loads at failure. The failure displacement of hardboard sheathed walls was 9% smaller than OSB walls. Although fiberboard sheathed walls were weaker, than OSB and hardboard sheathed walls, fiberboard walls failed at a similar displacement as the other two walls. GWB sheathed walls failed at a displacement of 65 mm (2.55 in) less than any other wall.

Double sided walls

As shown in Table 10, displacement at failure when using GWB, in combination with the other three sheathing materials, either remained constant or decreased up to 10% when compared to the corresponding single sided wall. The failure displacement of the hardboard walls stayed constant when GWB was included. GWB appears to increase the elastic stiffness and the peak load of the walls, however, after reaching the peak load, the slope of the enveloped curve tends to decrease faster in walls with GWB included. The increase in the slope indicated that the wall fails at a quicker rate, and reduced the displacement capacity of the wall. This tends to be a characteristic behavior when GWB is used with other sheathing materials tested in this study.

Ductility (walls with hold-down connections)

Single sided walls

Using the load-displacement envelope curve, the ductility factor was taken directly from the EEEP curve (illustrated in Figure 32). The EEEP curve was constructed using the average initial envelope curve. As shown in Table 10 the ductility factor for the initial cycle of the four wall types varied from 7.2 to 10.6 for the initial cycle loading. Hardboard sheathed walls were the most ductile. GWB sheathed walls had an average ductility factor of 9.3, and fiberboard sheathed walls had an average ductility factor of 8.4. OSB walls were the least ductile and had an average ductility factor of 7.2. It should be noted that only two specimens of each configuration were tested and the variability of the results is high. Therefore, the accuracy of the ductility number needs to be validated with additional tests. However, the factor does provide some indication of the wall behavior.

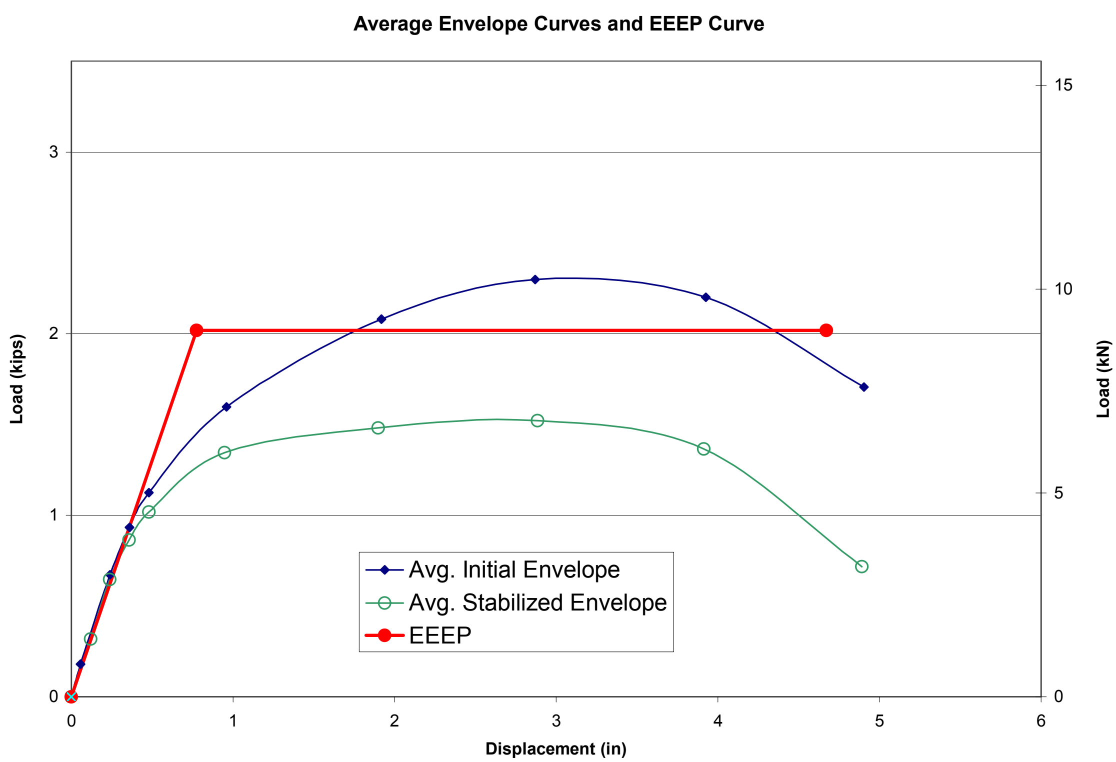

Figure 32: Typical EEEP curve for cyclic test with hold-downs

Double sided walls

There does not seem to be a direct correlation between the ductility factor and the addition of GWB to the walls. This is due to the definition of ductility, which is effected by the initial stiffness and the failure displacement. The addition of GWB generally decreased the failure displacement, but it also stiffened the wall, which decreased the yield displacement as well.

Hysteretic Energy (walls with hold-down connections)

Single sided walls

An interesting and informative method to display the hysteretic energy dissipation capabilities of the walls is to plot the equivalent viscous energy dissipation factor against racking displacement. The comparison for all wall types with hold-down connections and the initial and stabilized response is shown in Figure 34. OSB and hardboard sheathed walls dissipated the largest amount of energy per cycle. Both materials behaved the same until a displacement of 38 mm (1.5 in). At this displacement, the hysteretic energy of the hardboard walls leveled off, but the OSB walls were able to dissipate much more energy. When considering peak load, hardboard walls were slightly stronger. However, as shown in Figure 34, OSB walls were much tougher, meaning that they were able to dissipate more energy at a given displacement.

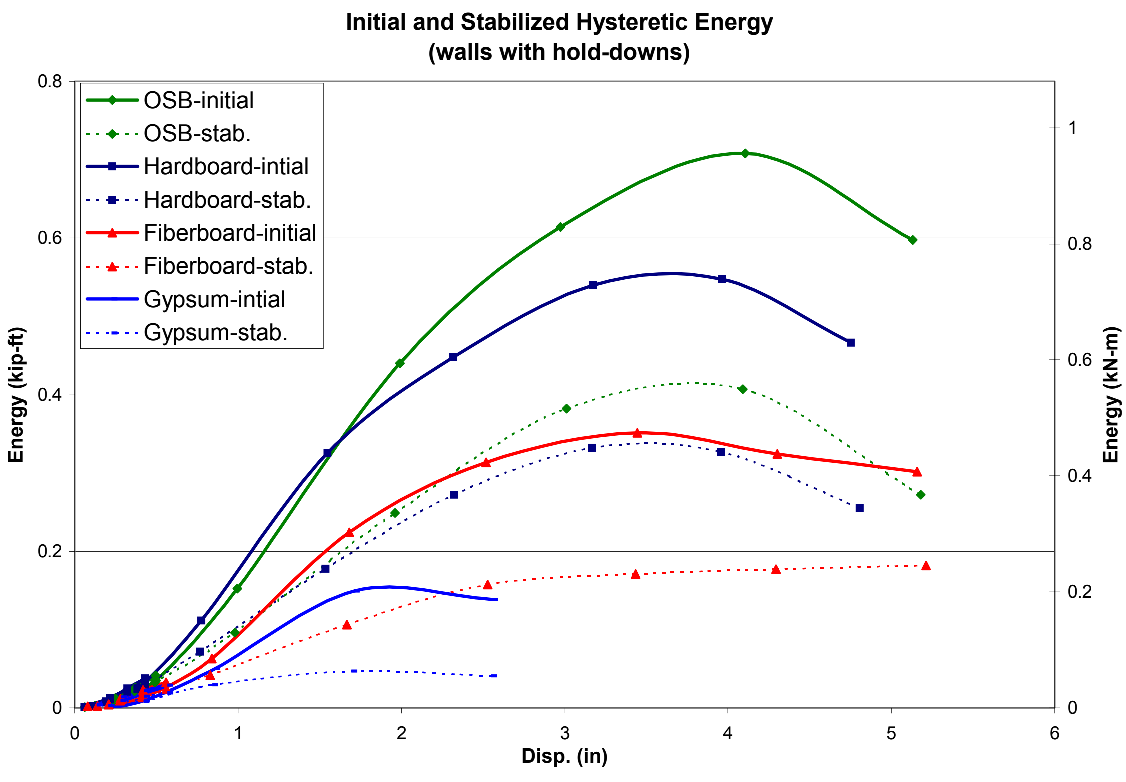

Figure 34: Initial and Stabilized Hysteretic Energy versus Displacement (walls with hold-down connections)

This is related to the different failure mechanisms for the two walls. The reduction in energy dissipated in the hardboard sheathed walls can be attributed to nail behavior. Nails were observed to withdraw from the framing, reducing the friction between the framing and the sheathing. Larger sheathing nails in the OSB walls did not withdraw as easily, which enabled the friction to continue to dissipate energy at a larger displacement.

The toughness of fiberboard sheathed walls was less than OSB and hardboard sheathed walls. The nails used to connect the fiberboard and GWB sheathing tore through the sheathing, which restricted the amount of energy that could be dissipated. The nailing schedule of the fiberboard walls was more dense than GWB walls, which has a direct influence on the energy dissipating capacity.

Reduced hysteretic energy between the initial and stabilized cycles was also observed. following the same trend as the strength, the reduction associated with stabilized cycles was about 35%. Considering that the displacement level was the same for initial and stabilized cycles, the reduction in strength will directly effect the amount of energy dissipated, which is the area contained within the load-displacement curve.

<u>Double sided walls</u>

The effect of GWB on the hysteretic energy of the walls is illustrated in Figure 35. Hysteretic energy increases when GWB is included. The increase is most notable at the peak load and beyond, where the energy dissipating capabilities are crucial. However, when tested alone, the hysteretic energy of GWB was larger than when combined with other sheathing materials. Therefore, the GWB is not linearly additive in regards to hysteretic energy dissipation.

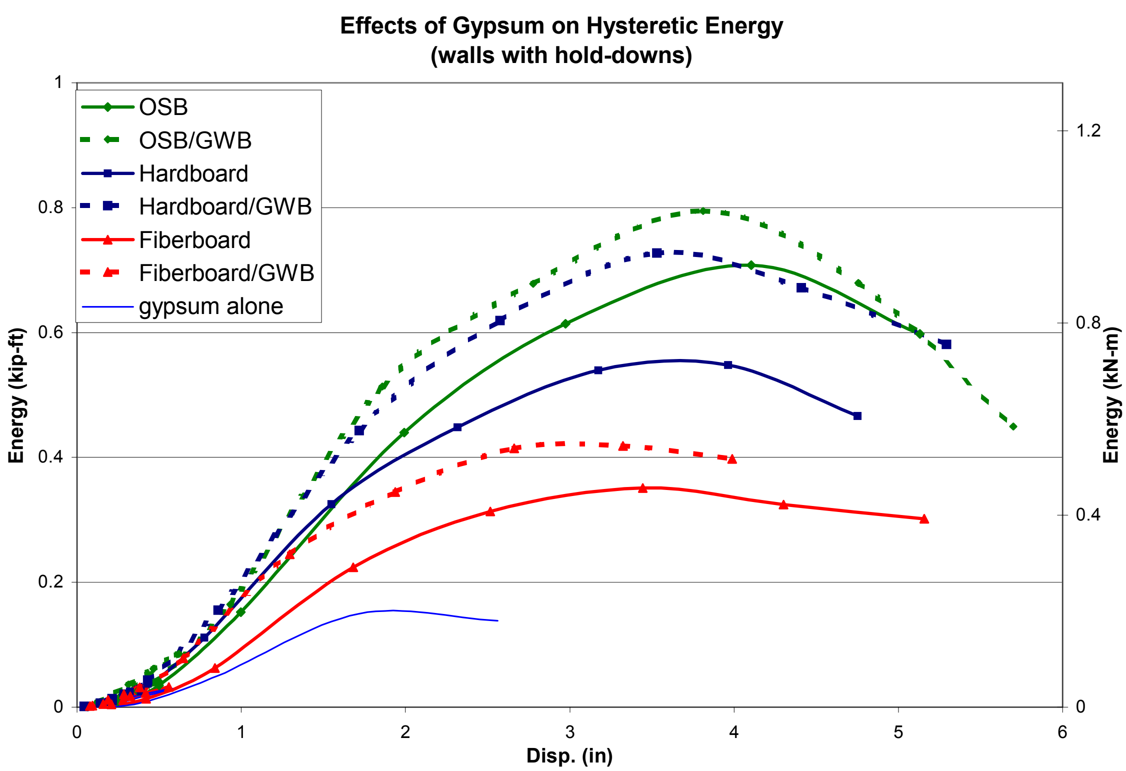

Figure 35: Contribution of gypsum to hysteretic energy (walls with hold-down connections)

Equivalent Viscous Damping (walls with hold-down connections)

Single sided walls

By plotting the equivalent viscous damping ratio (EVDR) with respect to racking displacement for the walls, similar information about the wall performance can be deduced which attempts to include the effect of strain energy stored in the wall. As shown in Figures 36 and 37, the EVDR was the smallest during the elastic cycles. During the initial cycles, the EVDT increased until a displacement of about 44.5 m (1.75 in). After reaching peak load, damping progressively decreased until failure. During the stabilized cycles, the EVDR was the smallest during elastic cycles then increased to a maximum at 44.5 mm (1.75 in), but maintained at that level until failure. OSB walls displayed the largest damping ratio.

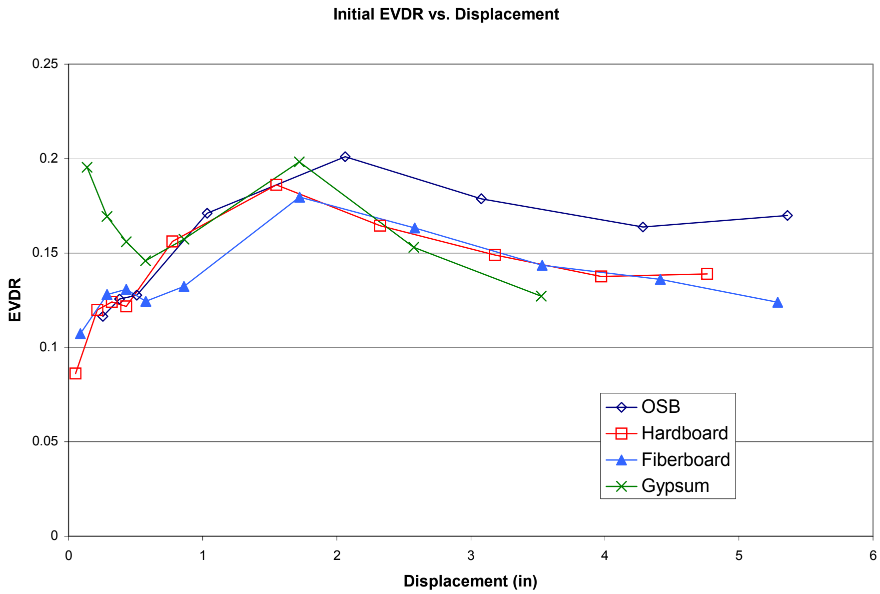

Figure 36: Initial EVDR of sheathing panels

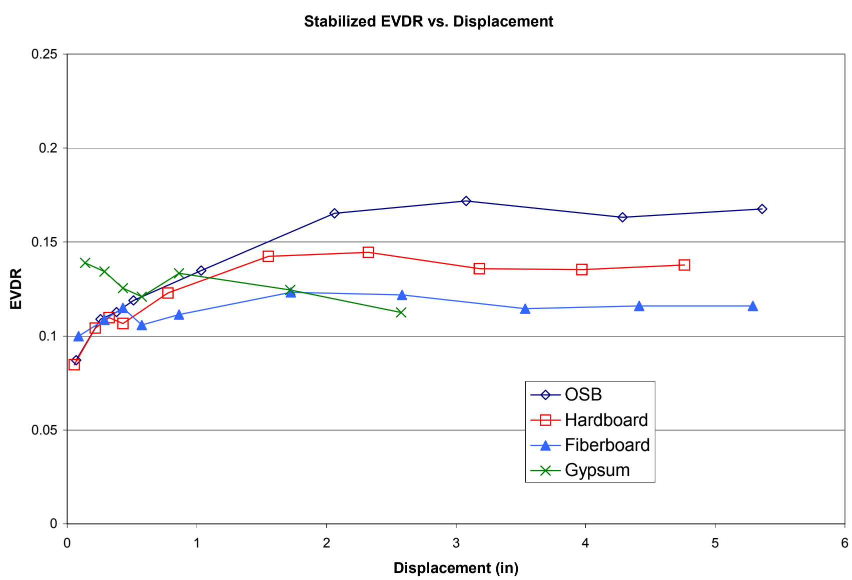

Figure 37: Stabilized EVDR of sheathing panels

Double sided walls

The EVDR is relatively unaffected by the addition of GWB, as shown in Figure 38. The walls with GWB follow the same curve as the walls without GWB. The reason is that when GWB is included, the increase in energy from the hysteresis look is proportional to the increase in the strain energy, which results in no change to the damping ratio.

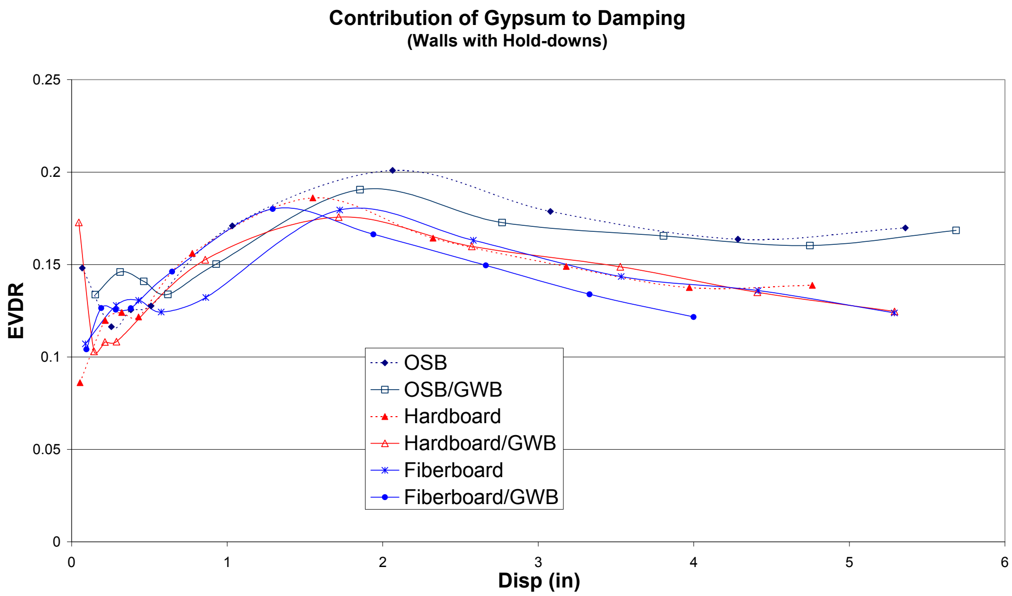

Figure 38: Effect of gypsum on damping (walls with hold-down connections)

Cyclic Stiffness (walls with hold-down connections)

Cyclic stiffness can be calculated during each cycle of a test as described earlier. When plotted against displacement, it can serve as an indication of the stiffness degradation of the structure subjected to cyclic loading. Cyclic stiffness degradation is a result of the sheathing-to-frame connection becoming distorted. In some cases, the nails tore through the sheathing, while other times the nails withdrew from the framing. Regardless of the nail behavior, the continuously reversed cycles caused the stiffness to decrease with each loading phase. This occurred because after every cycle, the nail would tear a larger hole, or withdraw from the framing a little more. Therefore, the next cycle would have to be displaced that additional amount before the structure could resist more load, resulting in a reduction in the stiffness of the wall. The initial stiffness was large, but the stiffness degradation was exponential and eventually approached zero. It can easily be seen that the stiffness degradation does not correspond to the strength degradation because the stiffness decreases while the strength still increases. Due to the reduction of the stiffness, the strength increase is not as rapid as the displacement increase, which gives the nonlinear response of the wall.

The stiffness degradation of the initial and stabilized cycles are shown in Figures 39 and 40, respectively. All the sheathing materials behave in the same general behavior. OSB and hardboard panels degrade at the same rate and remain stiffer than the fiberboard and gypsum. The difference in the initial and stabilized stiffness is small. This occurs because the strength is slightly reduced during the stabilization cycles, but the displacement level is the same for both.

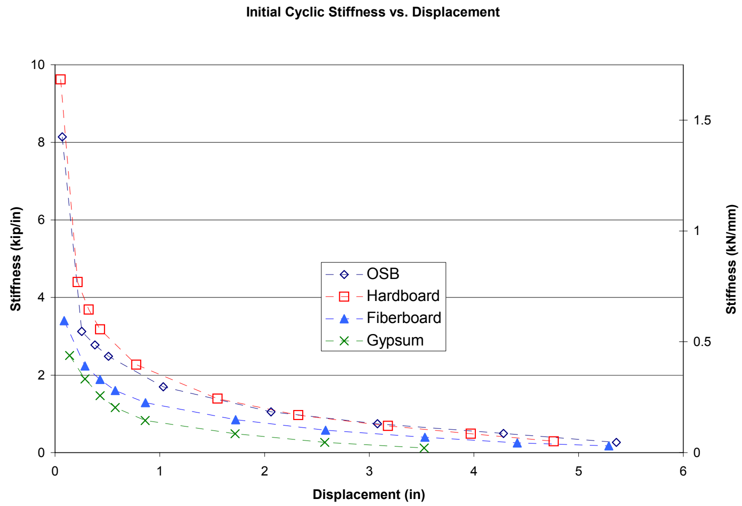

Figure 39: Initial Cyclic Stiffness (walls with hold-down connections)

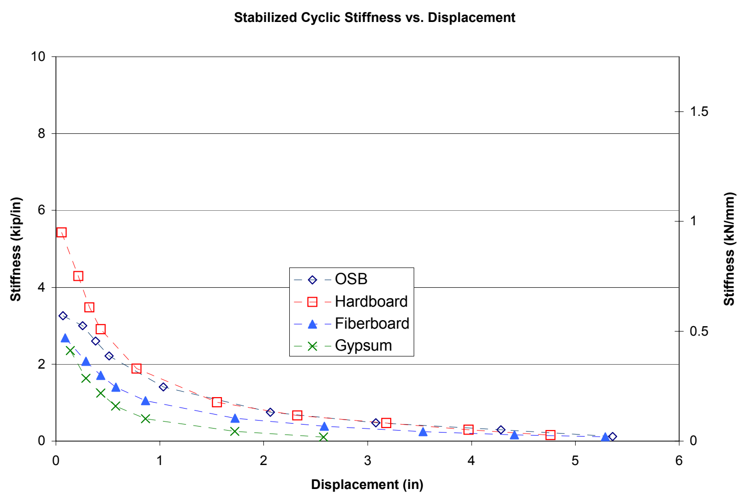

Figure 40: Stabilized Cyclic Stiffness (walls with hold-down connections)

<u>Single sided walls</u>

The stiffness degradation of the initial and stabilized cycles are shown in Figures 39 and 40, respectively. All walls exhibited the same general behavior. OSB and hardboard sheathed walls degraded at the same rate and remained stiffer than the fiberboard and GWB sheathed walls. The difference in the initial and stabilized stiffness is small.

<u>Double sided walls</u>

The cyclic stiffness when GWB is included with the OSB sheathing is presented in Figure 41. The addition of GWB increases the cyclic stiffness by a substantial amount when the displacements are small. At larger displacements, typically 76 mm (3 in) or greater, the contribution of GWB was zero, the two curves were equal. The GWB would always fail first, so at larger displacements, it could not contribute to the performance of the walls.

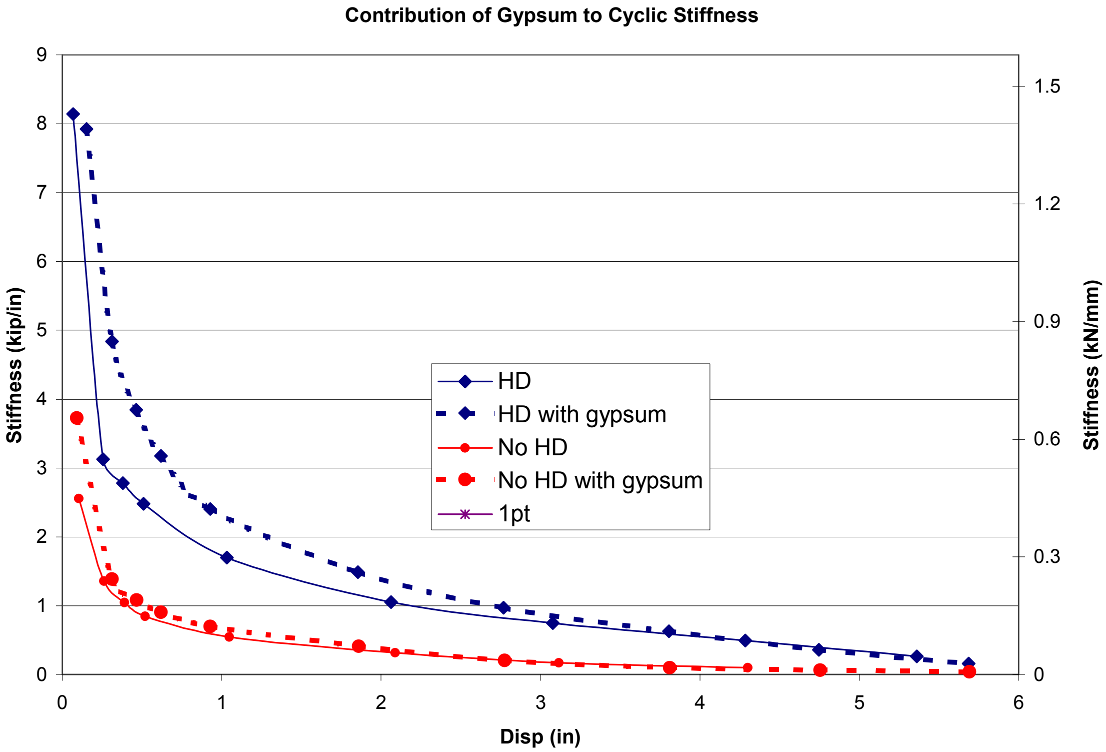

Figure 41: Contribution of Gypsum to Cyclic Stiffness

Wall Behavior and Mode of Failure (walls with hold-down connections)

Single sides and double sided walls

Modes of failure in the cyclic tests were consistent with those observed in monotonic tests. Examples of some the failure modes observed are shown in Figures 42 – 46.

Figure 42: Sheathing nails unzipping along bottom plate of OSB

Figure 43: Nail pullout in hardboard panels

Figure 44: Nail tore-through in fiberboard panels

Figure 45: Gypsum nail rip-through along end stud

Figure 46: Gypsum nail rip-through along end stud

Comparison of Sheathing Materials Without Hold-downs

Average test results are provided in Table 12. Complete results of the cyclic tests can be found in Appendix B and Appendix C. The individual specimen results of the cyclic tests with hold-down connections are given in Appendix B, while the individual specimen results of the cyclic tests without hold-down connections are given in Appendix C. In Appendices B and C, a description of the test observations is given for each test, along with the mode of failure. A listing of every desired shear wall parameter is given in tabular form. Load-displacement curves are also provided in Appendices B and C. Figure 47 depicts a typical cyclic test load-displacement curve for a test of a wall without hold-down connections.

Peak Load (walls without hold-down connections)

<u>Single sided walls</u>

As shown in Figure 48, OSB, hardboard, and fiberboard sheathed walls reached nearly the same average peak load of 3.2 kN (0.72 kips). The walls without hold-down connections were observed to fail at a lower displacement value resulting from the nails unzipping along the bottom plate. The GWB wall reached a peak load of 1.96 kN (0.44 kips). The strength of the stabilized cycles typically ranged from 70 – 75% of the initial cycles.

<u>Double sided walls</u>

The initial envelop curves for cyclic test of walls without hold-down connections are shown in Figure 49. The peak load of the double sided walls without hold-down connections are compared to walls sheathed with one material to determine the contribution of GWB to the peak load of the wall. If the assumption of linear addition of peak loads is made for the addition of GWB to the other three materials, the predicted load is shown in Table 13. The actual peak loads for the three configurations tested are also included in Table 13, long with the ratio of the two loads. If values shown in Table 12 are considered, the GWB adds almost nothing to the peak load of the walls when hold-down connections are not used.

Table 12: Average results from cyclic wall tests (walls without hold-down connections).

Wall Type and Cycle	Peak Load (kN)	Peak Load (kip)	Δ_{Peak} (mm)	Δ_{Peak} (in.)	Failure Load (kN)	Failure Load (kip)	$\Delta_{Failure}$ (mm)	$\Delta_{Failure}$ (in.)	Yield Load (kN)	Yield Load (kip)	Δ_{Yield} (mm)	Δ_{Yield} (in.)	Ductility Ratio μ	Energy Dissipation (kN-m)	Energy Dissipation (kip-ft)
Single Sided Walls															
OSB															
Initial	3.12	0.70	46	1.82	2.50	0.56	83	3.28	3.99	0.90	10	.38	8.3	3.13	2.32
Stabilized	2.25	0.51	33	1.28	1.80	0.41	55	2.16	1.69	0.38	4	.18	13.8	3.00	2.21
Hardboard															
Initial	3.28	0.74	44	1.75	2.62	0.59	79	3.11	2.70	0.61	12	0.46	7.7	3.53	2.60
Stabilized	2.39	0.54	32	1.27	1.91	0.43	55	2.17	1.59	0.36	5	0.20	10.4	3.82	2.82
Fiberboard															
Initial	3.15	0.71	38	1.48	2.52	0.57	58	2.28	2.66	0.60	11	0.45	19.5	2.11	1.56
Stabilized	2.21	0.50	30	1.20	1.77	0.40	39	1.54	1.66	0.37	2	0.08	29.3	3.34	1.73
GWB															
Initial	1.94	0.44	36	1.42	1.69	0.36	54	2.11	1.70	0.38	8	0.31	6.8	1.52	1.12
Stabilized	1.47	0.33	21	0.82	1.17	0.26	23	0.89	1.24	0.28	5	0.18	5.8	0.09	0.07
Double Sided Walls															
OSB/GWB															
Initial	3.40	0.77	47	1.85	2.72	0.61	67	2.65	2.87	0.65	8	0.31	8.4	1.72	1.28
Stabilized	2.27	0.51	20	0.77	1.82	0.41	36	1.40	1.61	0.36	1	0.05	36	1.44	1.06
Hardboard/GWB															
Initial	3.68	0.83	38	1.50	2.94	0.66	65	2.58	3.25	0.73	8	0.31	9.2	2.20	1.62
Stabilized	2.49	0.56	24	0.95	2.00	0.45	49	1.95	2.21	0.50	5	0.20	9.8	2.37	1.75
Fiberboard/GWB															
Initial	3.04	0.68	33	1.29	2.43	0.55	54	2.14	2.69	0.61	7	0.28	7.7	2.02	1.49
Stabilized	2.15	0.48	16	0.64	1.72	0.39	41	1.62	1.74	0.39	5	0.20	8.2	1.80	1.33

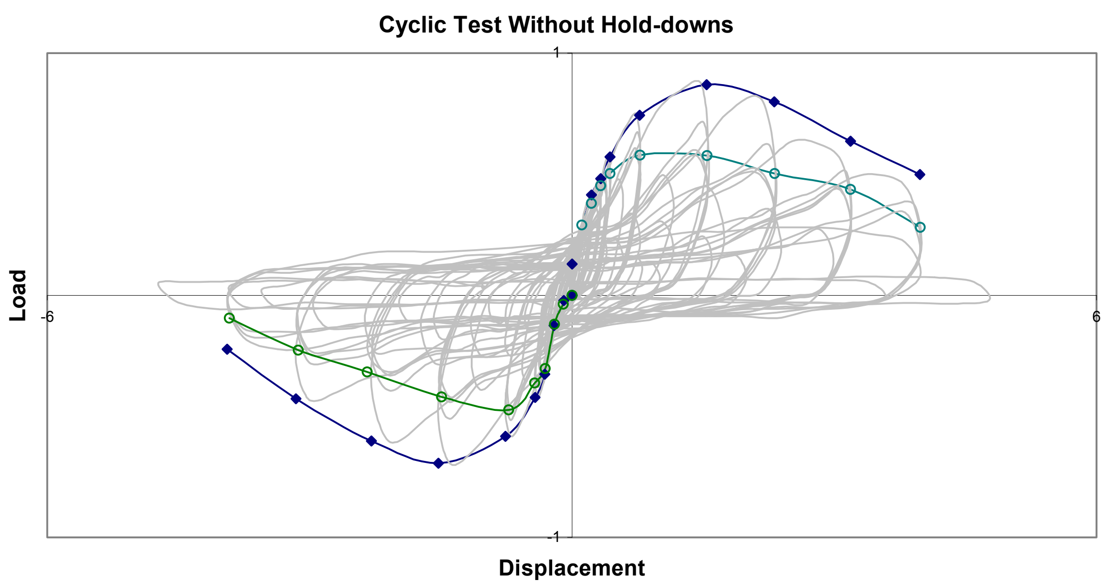

Figure 47: Typical load-displacement graph of cyclic tests without hold-downs

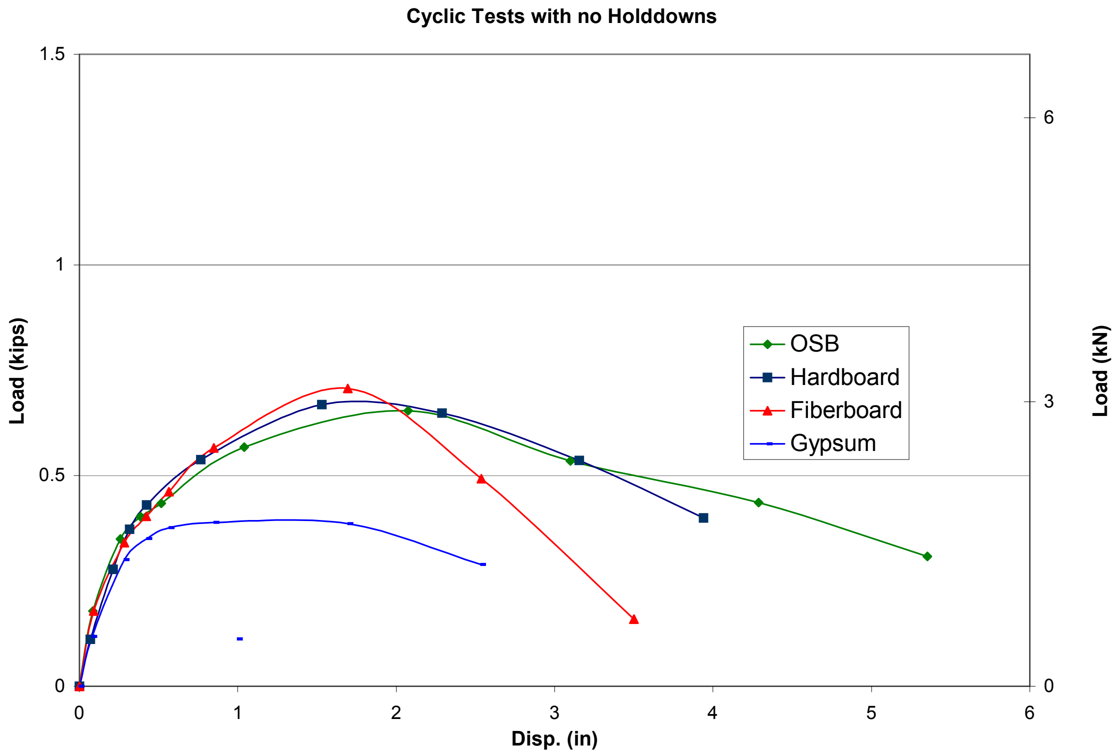

Figure 48: Average initial envelope curve of tests without hold-downs

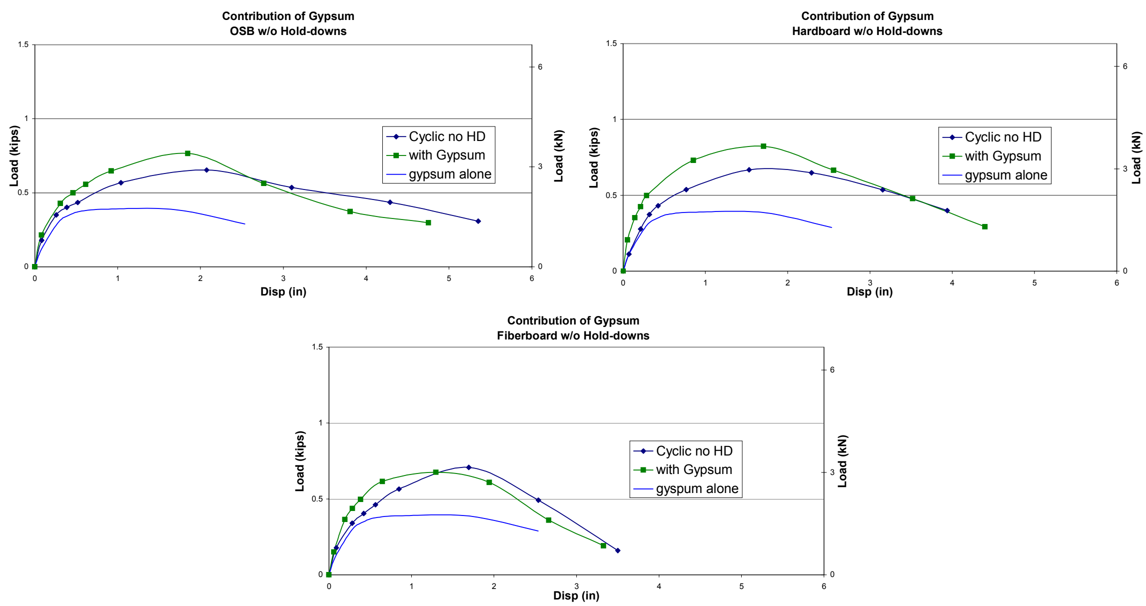

Figure 49: Contribution of gypsum to cyclic tests without hold-downs

Table 13: Comparison of walls with and without GWB (cyclic tests without hold-down connections.)

Wall Type	Predicted Load (kip) (1)	Actual Load (kip) (2)	Ratio (2)/(1)
OSB/GWB	5.06	3.40	0.67
Hardboard/GWB	5.22	3.68	0.70
Fiberboard/GWB	5.09	3.04	0.62

Elastic Stiffness (wall without hold-down connections)

Single sided walls

The elastic stiffness of the hardboard, fiberboard, and GWB sheathed walls were similar at average values in the range of 0.22 kN/mm (1.3 k/in). The OSB sheathed walls had an average elastic stiffness almost double the value of the other sheathing materials, with a value of 0.41 kN/mm (2.34 k/in). The stabilized elastic stiffness values were erratic due to the definition of yield and elastic stiffness. The values ranged from 0.27 to 4.21 kN/mm (1.55 to 24.07 k/in).

Double sided walls

The combination of GWB with hardboard and fiberboard increased the average elastic stiffness of these walls to values almost double the single sided wall values. Hardboard/GWB and fiberboard/GWB walls had average elastic stiffness of 0.43 and 0.38 kN/mm (2.47 and 2.16 k/in), respectively. The combination of OSB and GWB resulted in a reduction in average elastic stiffness, with a value of 3.4 kN/mm (1.95 k/in). The stabilized elastic stiffness values were erratic, similar to the single sided wall values.

Yield Load and Displacement (walls without hold-down connections)

Single sided walls

Average yield load values for single sided walls are shown in Table 12. The values can be divided into three groups with the OSB sheathed walls having the highest values (3.99 kN (0.90 kips)). Hardboard and fiberboard sheathed walls can be grouped together with values in the 2.70 kN (0.6 kip) range, and GWB had the lowest value at 1.70 kN (0.38 kip). The stabilized yield load could be grouped into two groups with the OSB, hardboard, and fiberboard sheathed walls having similar average values in the 1.59 – 1.66 kN (0.36 – 0.37 kip) range, and GWB having an average value of 1.24 kN (0.28 kip).

Associated yield displacements all were in the 8 – 12 mm (0.31 – 0.46 in) range. For practical ruse, the walls could be considered to all have equivalent yield displacements. Stabilized yield displacement values were all small (<0.18 in) and erratic.

Double sided walls

Double sided walls had a different grouping for the yield load. The OSB/GWB and Fiberboard/GWB sheathed walls could be grouped together with yield load values in the 2.69 – 2.87 kN (0.61 – 0.65 kip) range. The hardboard/GWB sheathed wall configuration had the highest average yield load value of 3.25 kN (0.73 kip). The average stabilized yield load values for the OSB/GWB was 1.61 kN (0.36 kip), the hardboard/GWB walls had a value of 2.21 kN (0.50 kip), and the fiberboard/GWB sheathed walls have a value of 1.74 kN (0.39 kip).

Average initial cycle yield displacement values for the double sided walls all were in the 7 – 8 mm (0.28 – 0.33 in). The stabilized yield displacement for the OSB/GWB configuration was 1 mm (0.05 in), while the rest of the double sheathed walls had average values of 5 mm (0.20 in).

Failure Load and Displacement (walls without hold-down connections)

Single sided walls

Average peak load and failure load for OSB, hardboard, and fiberboard sheathed walls were similar. Displacement at failure was different OSB walls failed at 83 mm (3.28 in) and hardboard walls failed at 79 mm (3.11 in). Fiberboard sheathed walls reached the same peak load as OSB and hardboard sheathed walls, but the displacement at failure was smaller. After reaching peak load, the fiberboard sheathed wall exhibited an increased rate of reduction in strength likely due to tear through of the sheathing nails. Displacement at peak load was 38 mm (1.5 in), while the wall failed at a displacement of only 58 mm (2.28 in).

Double sided walls

Displacement at failure when using GWB typically decreased when compared to its corresponding wall without GWB. As shown in Table 12, the average failure displacement of the walls tested decreased by 15 mm (0.59 in) when GWB was included. After reaching peak load, the envelope curves tended to decrease as a faster rate when GWB was included, which lead to a reduced failure displacement. This tends to be a characteristic behavior when GWB is used with the other sheathing materials tested in this study.

Ductility (walls without hold-down connections)

Single sided walls

Average ductility ratios for the walls tested are presented in Table 12. Ductility values ranged from 6.8 for GWB sheathed walls to 19.5 for fiberboard sheathed walls (based on the initial cycle envelop curves. OSB and hardboard sheathed walls had average ductility factors close to that determined for GWB (8.3 and 7.7 respectively.) Stabilized ductility factors were very erratic and ranged from 5.8 to 156.8. The stabilized ductility factor values illustrate the problem with the definition of ductility factor in that the yield displacement is not clearly defined or consistent.

Double sided walls

Double sided wall results show that the initial cycle ductility for walls with GWB are remarkably consistent, with values ranging from 7.7 to 9.2. The stabilized values were similar magnitudes, with the value for OSB. Being the one inconsistent value due to unknown reasons.

Hysteretic Energy Dissipation (walls without hold-down connections)

Single sided walls

The energy dissipated during the initial and stabilized cycle of each phase is displayed in Figure 50. OSB, hardboard, and fiberboard sheathed walls all exhibit similar energy dissipating capability until a displacement level of 63.5 mm (2.5 in). At this displacement, the hysteretic energy of fiberboard sheathed walls quickly reduced. The sharp reduction observed in fiberboard sheathed walls results from the nails tearing through the sheathing along the bottom plate. As observed in walls with hold-down connections, there was also a reduction in the hysteretic energy of the initial and stabilized cycles for walls without hold-down connections (approximately 35%). Considering that the displacement level was the same for initial and stabilized cycles, the reduction in strength will directly effect the amount of energy dissipated, which is the area contained by the load-displacement curve.

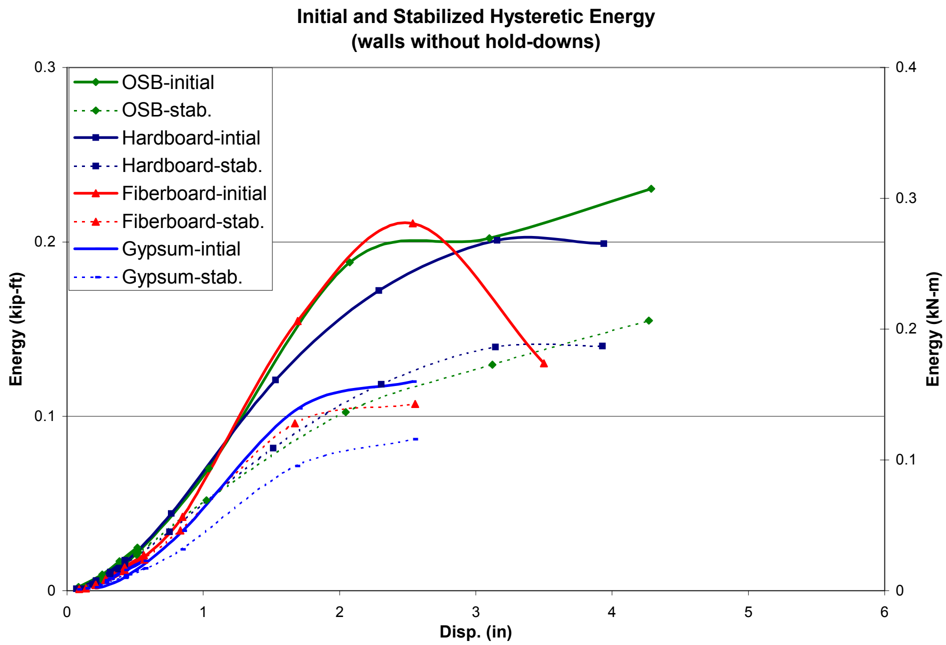

Figure 50: Hysteretic energy versus displacement for walls without hold-downs

<u>Double sided walls</u>

The hysteretic energy dissipation characteristics for double sided walls without hold-down connections is presented in Figure 51. The hysteretic energy is increased slightly for the hardboard/GWB sheathed wall compared to the hardboard single sided wall. For other cases of GWB, the addition of GWB does not increase the hysteretic energy.

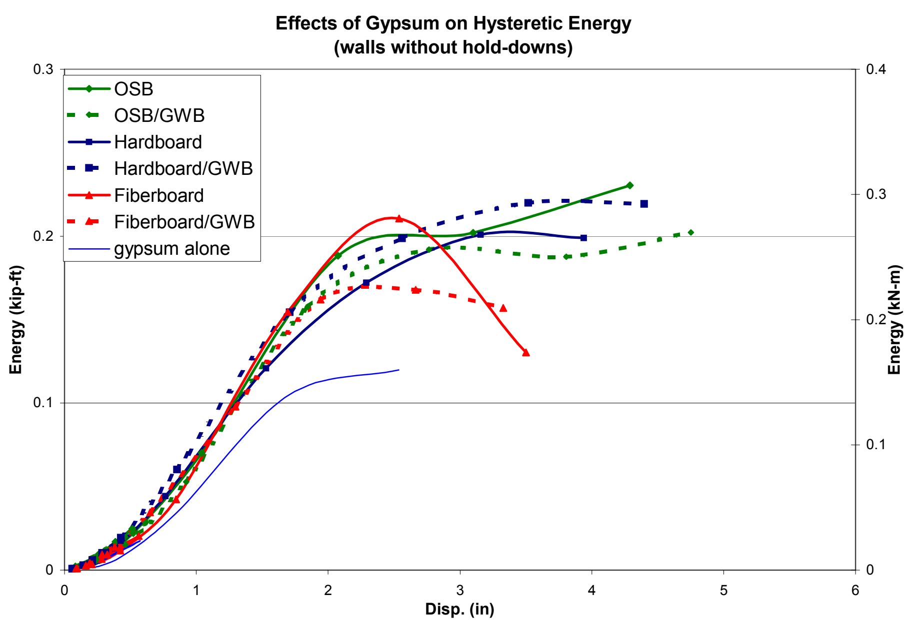

Figure 51: Contribution of gypsum to hysteretic energy in walls without hold-downs

Equivalent Viscous Damping (walls without hold-down connections)

Single sided walls

The EVDR is plotted with respect to racking displacement for the single sided walls tested in Figure 52. The EVDR is smallest during the elastic cycles. The EVDR was random and did not follow a definite pattern. OSB and hardboard sheathed walls exhibited similar values with a relatively constant EVDR throughout all displacement levels. The EVDR for fiberboard and GWB sheathed walls was small when the wall was still elastic. As the displacement levels increased, so did the damping ratio. When the hysteresis loops of these walls were examined, it was discovered that the loops did not follow the common shape shown in Figure 47. Instead of gradually increasing, the hysteresis loop was much more circular, which increased the damping ratio by increasing hysteretic energy.

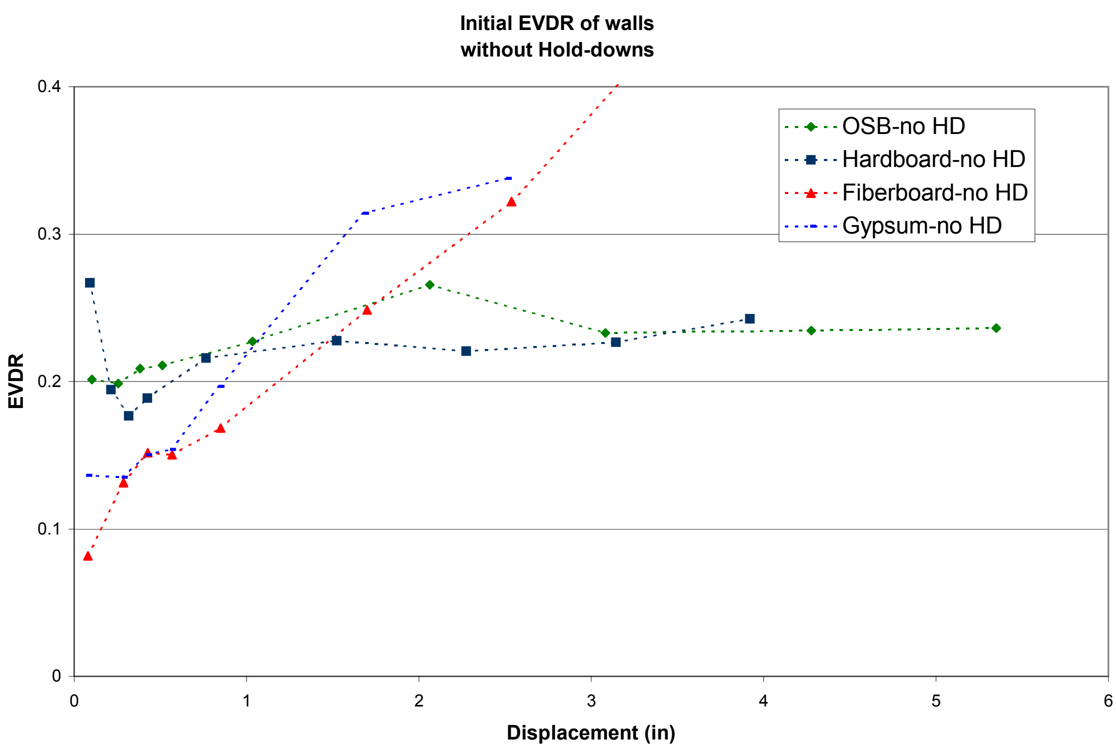

Figure 52: EVDR of walls without hold-downs

Double sided walls

The EVDR is relatively unaffected by the addition of GWB, as shown in Figure 53. The walls with GWB follow the same curve as the walls without GWB. The reason is that when GWB is included, the increase in energy from the hysteresis loop is proportional to the increase in the strain energy, which results in no change to the damping ratio. A direct relationship between the viscous damping ratio and the GWB cannot be made for walls without hold-down connections. At low displacement levels, the damping ratio is lower when GWB is included, but as the displacement levels increase, so does the damping ratio of the GWB walls.

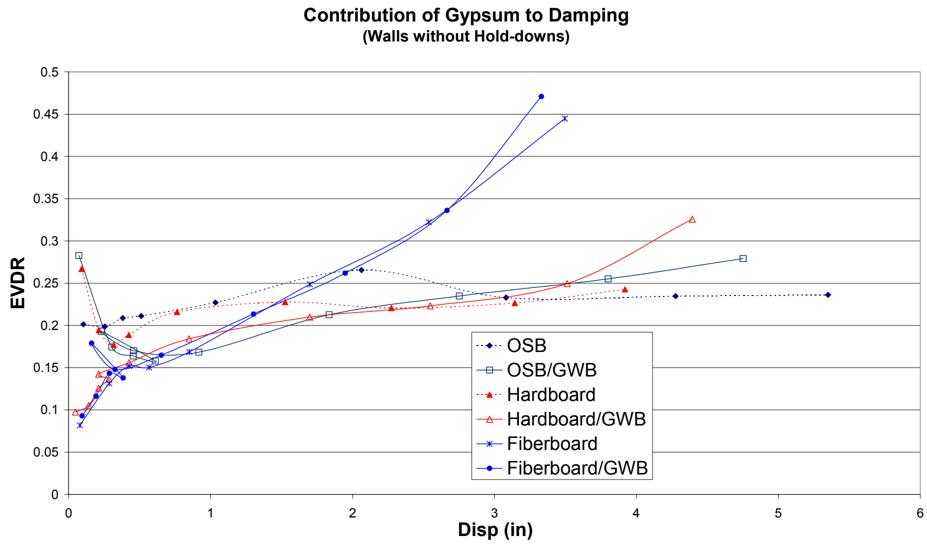

Figure 53: EVDR of walls without hold-downs

Cyclic Stiffness (walls without hold-down connections)

Single sided walls

The cyclic stiffness degradation of the initial cycles is plotted with respect to racking displacement in Figure 54 for walls without hold-down connections. All of the walls behaved similarly. OSB, hardboard, and fiberboard sheathed walls degrade similarly throughout the tests. Since the load displacement graphs were similar for these walls, it confirms that the cyclic stiffness will also be similar. The cyclic stiffness of GWB walls was slightly less than the other walls. Although not shown, the cyclic stiffness of the stabilized cycles was slightly smaller than the initial cycles. The reduction was similar to that observed for walls with hold-downs.

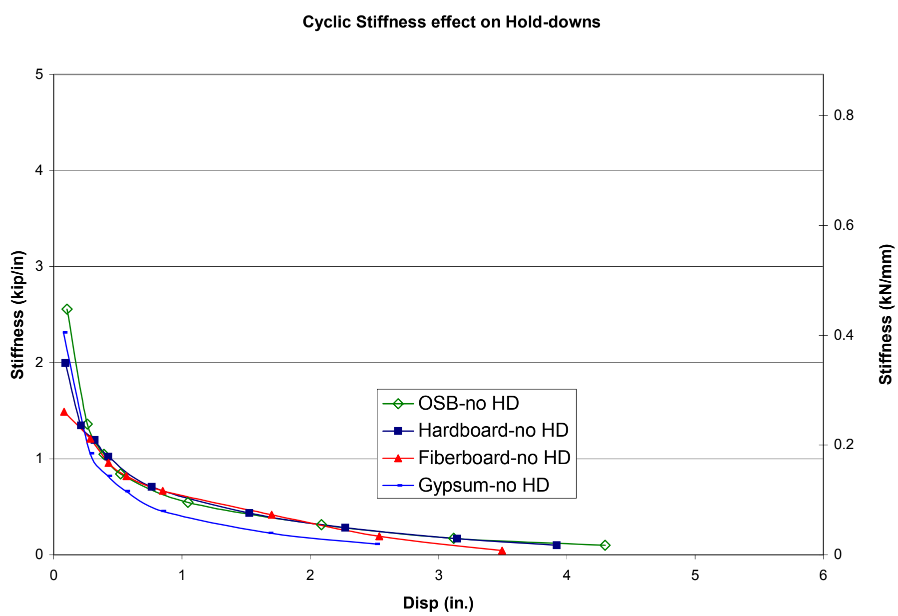

Figure 54: Cyclic Stiffness of walls without hold-downs

Double sided walls

The relationship for cyclic stiffness of walls without hold-down connections for when GWB is included is illustrated in Figure 55. The addition of GWB did not contribute to the cyclic stiffness of any of the walls tested without hold-down connections. The curves are almost identical at all displacement levels.

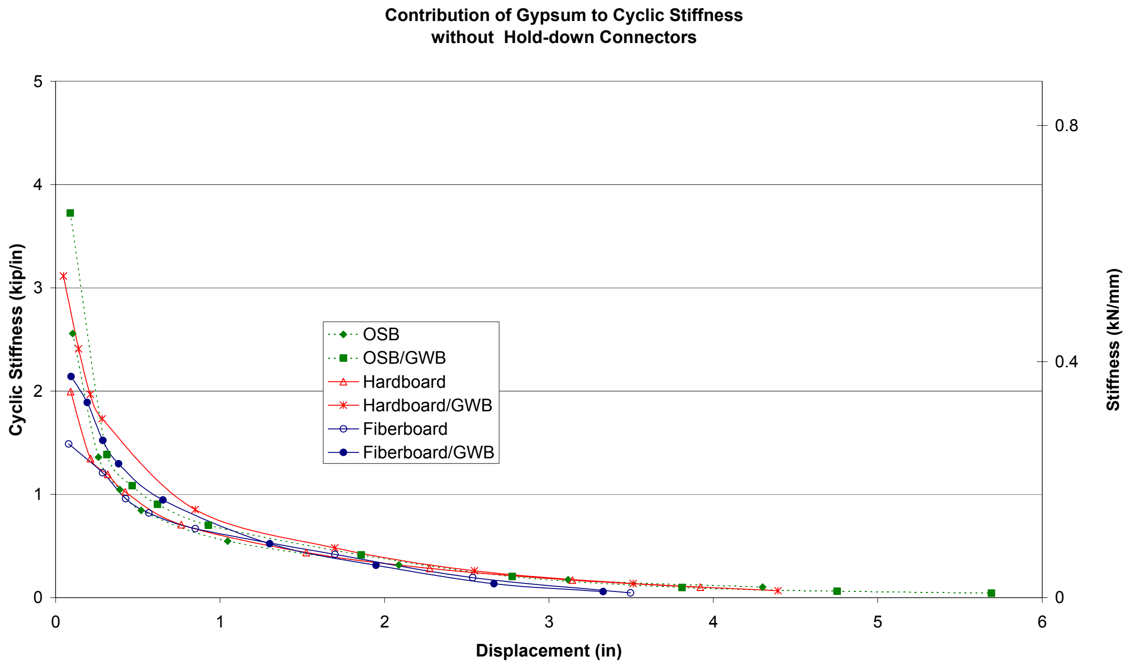

Figure 55: Effect of GWB on Cyclic Stiffness When Combined With Other Sheathing Materials (walls without hold-down connections)

Wall Behavior and Failure Mode (walls without hold-down connections)

Single and double sided walls

All of the walls without hold-downs failed along the bottom plate. The predominant failure modes of walls without hold-downs are illustrated in Figures 56 – 59. After the nails unzipped along the bottom plate, the sheathing panels were no longer effective in resisting shear. As the cyclic tests proceeded after unzipping, the wall was observed to separate from the bottom plate. There were two ways that the wall separated from the bottom plate. The most common way was the end stud pulling away from the bottom plate (Figures 56 and 57). Fiberboard and gypsum walls experienced this type of failure every time. The other method of failure occurred from the bottom plate splitting parallel to the grain (Figure 58) or fracturing due to weak axis bending at the location of the shear anchor bolt (Figure 59). In some instances, there was observed to be a knot, or imperfection in the wood at the point of fracture. About half of the OSB and hardboard walls experienced bottom plate damage at relatively large displacements and after the onset of yielding of the sheathing nails.


Figure 56: Wall separated from bottom plate after failure


Figure 57: Nail pull through exclusively on bottom plate (fiberboard)


Figure 58: Bottom plate splitting parallel to grain

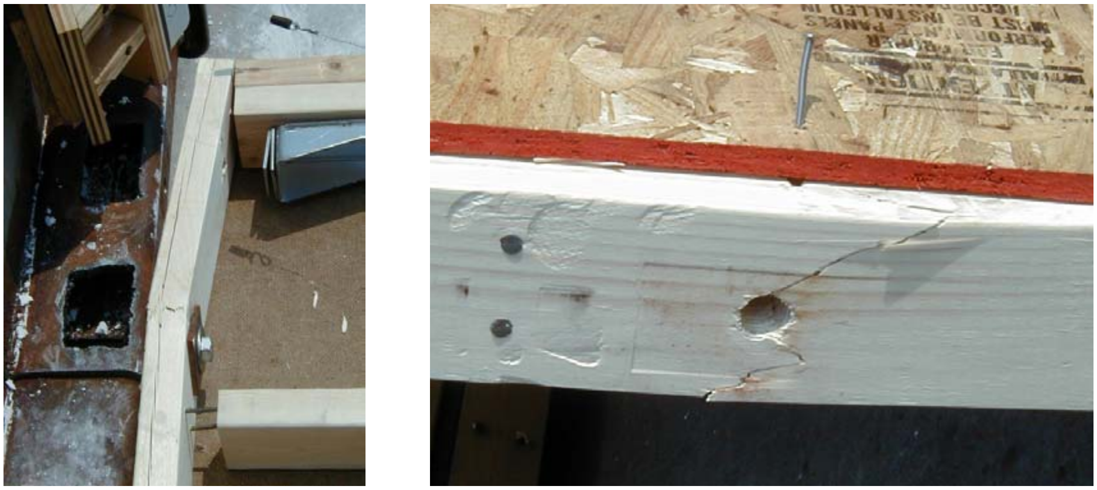

Figure 59: Bottom plate fracture at anchor bolt location

Summary Comparison of Cyclic Tests of Walls With and Without Hold-Down Connections

Peak Load

The ratios of peak load for walls without and with hold-down connections are shown in Table 14, and load-displacement envelope curves are shown in Figure 60. Peak loads for gypsum and fiberboard walls were least affected by the absence of hold-down connections.

Table 14: Ratio of Peak Load for Walls With and Without Hold-Down Connections

Wall Type	**With Hold-Down Connections (1)**		**Without Hold-Down Connections (2)**		Ratio (2)/(1)
	(kN)	**(kips)**	**(kN)**	**(kips)**	
Single Sided Wall					
OSB	9.8	2.21	3.1	0.70	0.32
Hardboard	10.0	2.25	3.3	0.74	0.33
Fiberboard	6.5	1.46	3.1	0.71	0.49
Gypsum	3.7	0.84	1.9	0.44	0.52
Double Sided Wall					
OSB/GWB	12.3	2.77	3.4	0.77	0.28
Hardboard/GWB	12.8	2.87	3.7	0.83	0.29
Fiberboard/GWB	9.2	2.06	3.0	0.68	0.33

Elastic Stiffness

For walls without hold-down connections, the elastic stiffness increased with the addition of GWB. The ratios of elastic stiffness for the walls tested cyclically are presented in Table 15. Walls with hold-down connections had 100 – 150% higher stiffness than walls without hold-down connections.

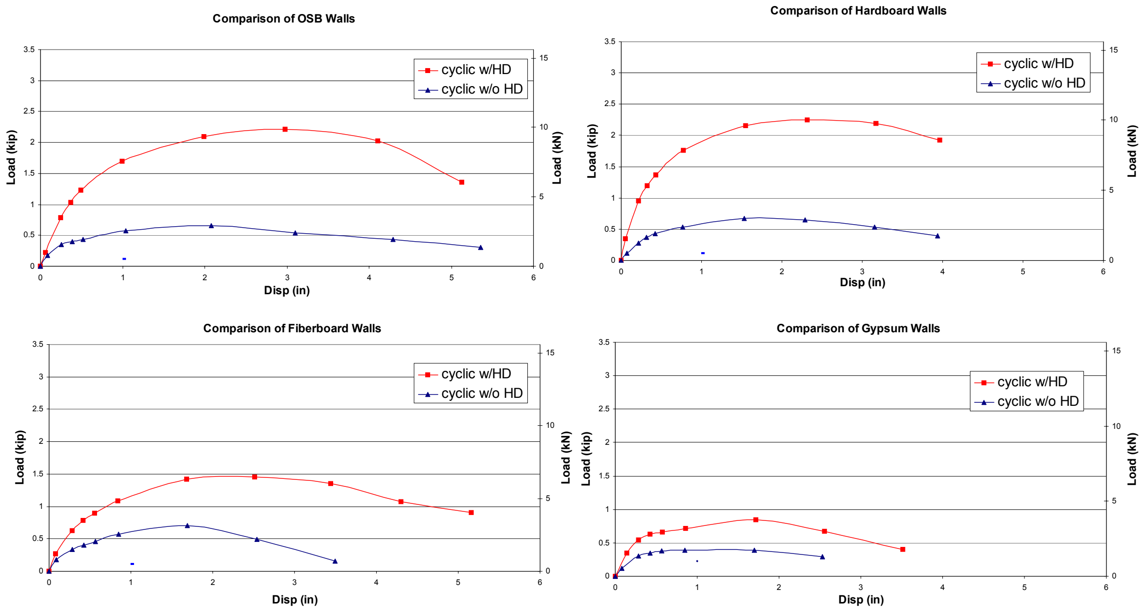

Figure 60: Load-Displacement Curves for Cyclic Tests With and Without Hold-Down Connections

Table 15: Ratio of Elastic Stiffness for Walls With and Without Hold-Down Connections

Wall Type	**With Hold-Down Connections (1)**		**Without Hold-Down Connections (2)**		**Ratio (2)/(1)**
	(kN/mm)	**(kips/in)**	**(kN/mm)**	**(kips/in)**	
Single Sided Wall					
OSB	0.59	3.37	0.27	1.57	0.47
Hardboard	0.92	5.27	0.23	1.34	0.25
Fiberboard	0.51	2.91	0.24	1.39	0.48
Gypsum	0.45	2.56	0.22	1.25	0.49
Double Sided Wall					
OSB/GWB	1.1	6.28	0.38	2.17	0.35
Hardboard/GWB	0.83	4.74	0.43	2.48	0.52
Fiberboard/GWB	0.93	5.34	0.39	2.22	0.42

Failure Load and Displacement

Failure displacement decreased when hold-downs were absent. The ratios of the failure displacement for walls tested with and without hold-down connections are presented in Table 16. The largest reduction in failure displacement occurred in the fiberboard sheathed walls, which experienced a reduction of 51 mm (2 in). During tests without hold-down connections, the sheathing nails along the bottom plate resisted both shear and overturning forces inducing failure at a smaller displacement.

Table 16: Ratio of Failure Displacement for Walls With and Without Hold-Down Connections

Wall Type	With Hold-Down Connections (1)		Without Hold-Down Connections (2)		Ratio (2)/(1)
	(kN/mm)	(kips/in)	(kN/mm)	(kips/in)	
Single Sided Wall					
OSB	116	4.58	83	3.28	0.72
Hardboard	106	4.17	76	2.98	0.71
Fiberboard	109	4.29	58	2.28	0.53
Gypsum	65	2.55	53	2.10	0.82
Double Sided Wall					
OSB/GWB	103	4.07	67	2.65	0.65
Hardboard/GWB	106	4.19	66	2.58	0.62
Fiberboard/GWB	96	3.78	54	2.14	0.57

Hysteretic Energy Dissipation

As shown in Figure 61, a large reduction in hysteretic energy dissipation is observed when walls do not have overturning restraints. The most drastic reduction occurs for OSB panels due to the ability to distribute the nail deformation to all of the perimeter nails when hold-down connections are present. The least reduction occurs for fiberboard panels, which had the lowest hysteretic energy dissipation values when tested with hold-downs. The absence of overturning restraints greatly limited the energy dissipating capacity because only the nails along the bottom plate resisted both shear and overturning forces. Peak hysteretic energy is achieved at a much lower displacement for walls without hold-downs.

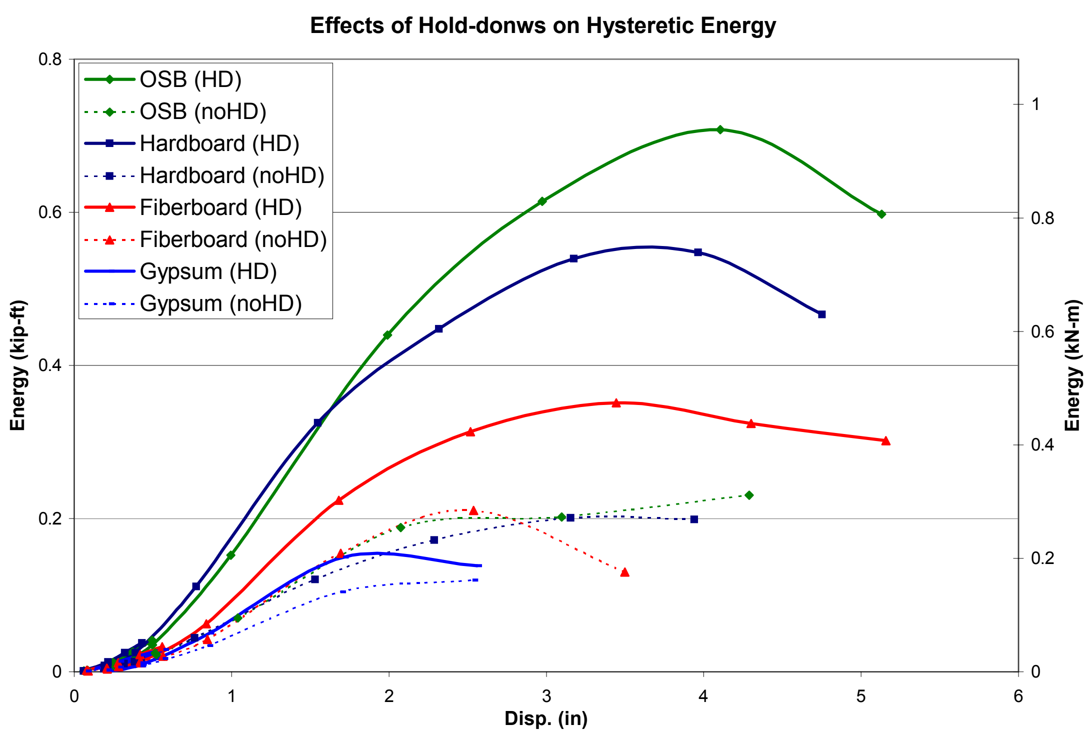

Figure 61: Effects of Hold-Downs on Hysteretic Energy Dissipation

Equivalent Viscous Damping

The equivalent viscous damping ratios for walls without hold-down connections were consistently larger than the walls with hold-downs. The reason is that the strain energy at a given interstory drift is lower when the end studs are not restrained. Without hold-down connections, the bottom of the wall could easily separate. This additional displacement can be viewed as a plastic deformation, which can store no potential energy. When hold-downs are present, the wall cannot separate from the bottom plate as easily, and behaves in an elastic manner. Before the wall begins to separate from the bottom plate, the EVDR of walls with and without hold-down connections should be similar. As shown in Figure 62, the curves are closer together during the low displacements. After the walls reach peak load, the EVDR of the walls with no hold-down connections begin to increase compared to the walls with hold-down connections. This is most evident in the walls sheathed with hardboard, fiberboard and GWB sheathed walls. The large increase in EVDR of the fiberboard occurs after peak load is reached.

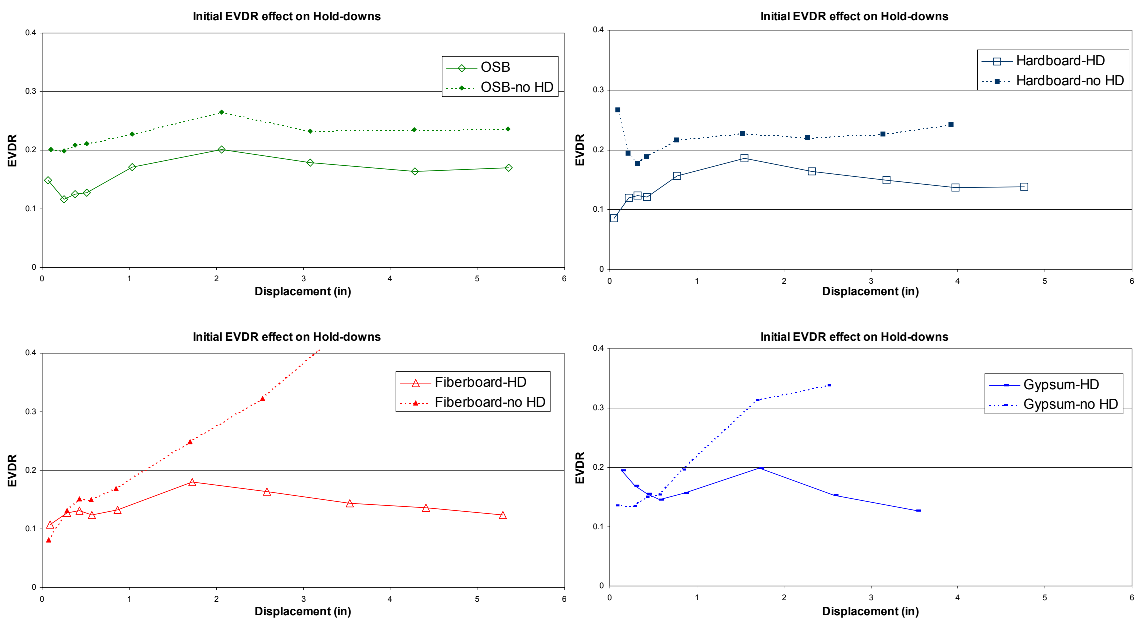

Figure 62: Effect of hold-downs on damping

Cyclic Stiffness

The effect of including hold-down connections on the cyclic stiffness of OSB and OSB/GWB sheathed walls is illustrated in Figure 63. The effect of hold-down connections on the cyclic stiffness of walls sheathed with the other materials is similar. There is a definite increase in the cyclic stiffness when hold-down connections are used. The increase becomes smaller as the displacement gets larger, and both curves approach zero exponentially. When the displacement reaches failure, the cyclic stiffness of the walls without hold-downs is minimal (e.g., close to zero), while there is still some stiffness in the walls with hold-down connections after failure.

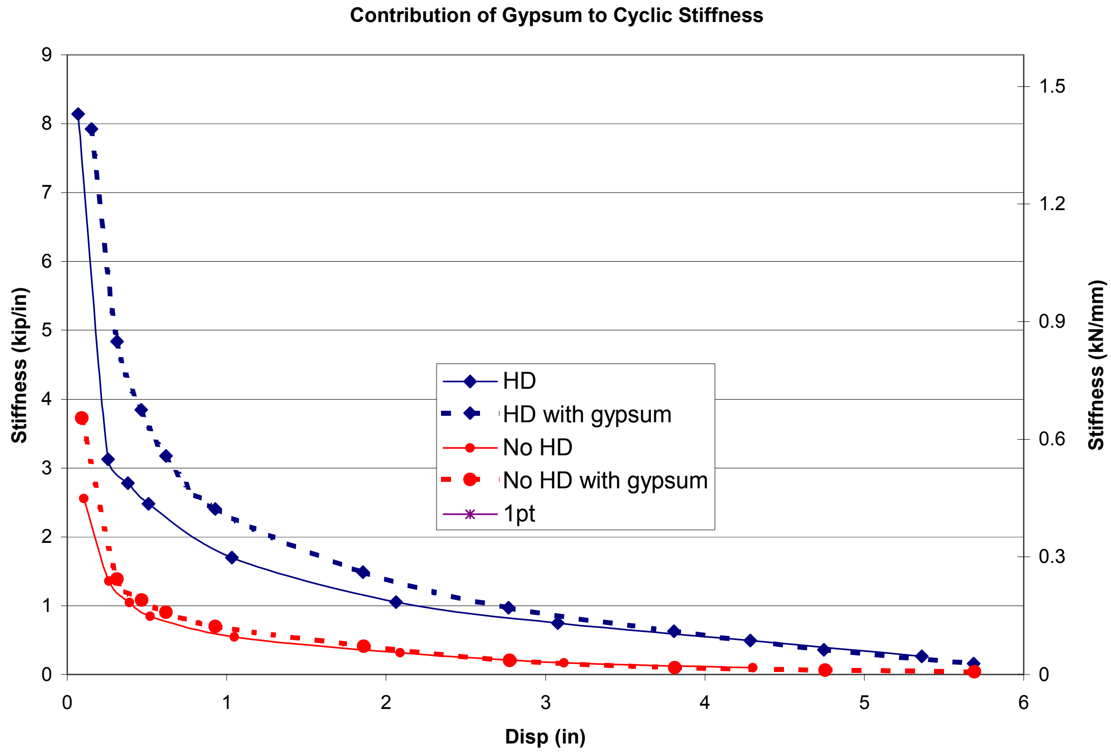

Figure 63: Contribution of Gypsum to Cyclic Stiffness

End Stud Uplift

The difference in the amount of uplift at the peak load when hold-down connections were not present was typically two to three times the amount when hold-down connections were included. The values for end stud uplift for walls tested with and without hold-down connections are presented in Table 17. It can be seen that the total uplift of the left end stud was always slightly more than the right end stud. The right end stud is the stud closest to the actuator. At peak load, the walls without hold-down connections had not experienced heavy damage and the bottom plate was still attached to the sheathing material, which helped minimize the uplift. After the peak load was achieved, the walls began to fail and pull away from the bottom plate. At this point, the uplift of walls without hold-down connections began to increase very rapidly and by failure, the uplift was an order of magnitude larger than walls with hold-down connections. Typical plots of the uplift of the end studs versus racking displacement of the wall are shown in Figure 64.

Table 17: End Stud Displacement Between Positive and Negative Drifts at Peak Load.

Material	With Hold-Downs		Without Hold-Downs	
	(mm)	(in.)	(mm)	(in.)
OSB:				
Left end stud	9.9	0.39	36.8	1.45
Right end stud	9.4	0.37	24.4	0.96
Hardboard:				
Left end stud	10.9	0.43	23.4	0.92
Right end stud	8.1	0.32	22.4	0.88
Fiberboard:				
Left end stud	7.1	0.28	19.1	0.75
Right end stud	5.9	0.23	16.3	0.64
GWB				
Left end stud	3.6	0.14	8.9	0.35
Right end stud	3.3	0.13	7.1	0.28

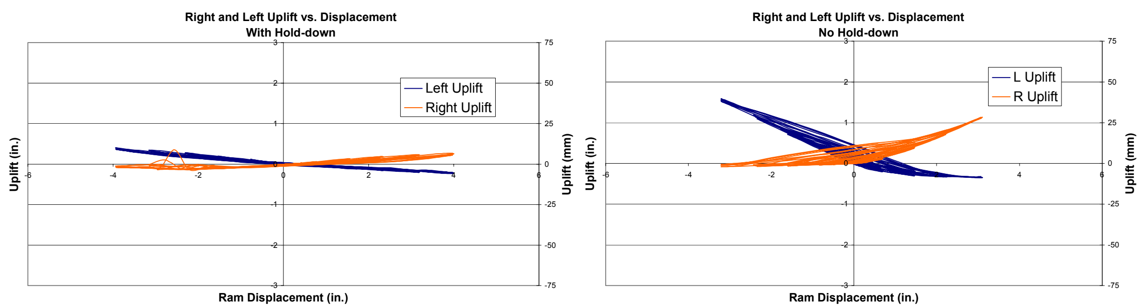

Figure 64: Vertical Uplift in Walls With and Without Hold-Down Connections

Monotonic and Cyclic Comparisons (walls with hold-down connections)

Average load-deflection curves for the monotonic and initial cyclic envelop curves for each configuration of wall tested are presented in Figures 65 – 71. The largest difference in the curves noticed is the effect of using hold-down connections. One also notes that the monotonic and cyclic envelope curves for when hold-down connections are present are similar, especially in the 0 – 50 mm (0 – 2 in) displacement range.

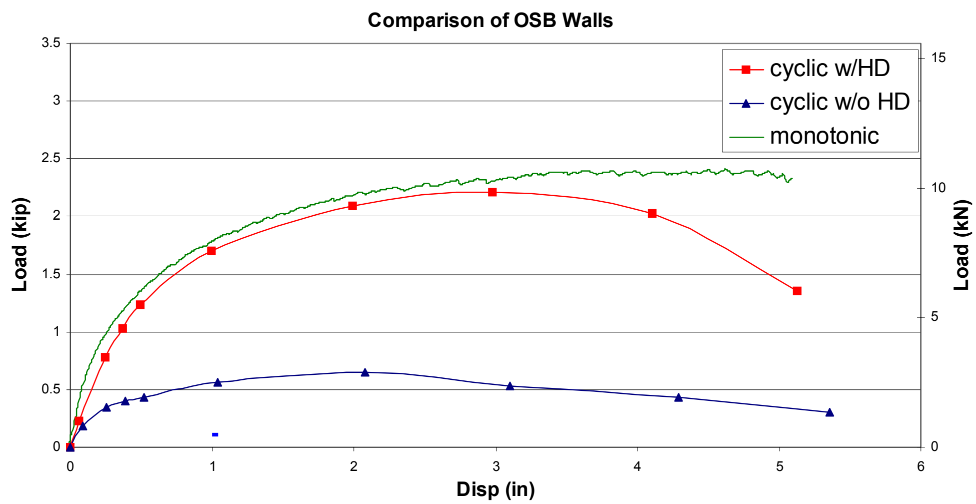

Figure 65: All Curves For OSB Sheathed Walls

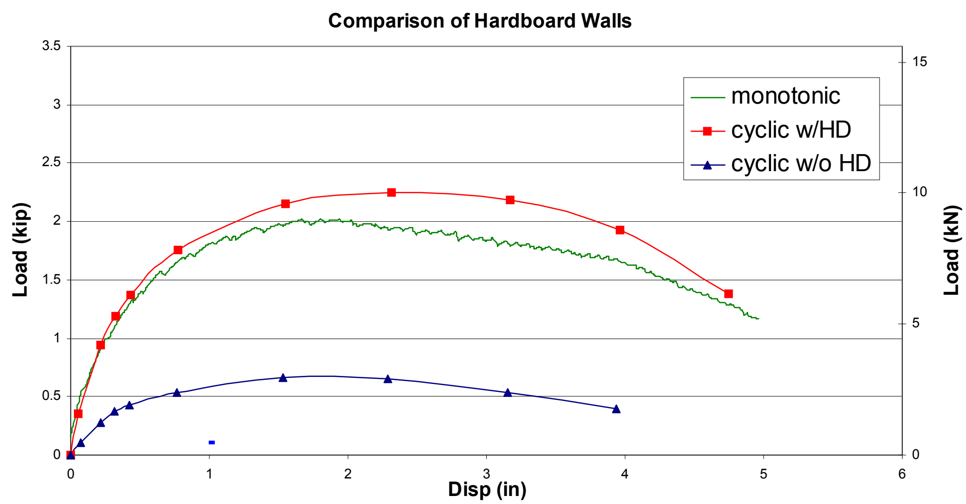

Figure 66: All Curves For Hardboard Sheathed Walls

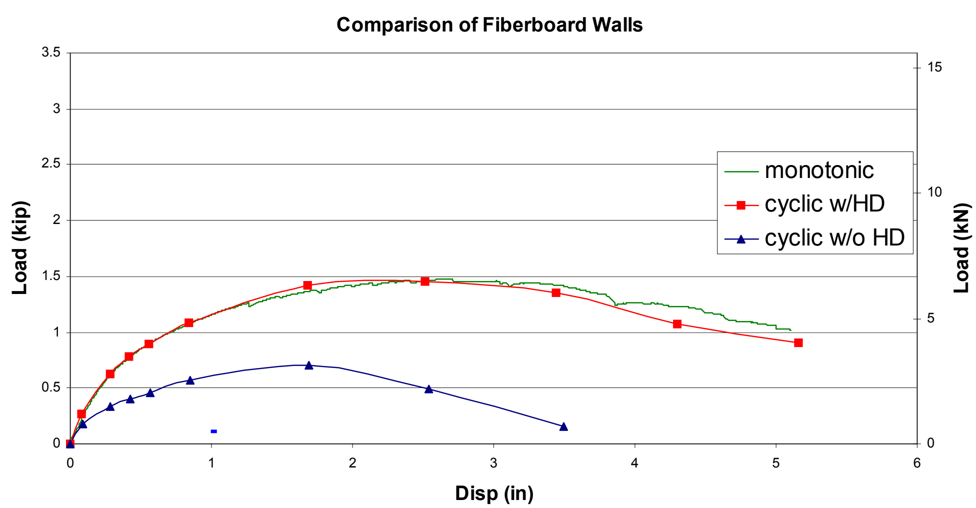

Figure 67: All Curves For Fiberboard Sheathed Walls

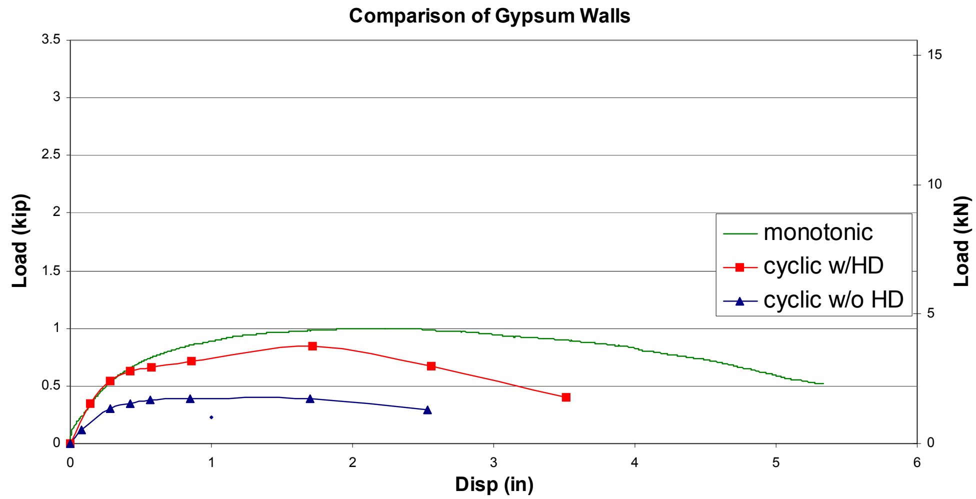

Figure 68: All Curves for GWB Sheathed Walls

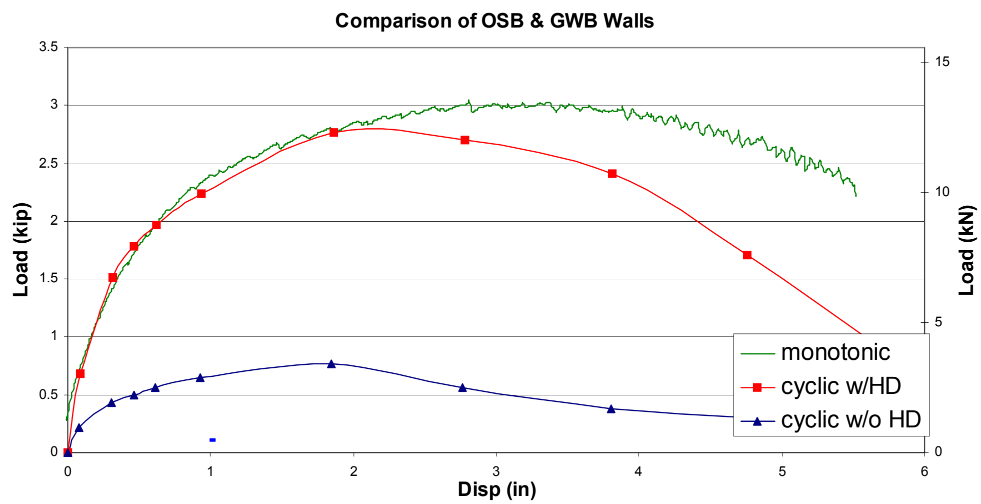

Figure 69: All Curves For OSB/GWB Sheathed Walls

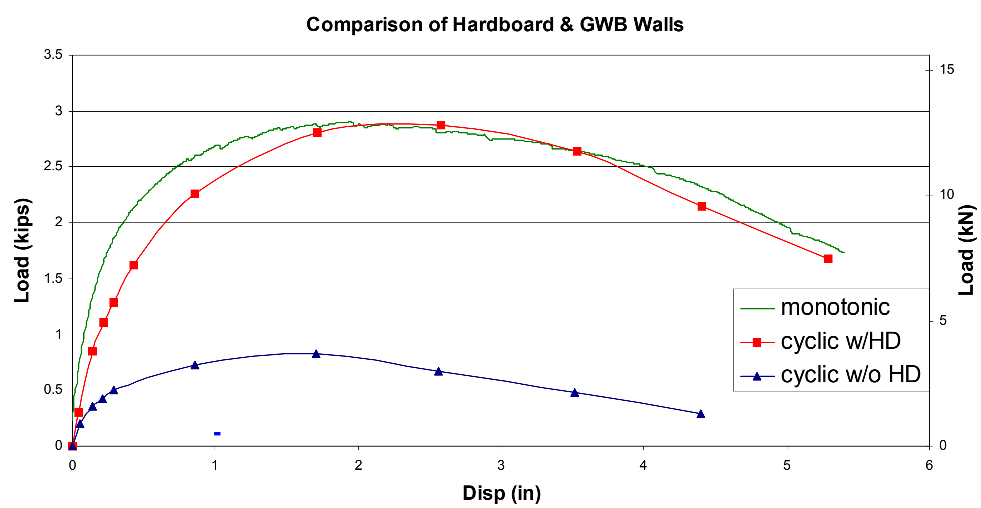

Figure 70: All Curves For Hardboard/GWB Sheathed Walls

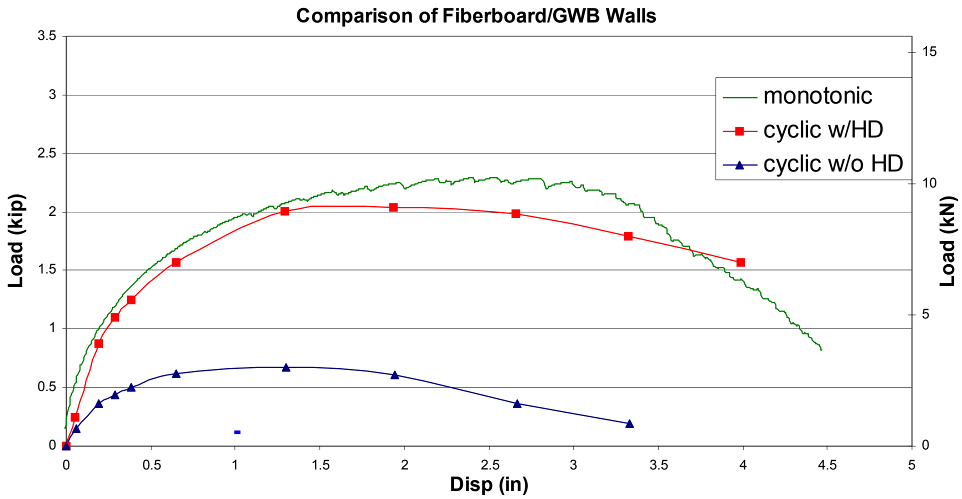

Figure 71: All Curves For Fiberboard/GWB Sheathed Walls

Peak Load and Displacement

Peak load values for the monotonic and cyclic tests with hold-downs are presented in Table 18, along with the ratio of cyclic peak load to monotonic peak load. The wall type that experienced the most reduction was GWB when tested alone. All of the walls displayed reduced strength when tested cyclically except for hardboard-sheathed walls, which actually increased in strength.

Table 18: Ratio of Peak Load (walls with hold-down connections)

Wall Type	Monotonic (1)		Cyclic (2)		Ratio (2)/(1)
	(kN)	(kips)	(kN)	(kips)	
Single Sided Walls					
OSB	11.2	2.51	9.8	2.21	0.88
Hardboard	9.3	2.08	10.0	2.25	1.08
Fiberboard	6.8	1.52	6.5	1.46	0.96
Gypsum	4.4	1.00	3.7	0.84	0.84
Double Sided Wall					
OSB/GWB	13.5	3.04	12.3	2.77	0.91
Hardboard/GWB	13.0	2.92	12.8	2.87	0.98
Fiberboard/GWB	10.3	2.31	9.2	2.06	0.89

As shown in Figures 65 and 69, OSB-sheathed walls typically reached maximum load at a larger displacement when tested under monotonic loading. The continuous fully reversed cycles caused more damage to the sheathing to framing connection than when the walls were subjected to monotonic loading.

Displacements at the peak load for monotonic and cyclic tests are displayed in Table 19. For all of the sheathing materials except hardboard, the average reduction in the displacement at peak load is close to 25%. The same reduction was experienced for the single-sheathed walls and the double-sheathed walls. Displacement at peak load during the hardboard tests actually increased during cyclic loading.

Table 19: Ratio of Displacements at Peak Load (walls with hold-down connections)

Wall Type	**Monotonic (1)**		**Cyclic (2)**		**Ratio (2)/(1)**
	(mm)	**(in.)**	**(mm)**	**(in.)**	
Single Sided Walls					
OSB	97.5	3.84	75.7	2.98	0.77
Hardboard	57.4	2.26	58.9	2.32	1.03
Fiberboard	86.4	3.4	64.0	2.52	0.74
Gypsum	56.9	2.24	43.7	1.72	0.77
Double Sided Walls					
OSB/GWB	82.0	3.23	53.1	2.09	0.65
Hardboard/GWB	50.3	1.98	60.2	2.37	1.20
Fiberboard/GWB	66.3	2.61	49.8	1.96	0.75

Displacement at Failure

Displacements at failure of all tests are compared in Table 20. Every cyclic test experienced a reduction in the failure displacement when compared to its monotonic test. The largest reduction occurred during GWB sheathed wall tests. During fully reversed cycles, the nails would tear through the GWB sheathing in both directions at a faster rate, which enabled the wall to fail at a lower displacement. OSB-sheathed walls experienced a reduction of nearly 25mm (1 in.) during the cyclic tests. The displacement at failure of hardboard and fiberboard appeared to be the least affected by the cyclic loading. The decrease in failure displacement can be attributed to the nail behavior. During cyclic loading, the nails are subjected to many fully reversed cycles increasing damage of the sheathing connection. The displacement at failure of the fiberboard actually increased by a small amount when tested under cyclic loading.

Table 20: Ratio of Displacement at Failure (walls with hold-down connections)

Wall Type	**Monotonic (1)**		**Cyclic (2)**		**Ratio (2)/(1)**
	(mm)	**(in.)**	**(mm)**	**(in.)**	
Single Sided Walls					
OSB	142	5.6	116	4.6	0.82
Hardboard	117	4.6	106	4.2	0.91
Fiberboard	117	4.6	109	4.3	0.93
Gypsum	104	4.1	65	2.6	0.63
Double Sided Walls					
OSB/GWB	128	5.1	103	4.1	0.80
Hardboard/GWB	114	4.5	106	4.2	0.93
Fiberboard/GWB	93	3.7	96	3.8	1.03

Elastic Stiffness

Due to the variation of the elastic stiffness, a definite relation between the elastic stiffness and the loading procedure cannot be established. As shown in Table 21, the elastic stiffness increases in some cases and decreases in other cases when subjected to cyclic loading. Values shown in Table 21 are relatively close, and given the effects of initial load, it explains why some of the values are higher and some are lower during cyclic tests. The wall configuration that had a large variation was the Hardboard/GWB. Elastic stiffness decreased by more than 50% when tested cyclically. When reviewing the two specimens tested under monotonic loading, the initial loads were 170 and 255 lbs. Therefore, the elastic stiffness is actually closer to 0.8 kN/mm (4.74 kip/in) than 1.96 kN/mm (11.14 kip/in) as calculated during monotonic tests.

Table 21: Ratio of Elastic Stiffness (walls with hold-down connections)

Wall Type	Monotonic (1)		Cyclic (2)		Ratio (2)/(1)
	(kN/mm)	(kip/in)	(kN/mm)	(kip/in)	
Single Sided Walls					
OSB	0.76	4.31	0.57	3.27	0.76
Hardboard	0.73	4.18	0.93	5.27	1.26
Fiberboard	0.38	2.18	0.51	2.91	1.33
Gypsum	0.39	2.22	0.45	2.56	1.15
Double Sided Walls					
OSB/GWB	1.01	5.72	1.10	6.28	1.10
Hardboard/GWB	1.96	11.14	0.83	4.74	0.43
Fiberboard/GWB	1.11	6.34	0.94	5.34	0.84

Ductility

Ductility values of the cyclic tests varied by a large amount when compared to monotonic tests. Average values for ductility ratio are shown in Table 22. Ductility tended to slightly increase or stay the same when tested cyclically. Although the failure displacement decreased during cyclic loading, the yield displacement also decreased, which increased the ductility. Large initial loads induced during the monotonic tests yielded an unrealistic yield displacement, which increased the ductility.

Table 22: Ratio of Ductility Ratios (walls with hold-down connections)

Wall Type	Monotonic (1)	Cyclic (2)	Ratio (2)/(1)
Single Sided Walls			
OSB	7.2	7.7	1.07
Hardboard	8.7	10.4	1.20
Fiberboard	7.3	9.4	1.29
Gypsum	10.0	8.9	0.89
Double Sided Walls			
OSB/GWB	10.3	10.5	1.02
Hardboard/GWB	18.5	7.5	0.41
Fiberboard/GWB	11.0	10.5	0.95

Energy Dissipation

The energy dissipated during monotonic tests is calculated by determining the area under the load-displacement graph. The total amount of energy dissipated during cyclic tests is the summation of all the cycles. However, for comparison purposes, the energy dissipation values shown in Table 23 were determined from the area under the initial envelope curves. The limits of the energy dissipation are from the point of zero displacement to the failure displacement, which was taken at a displacement equal to $0.8F_{peak}$.

Table 23: Ratios of Energy Dissipation (walls with hold-down connections)

Wall Type	Monotonic (1)		Cyclic (2)		Ratio (2)/(1)
	(kN-m)	(kip-ft)	(kN-m)	(kip-ft)	
Single Sided Walls					
OSB	1.31	0.97	0.96	0.71	0.73
Hardboard	0.95	0.70	0.91	0.67	0.96
Fiberboard	0.66	0.49	0.63	0.46	0.94
Gypsum	0.41	0.30	0.23	0.17	0.57
Double Sided Walls					
OSB/GWB	1.49	1.10	1.08	0.80	0.73
Hardboard/GWB	1.24	0.92	1.17	0.86	0.93
Fiberboard/GWB	0.81	0.60	0.75	0.56	0.93

As shown in Table 23, the amount of energy dissipated by the walls under cyclic loading decreased when compared to monotonic tests. This occurred because the peak load and failure displacement was normally higher during monotonic tests, which increased the area under the load-displacement curve. As stated before, the cyclic tests dissipated much more total energy than the monotonic tests due to the numerous cycles that overlapped, but for comparison purposes only the initial envelope curve is considered. Because of the numerous cycles applied during the cyclic tests, nail fatigue contributed to the decreased energy dissipating capacity of the walls.

Response curves of the four sheathing materials behaved differently when considering energy dissipation. OSB panels experienced a large reduction in strength when tested cyclically. The energy reduction was 27% and can be explained by the failure displacement of the walls. During monotonic tests, OSB panels did not experience a sudden drop in strength after reaching peak load. Nails in the OSB would bend, but they were still attached to the wall and could resist load through a large displacement. When tested cyclically, OSB experienced a definite drop in strength after reaching peak load, which was a result of sheathing nail connection damage. As shown in Figure 64, the response during the two loading conditions were nearly identical until reaching peak load, but quite different after peak load.

The reduction in energy dissipation of hardboard and fiberboard-sheathed walls was small. The reduction during hardboard tests was 4%, while the reduction during fiberboard tests was only 6% when subjected to cyclic loading. As shown in Figures 66 and 67, the average response was nearly identical during monotonic and cyclic tests. For both loading conditions, the nail behavior was similar. During the hardboard tests, the sheathing nails pulled out of the framing until the panels were no longer affectively attached to the framing. The sheathing nails in the fiberboard typically pulled out of the framing, or tore through the low density panels during monotonic and cyclic loading. Similar nail behavior allowed the walls to fail in the same manner and at the same displacement.

GWB experienced the largest reduction of energy when tested cyclically (43%). This was not surprising given the ease of which the nails in the GWB could tear through the sheathing. The actuator displacement moved at a much more rapid speed during the cyclic tests. The increased rate damaged the GWB sheathing more drastically than the slower, more constant rate experienced during monotonic tests. When comparing the loading procedures, the GWB seems to be the least effective when combined with OSB panels, and the most productive when combined with hardboard panels.

SUMMARY

A total of 45 walls were tested under monotonic loading using ASTM E564 and under cyclic loading using ASTM E2126. All of the walls were 1.2 x 2.4m (4 x 8ft), and to be conservative, there were no gravity loads applied to the walls. The intent of the test was to investigate the effect of combining Gypsum Wallboard (GWB) with 3 other typical light-frame sheathing materials (Oriented Strandboard (OSB), Hardboard, and Fiberboard.) Replicates of 2 were used throughout the investigation, with each of the sheathing types tested as single sided walls and then GWB was added to walls sheathed on one with one of the other three sheathing materials. Comparisons are made between each of the material types as well as the effect of GWB on the various performance parameters (i.e., peak load, yield load, stiffness, energy dissipation, etc.)

The results indicate that the monotonic and cyclic response of all of the walls were similar in stiffness and load up to a displacement of approximately 50 mm (2 in). GWB was effective in increasing the peak load for walls with hold-down connections, and was close to being linearly additive for walls tested monotonically. GWB was not as effective for walls with hold-down connections that were tested cyclically. GWB was not effective in changing the peak load for walls without hold-down connections.

Wall specimens using hold-down connections had higher peak load and stiffness values than walls without these connections. The walls with hold-down connections also had a higher toughness, in that the displacement capacity and energy dissipation characteristics were significantly improved. This performance improvement is due to the more distributed resistance of the sheathing nails in walls with hold-down connections. Walls without hold-down connections concentrated the resistance at the bottom row of nails into the bottom plate. These nails had to resist both the shear and overturning forces when the hold-down connections were not present.

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