MONOTONIC AND CYCLIC PERFORMANCE OF LIGHT-FRAME SHEAR WALLS WITH VARIOUS SHEATHING MATERIALS

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(Abstract)

The racking performance of light-frame shear walls subjected to monotonic and cyclic loading is the focus of this thesis. The sheathing materials investigated are oriented strandboard (OSB), hardboard, fiberboard, and gypsum wallboard. The objectives of this study were to (1) obtain and compare performance characteristics of each sheathing material; (2) compare the effects of monotonic loading versus the cyclic loading response; (3) investigate the contribution of gypsum in walls with dissimilar sheathing materials on opposite sides of the wall; and (4) study the effects of using overturning anchors. The monotonic tests, which incorporated the use of hold-downs, were performed according to ASTM E564. Half of the cyclic tests were performed with hold-downs, and half were performed without hold-downs. The cyclic tests were performed according to the recently adopted cyclic testing procedure ASTM E2126.

A total of forty-five walls were tested with various configurations. The size of the walls was 1.2 x 2.4m (4 x 8ft). Two tests were performed with each sheathing material subjected to each type of loading: monotonic, cyclic with hold-downs, and cyclic without hold-downs. Two tests were then performed with OSB, hardboard, or fiberboard on one side of the wall and gypsum on the other side of the wall to study the effects of using dissimilar sheathing materials on the shear walls. The OSB and hardboard exhibited similar performance, and were the strongest of the four sheathing materials. Fiberboard performed better than gypsum, but worse than OSB and hardboard. In general, the performance indicators decreased when the walls were subjected to cyclic loading. The contribution of gypsum to walls with hold-downs was significant, but was not linearly additive. The use of hold-downs had a large effect on the performance of the walls. All shear wall performance indicators decreased when hold-downs were not included, with an average reduction of 66% in the peak load.

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Chapter 1: Introduction and Literature Review

INTRODUCTION

1.1 Problem Overview

Every year in America, between one and two million new homes are built, of which timber framed houses account for more than 90 percent. Due to environmental concerns and the large quantity of wood being used, more efficient and cost effective use of wood is highly desired. Deterioration of wood and natural hazards, such as hurricanes and earthquakes, are two major occurrences that can lead to collapse or expensive repairs to houses. This study focuses on understanding how light-frame buildings react when subjected to lateral forces caused by natural hazards. More specifically, how the shear walls that resist lateral forces perform.

The lateral load resisting mechanism in a building is typically referred to as a shear wall. The load path in a residential house is shown in Figure 1.1. Shear walls are used to resist three major load-carrying components: vertical loads, transverse wind loads, and in-plane lateral forces imposed by wind and seismic forces. Shear walls support the horizontal diaphragm and transfer the lateral forces down to the foundation of the structure. They are usually constructed out of 51 x 102mm (2 x 4in.) framing materials. Studs supply the vertical load carrying capacity with typical spacing of 406mm (16 in.) on center. The bottom plate and top plate complete the frame and connect to the foundation or the adjacent floor. The material that covers the framing, traditionally plywood or oriented strandboard (OSB), provides the shear capacity. When the sheathing material is connected to the framing with nails or other type of fasteners, it provides a stiff, yet ductile wall, which is desirable in the event of an earthquake or hurricane.

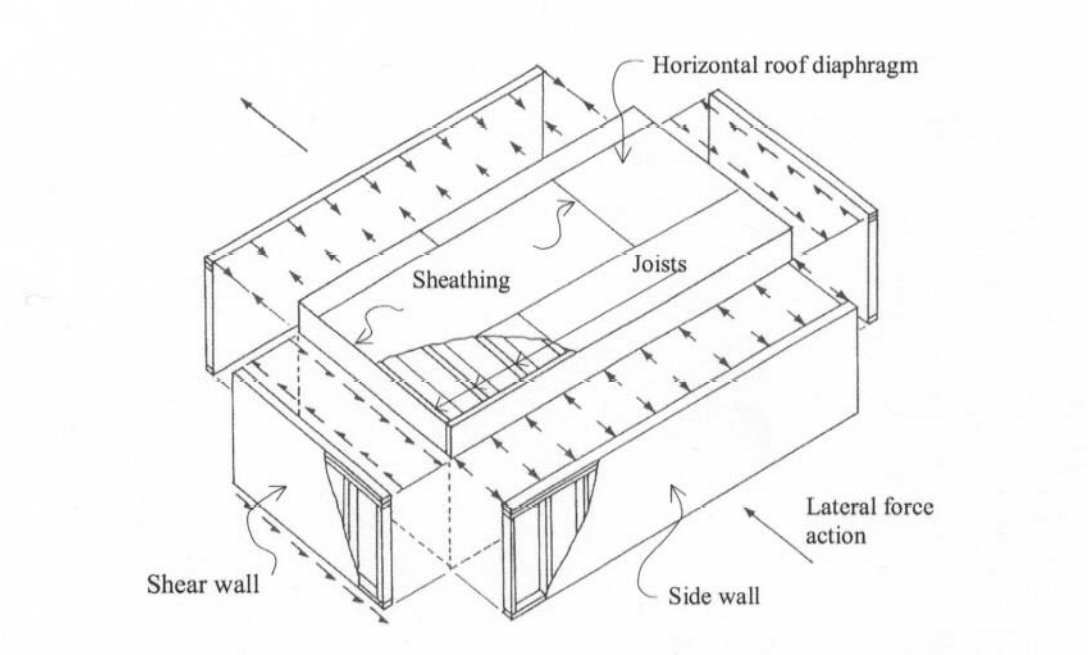

Figure 1.1: Lateral force resisting system of light-frame building (Diekmann 1995)

Wood structures have historically performed well when subjected to lateral loads. This performance is due partly to wood's low weight/high stiffness ratio, the ductility of timber structures, and the redundancy of the system represented by the many fastener connections of the sheathing to the framing. As a result of the ductility, or ability to deform without breaking, shear walls have a capacity to dissipate large amounts of energy produced by earthquakes and hurricanes (Leiva-Aravena 1996).

Wood structural panels, which include plywood, OSB, waferboard, composite panels, and structural particleboard--are widely used building materials with many structural and nonstructural applications. Major structural uses are for roof, floor, and wall sheathing used as lateral load resistance for horizontal and vertical shear walls. The relatively high allowable strength and ease with which the panels can be installed make wood structural panels an economical choice for shear wall applications (Breyer et al. 1999).

To better use our natural resources, new products are developed and researched. Plywood and OSB are primarily used as the sheathing material for shear walls. However, other materials such as fiberboard and hardboard can also be effectively used as sheathing material to resist shear loads in houses. This study investigates these two

products as well as the contribution of gypsum wallboard to the lateral resistance of shear walls.

1.2 Objectives

The purpose of this study is to investigate the performance of light-frame shear walls with and without overturning restraints when sheathed with various structural panels. OSB, hardboard, fiberboard, and gypsum wallboard panels were used as sheathing materials and subjected to monotonic and fully reversed cyclic loading. The main objectives of this investigation are outlined below.

1. Quantify the resistance parameters of hardboard, fiberboard, and gypsum wallboard. These sheathing materials have not been tested and researched in as much detail as OSB and plywood has for shear wall applications. The OSB in this study serves as a means of comparison with the other sheathing materials.
2. Compare the results of the monotonic tests to the cyclic tests. Due to the numerous fully reversed cycles of cyclic loading, sheathing nails tend to fatigue and fail at a faster rate than during monotonic loading. Therefore, the need to quantify the reduction is important
3. Investigate the performance of dissimilar sheathing materials on opposite sides of a shear wall. Half of the walls were tested with one sheathing material on the exterior of the wall only, and half of the walls were sheathed with OSB, hardboard, or fiberboard on the exterior of the wall and gypsum wallboard on the interior in order to understand the behavior of two sheathing materials when used together in a structural system. Of interest is whether gypsum wallboard is linearly additive as specified in design codes.
4. Investigate the effects of using overturning anchors in the form of hold-downs. All of the walls subjected to cyclic loading were tested with and then without hold-downs to investigate the differences in shear wall behavior due to overturning anchors.

All of the walls were tested using a 2:1 aspect ratio. The walls were built eight feet high and four feet wide. This aspect ratio was chosen because it represents the

maximum height-to-width ratio that can be used for gypsum wallboard. The 2:1 aspect ratio is also the maximum ratio permitted for wood structural panels used to resist seismic forces without further reduction in design values.

1.3 Thesis Overview

Past shear wall studies, including an investigation of experimental testing of walls subjected to monotonic and cyclic loading, are briefly reviewed in Sections 1.4-1.8. Numerous attempts to model shear wall behavior are also described in the literature review. Finally, a history of the loading procedures is described, including changes in the monotonic loading procedure and adoption of the ASTM cyclic testing protocol.

Described in Chapter 2 are the materials and procedures used to test the shear walls in this study. The sheathing materials and the framing members are described and the method used to construct the walls is detailed. The testing apparatus and data acquisition are described in detail. The parameters used to analyze the performance of the shear walls are also defined in Chapter 2. The results of the monotonic tests are discussed in Chapter 3, while the results of the cyclic tests are discussed in Chapter 4. The performance of the walls subjected to monotonic and cyclic loading is compared in Chapter 5. The conclusions of this study are provided in Chapter 6.

LITERATURE REVIEW

1.4 Introduction

As a major lateral load resisting system in most houses and small buildings, shear walls have been the focus of numerous studies and research projects. Wood shear walls are used to resist three major load-carrying components: vertical loads, transverse wind loads, and in-plane lateral forces imposed by wind and seismic forces (Lam, et al. 1997). Shear walls typically consist of 51 x 102mm (2 x 4in.) studs as the framing, with structural wood panels to sheath the outside and gypsum wallboard to sheath the inside of the wall. Depending on the material, a specified fastener schedule is used. This chapter reviews notable studies performed on light-framed timber shear walls. Many of the experimental tests reviewed here have been performed on 2.4 x 2.4m (8 x 8ft) shear walls, which represent a typical wall in a residential home. Early research focused

mainly on monotonic (static) testing which is a one-directional loading used to obtain the ultimate strength of a wall. This method was sufficient for most regions, but in areas of high seismic activity, the performance of shear walls under reversed cyclic loading needed to be studied. Therefore, cyclic (reversed) loading has been the main focus of recent shear wall studies. By performing monotonic and cyclic tests, researchers have gained insight into the behavior of shear walls and comparisons between the two types of loading can be easily understood.

Prior tests have considered many different variables. The contribution of various sheathing materials has been studied along with different length-to-width ratios (aspect ratios). The effects of shear walls with openings and the use of hold-down connectors have also been studied. Because the sheathing-to-framing connection is the single most important variable in shear wall performance, many connection tests have been performed. During connection tests, the use of various fastener types and fastener spacing has been studied. This chapter reviews several other shear wall parameters as well.

In addition to experimental studies, numerous mathematical models and prediction equations have been developed to help understand the performance of shear walls. An accurate mathematical model can produce more informative results in a faster and less expensive manner than experimental tests. Thus, the literature review also reviews mathematical models.

1.5 Monotonic Loading

1.5.1 Experimental Testing

For many years, monotonic loading was the standard method for testing shear walls because it provided a good indication of the performance under one-directional loading, or wind loading. Many studies have evaluated and predicted the performance of shear walls subjected to monotonic loading. Tuomi and Gramala (1977) examined the rate of loading, sheathing material, and let-in corner bracing. Price and Gromala (1980) tested ten types of structural flakeboard and two types of southern pine plywood to determine the racking strength of the walls. They tested 8 x 8ft and 2 x 2ft walls according to ASTM E72-98 (2000). They determined that panels sheathed with

flakeboard, containing a mixture of hardwood and pineflakes, were slightly stiffer than southern pine plywood when subjected to a racking load. However, the plywood sheathed panels provided slightly higher strengths for the full-size racking test.

Wolfe (1983) tested 30 walls to investigate the contribution of gypsum wallboard to the racking resistance of shear walls. Performing the tests according to ASTM E564-95 (2000), he evaluated the influence of wall length, panel orientation, and the wallboard/frame interaction. Wolfe determined that the racking resistance of walls with gypsum and structural wood panels appeared to equal the sum of contributions of the elements tested independently. Walls tested with panels oriented horizontally were more than 40% stronger and stiffer than those with panels oriented vertically. Wolfe concluded that gypsum wallboard could provide significant contribution to the racking resistance when subjected to monotonic loading.

Griffiths (1984) examined racking loads of 2.4 x 2.4m (8 x 8ft) shear walls with various sheathing materials and small variations in nailing schedules. He found ASTM E72 unsuitable for racking tests because it over restrains the panel giving unrealistic failures.

McDowall and Halligan (1989) tested 22 wall panels having widths of 300mm (11.8 in.), 450mm (17.7 in.), and 600mm (23.6 in.). They also tested 25 two-panel and three-panel continuous wall systems. The purpose of the tests was to evaluate the racking resistance provided by short lengths of timber framed wall panels sheathed with particleboard. Conventional construction and rodded systems were tested using a method similar to that of ASTM E72 and E564. They concluded that the allowable design loads should be stiffness based, with a maximum deflection established.

Serrette et al. (1996) evaluated the effects of different sheathing materials that were attached to cold-formed steel under static and cyclic loading. The tests showed that the nominal capacity of plywood panels was approximately 17% greater than that of the 7/16" OSB panels. Plywood walls also exhibited much larger deformation capacity at the maximum load. However, under cyclic loading the plywood only provided 10% more resistance than the OSB panels. Depending on the fastener schedule, the static strength of shear walls may be as much as 55% more than the corresponding cyclic strength.

With small screw spacing, gypsum wallboard did not contribute to the overall strength of the wall when placed on the other side of walls sheathed with wood panels.

Shear walls with openings are important to understand because most houses and buildings have openings in the walls. Johnson (1997) performed monotonic and cyclic tests on long shear walls with several different aspect ratios (walls with different opening orientation and number of openings). From the monotonic and cyclic tests, he concluded that the sheathing above and below openings could resist shear. Gypsum wallboard helps resist shear in the low to moderate loading, but plywood resists most of the shear near capacity under monotonic loading. Pull-through of the plywood sheathing nails was the predominant mode of failure for monotonic loading. The method developed by Sugiyama and Matsumoto (1994) to predict the behavior of shear walls with openings was found to be conservative when compared to the full-scale tests. As the area of openings increased, Sugiyama's prediction method became overly conservative. The natural log prediction method developed by Johnson proved to give good predictions of the capacity and stiffness of the shear walls for both loading conditions. However, it should be noted that Johnson's results were based on limited data.

For his thesis, Heine (1997) investigated the effects of overturning restraint (tie-down anchors) on the performance of light-framed wood shear walls subjected to monotonic and cyclic loading. He discovered that tie-down anchors enhanced the overall performance of shear walls. Walls with no overturning restraints failed due to nail tear through and stud separation along the bottom plate. These walls exhibited much lower stiffness and capacity than walls with maximum overturning restraint. Rigid body rotation arising from uplift and separation along the bottom plate occurred when tie-down anchors were used.

1.5.2 Modeling

Tuomi and McCutcheon (1978) developed a method for calculating the racking strength of timber-framed shear walls. Their theoretical model is compared with several laboratory studies. Input parameters involved were panel geometry, the number and spacing of nails, and the lateral resistance of a single nail. The theoretical solution, however, only considers the resistance contributed by the fasteners and does not consider

the contribution of the lumber frame. Seven different types of sheathing material (plywood, gypsum, and several grades and thickness of fiberboard) were used, and the agreement between theoretical and actual loads was close. However, the method is not applicable to stiffness computations and is only valid for small deformations since the load/distortion relationship for a single nail was assumed to be linear.

Foschi (1982b) tested shear walls and diaphragms with waferboard sheathing to evaluate the performance and to develop allowable racking design strengths for different combinations of waferboard thickness, nail type, and nail spacing. Tests were performed to verify the accuracy of a computer analysis program. Once validated, the computer program could analyze many scenarios to determine the behavior of shear walls and allowable design strengths. Selection of nail type significantly impacts the behavior of shear walls and diaphragms. Higher loads can be obtained by using high withdrawal-resistance nails.

Formulas to analyze shear walls were developed by Easley, et al. (1982). The formulas can be used for various types of sheathing panels attached with nails or other types of discrete fasteners. Formulas were derived for the sheathing fastener forces, the linear shear stiffness of a wall, and the non-linear shear load-strain behavior of a wall. The formulas derived were shown to be in close agreement with actual load tests and finite element analysis.

McCutcheon (1985) developed a method to predict racking deformations of wood-stud shear walls taking into account the nonlinear behavior of nails. The energy method used defines the wall performance in terms of the lateral nonlinear load-slip behavior of the nails attaching the sheathing to the framing. McCutcheon also considered the shear deformation of the sheathing when determining the racking strength. A major limitation of the method is it is accurate only for small deformations. As the deformation increases, the method underestimates displacement at higher loads.

Gupta and Kuo (1985) presented new mathematical models that represent the performance of shear walls. Although the nail load-slip characteristic governs the behavior of shear walls, the researchers also felt the need to include the bending stiffness of the studs and shear stiffness of the sheathing as important roles in modeling the stiffness of the walls. The model was in good agreement with finite element models and

tests. If the studs were assumed to be infinitely rigid, the model still compared closely to other models and reduced the degrees of freedom from six to three.

Gutkowski and Castillo (1988) developed a mathematical model to analyze light-frame shear walls subjected to monotonic loading. The nonlinear analysis covers either single-or double-sided sheathed walls using plywood and gypsum. The authors used previous experimental results to validate the mathematical model. Results of their model closely represent actual load-displacement curves of shear walls.

Ge (1991) developed a systematic approach to analyze the response of wood-frame houses to lateral loads. The model he developed was then compared to results of two previous full-scale houses that were tested under lateral loads. The model proved to be in good agreement with the experimental response. Previous shear wall diaphragm models were also evaluated and modified to consider the effects of window and door openings on the racking stiffness of wall diaphragms.

A simpler method for calculating the racking strength of shear walls was proposed by Sugiyama and Matsumoto (1993). Two simple assumptions were made: (1) only the direction along the perimeter of the shear wall contributes to the slip resistance of each nail, and (2) slip resistance of nails at intermediate studs is negligible. The authors' method estimates slightly smaller values of racking loads than those determined by other methods. However, the use of a bilinear equation makes the analysis of plywood-sheathed walls easier than the previously proposed exponential equation.

1.5.3 Connection Tests

Foschi (1982a) performed nailed connection tests to evaluate the performance of shear walls and diaphragms sheathed with waferboard panels. The objective of the study was to develop allowable racking design loads for different combinations of waferboard thickness, nail type, and nail spacing. Tests used 10d, 8d, and 6d nails; then three types of 8d nails were tested. From the three 8d nails tested, there was significant difference in the various companies' nails. The weakest of the 8d nails were then tested, as manufactured, and cleaned to show there was a larger load capacity when the nails were cleaned.

1.5.4 Other Parameters

De Klerk (1985) tested the effects of stud spacing on shear wall performance. The tests showed that the stud spacing directly influenced ultimate load capacity. As the stud spacing decreased, the capacity of the wall increased. However, a corresponding increase in the stiffness was not observed.

Salenikovich (2000) determined that low wood density reduced the nailed connection strength, and therefore, the shear wall strength and toughness were also reduced.

1.6 Cyclic Loading

1.6.1 Experimental Testing

Due in part to the major earthquakes in the past decade, the ability to predict the behavior of shear walls under cyclic loading was needed. This information, however, was not available because cyclic tests had not been performed to understand how walls behaved under earthquake loads. As opposed to monotonic tests that are one directional, cyclic tests are composed of continuous, fully-reversed cycles. Cyclic tests were slow to develop due to the complication of the testing method and the lack of consensus on a test protocol for conducting such tests (Rose 1998).

Several factors must be considered when walls are subjected to cyclic loading to simulate seismic loading conditions. Shear wall displacement limits are important to minimize the damage during earthquakes along with the structure's ductility, fatigue resistance of fasteners, and slip of shear wall hold-down connectors. Effects of monotonic loading versus cyclic loading on shear wall stiffness and strength need to be considered, as well as the contribution of dissimilar materials to stiffness and strength. (Rose 1998).

In an early investigation into cyclic loading, Medearis and Young (1964) performed an experimental and theoretical investigation into the energy-absorption characteristics of conventional plywood shear walls. They observed that nails showed good ability to bend and deform back and forth under cyclic loading. None of the shear walls experienced sudden failure, and there appeared to be no harmful effect from cyclic loading of plywood shear walls in the low-load range. Energy absorption properties of

the panels were large with approximately 60% of the input energy being absorbed during any given half-cycle.

Gray and Zacher (1988) recognized the need for a national standard for dynamic testing of earthquake-resisting assemblies since many shear wall aspects are inadequately measured by traditional static tests. The authors performed 2.4 x 2.4m (8 x 8ft) shear wall tests under cyclic loads to prove that shear wall properties under lateral loads are inadequately measured by traditional static tests. Gray and Zacher concluded that the framing-to-sheathing fastener interaction is the most important behavior in shear wall action, and that very little distortion of the plywood panel takes place. Distortion and energy dissipation are almost entirely due to nail flexure beyond yield point. The authors also proved that overdriven nails in plywood permit sudden failure with extremely reduced energy dissipation. Gypsum sheathing contributes little energy dissipation because nails excavate holes by back and forth motion, which also minimizes strength. They concluded that the allowable loads for gypsum sheathing were too high, and as a result were reduced from the findings of this research.

Dolan (1989) performed connection tests and dynamic tests on a total of 25 walls, and developed finite element models to determine the dynamic response of shear walls. After testing 11 walls sheathed with waferboard and 14 walls sheathed with plywood, Dolan proved that little difference occurred between the two sheathing materials when tested under static or dynamic loading. From full-scale tests, connection tests, and mathematical models, he showed that the density of the sheathing nails governs the behavior of shear walls more than any other single variable. Fasteners govern the load capacity, stiffness, and ductility of timber shear walls. From the models he developed, the programs' code can be modified to include nails manufactured by others companies not included in existing North American design codes. The models can also be used to analyze the effects of different fastener types such as adhesives, screws, and staples.

Yasumura (1992) tested wood-framed shear walls with various types of sheathing materials subjected to monotonic and reversed cyclic lateral loading. The effects of the loading protocol were much more evident on walls sheathed with inorganic materials (gypsum board, cemented wooden chip board) than on wood-based panels. The maximum loads in the reversed cyclic loading, which consisted of progressively

increasing cycles, were 9 to 33% lower than those obtained from monotonic tests using ASTM E72. There were few differences in the maximum deformation of plywood and OSB, while the maximum deformation decreased by 35 to 57% for the inorganic materials under cyclic loading.

Karacabeyli and Ceccotti (1996) tested the contribution of gypsum wallboard (GWB) to shear wall capacity. Nails and screws, used as a fastener for the gypsum, were also compared for 8 x 16ft shear walls. The tests determined that using gypsum on one side and OSB on the other side increased the ultimate resistance, but the ductility was lower than a wall with only OSB on one side. Up to a displacement of 1 ¼ inches, individual properties of OSB and GWB can be superimposed to determine the lateral resistance of walls with OSB on one side and GWB on the other side. The researchers also discovered that for a displacement over 0.4 inches, walls using nails exhibited greater ductility and resisted greater ultimate loads than walls using drywall screws.

Leiva-Aravena (1996) conducted full-scale tests to analyze the racking behavior of timber-framed shear walls. The testing procedure implemented was a reversed cyclic loading known as BRANZ P21, used in New Zealand. This unique test protocol requires a vertical restraint simulating what would occur in an actual building and utilizes a double amplitude cyclic test method. The tests showed that the shear walls exhibited very ductile behavior and possessed considerable energy absorption and damping capacity. Also, the racking behavior of shear walls at a given time was shown to be a function of its past loading history.

Johnson (1997) performed monotonic and sequential phased displacement (SPD) cyclic tests on long shear walls with several different aspect ratios (walls with different opening orientation and number of openings). From the monotonic and cyclic tests, he concluded that the sheathing above and below openings is able to resist shear. In cyclic loading, gypsum failed after repetitive cycles at low displacement levels, contributing little to the wall performance. Nail fatigue was the predominant mode of failure. The method developed by Sugiyama and Matsumoto (1994) to predict the behavior of shear walls with openings was found to be conservative when compared to the full-scale tests. As the area of openings increased, Sugiyama and Matsumoto's prediction method became overly conservative. The natural log prediction method developed by Johnson

proved to give good predictions of the capacity and stiffness of the shear walls for both loading conditions.

From studying the effects of tie-down anchors on long shear walls with openings subjected to SPD cyclic loading, Heine (1997) observed that the ultimate capacity of the shear walls increased as more tie-down anchors were used. Gypsum wallboard performed poorly during cyclic loading. Extensive damage around nails and taped joints occurred. The typical failure modes for maximum overturning restraints were nail fatigue and tear through. When no tie-down anchors were used, the failure mode was almost complete separation of studs and sheathing from the bottom plate. Ultimate capacities for SPD loading were as much as 23% lower when compared to monotonic test results for walls restrained against uplift. Heine also tested walls with corner framing and concluded that this construction provided a hold-down effect that increased wall capacity when compared to straight walls without overturning restraint. The corner framing provided comparable hold-down capacity that straight walls with tie-down devices provide.

Karacabeyli and Ceccotti (1998) compared the lateral resistance of shear walls subjected to different cyclic and pseudo-dynamic displacements schedules. The procedures tested were from a draft ASTM Standard, a draft CEN Standard, and a draft ISO Standard. The authors discussed the differences of the maximum load carrying capacity, ultimate displacement, envelope curves, and dissipated energy of each testing procedure.

Rose (1998) performed an in-depth study of shear walls subjected to cyclic loading. Some objectives were to obtain preliminary load-displacement curves and load capacity of wood-framed walls under cyclic loading using plywood and OSB as sheathing materials. Rose performed two cyclic load procedures to determine if they accurately represent the performance of shear walls under earthquake forces. The first method, developed during a joint U.S. and Japan Technical Coordinating Committee on Masonry Research (TCCMAR), was the sequential phased displacement (SPD) procedure (Porter 1987). The second method consisted of a “ramped” cyclic displacement procedure that did not utilize the subsequent “decay” cycles used in the TCCMAR procedure. Both tests yielded similar results, and the TCCMAR procedure was

recommended for cyclic load tests. Rose also evaluated whether gypsum wallboard attached to one side of the wall contributes strength and stiffness under cyclic conditions. Gypsum wallboard did increase the wall's stiffness, but did not contribute to the wall's maximum shear strength when subjected to cyclic loads.

Dinehart and Shenton (1998) performed shear wall tests to study the effects of static and cyclic loading. Eight of the twelve 8 x 8ft shear walls were tested cyclically using a proposed test standard developed by the Structural Engineers Association of Southern California (SEAOSC). Their research showed that the ultimate loads from static tests were slightly larger than from dynamic tests. However, the ductility of the wall from cyclic loading was between 34% and 42% less than the corresponding static ductility. Results suggest that actual load factors will be significantly smaller than the intended design value. The failure mode for monotonic loading was pullout of nails, while the failure mode from cyclic loading was nail fatigue.

Salenikovich (2000) tested fifty-six walls with various aspect ratios subjected to monotonic and cyclic loading. Three different anchorage conditions were tested, which consisted of (1) fully restrained using hold-down connectors, (2) shear bolts along the bottom plate, and (3) nailing along the bottom plate. When the walls were fully restrained, the performance of the wall did not depend on the number of full sized panels in the wall. Walls 4ft and longer developed the same strength on a unit length basis. However, 2ft walls experienced 50% lower stiffness and strength than the long walls. When the ends were not restrained from uplift, the performance of the walls depended on the wall size. Unrestrained walls depend solely on the sheathing-to-framing connection of the bottom plate and the racking is not distributed along the entire perimeter of the wall.

1.6.2 Models and Formulas

White and Dolan (1995) developed a finite-element program to analyze the monotonic and dynamic performance of wood shear walls. The program used a framing element, sheathing element, sheathing-to-framing connection element, and a sheathing-bearing element to model the behavior. The program matched the monotonic experimental results closely with the maximum strength of a walls subjected to

monotonic loading predicted within two percent. The program also correlated well with dynamic test results.

Ni et al. (1998) developed two methods to predict the shear capacity of walls with openings. The methods proved to be more accurate than Sugiyama and Matsumoto's (1994) method and considered the effects of vertical loads on the structure. One method considered the shear capacity of all the wall segments and components above and below all openings. The other method was simplified by neglecting the contribution of sheathing above and below the openings to the shear wall capacity. The simplified method was more conservative and easier to use, so the researchers proposed including it in the new code. Ten walls were tested using either monotonic or cyclic testing to validate their mathematical method. Their tests were unique because they tested the effects of vertical loads versus no vertical loads. They found that the maximum displacements were similar; however, the ultimate lateral load capacity was increased by about 15% when vertical loads were applied because the vertical loads reduce the stud uplift and affectively works as a hold-down.

1.6.3 Connection Tests

As the single most important component of a shear walls, the sheathing to framing connection has been the focus of many research projects. Soltis and Mtenga (1985) examined the strength of nailed wood joints subjected to dynamic loads. At small deformations, the increase in joint capacity due to a higher rate of loading was offset by decreased joint capacity due to load cycling.

Polensek and Bastendorff (1987) performed typical nailed joint tests to evaluate the stiffness moduli and damping ratios of the joints under a static cyclic load. Results from the study showed that nailed joints not only dissipate energy under earthquake loading but also contribute to the composite strength and stiffness of a structure. The authors also concluded that lumber species, sheathing material, and nail size were the factors that most influence the damping and stiffness of nailed joints.

Dean (1988) thoroughly investigated the cyclic behavior of sheathing-to-framing connections. He studied the dependence of the nail bending shape and nail withdrawal on the nail and sheathing dimensions.

Girhammar and Anderson (1988) studied the effect of load rate on nailed joints. Various thickness of members and angles of load to grain direction were tested. Results show an increase in dynamic strength relative to static strength but higher deformations in the wood. Dolan (1989) also tested nailed connections to study the effects of cyclic loading.

1.6.4 Other Parameters

The effects of using oversized OSB panels versus regular-sized panels were tested by Lam, et al. (1997) under monotonic and cyclic loading. From the tests, they observed that shear walls with oversized panels exhibit increased stiffness and lateral load capacity but had comparable ductility ratios. However, the regular-sized panels dissipate more energy under cyclic loading. The failure mode for monotonic tests was nail withdrawal while the failure mode for cyclic testing was nail fatigue. In the second phase of this study, the effects of oversized panels in walls with openings were studied (He, et al. 1999). As expected, openings in the shear walls reduced the shear strength because of a reduction in the effective sheathing area. Using the oversized panels effectively increased the racking resistance of the wall by a substantial amount compared to regular-sized panels with openings. The failure mode of walls with openings was different from the walls without openings. Instead of strictly nail fatigue as the cause of failure, panel fractures and nail withdrawal contributed to the failure of walls with openings.

Rose (1998) evaluated shear wall performance when the sheathing was fastened with pneumatically driven "short" (diaphragm) nails versus full-length hand-driven nails. The wall with short nails exhibited 5% less maximum shear load than the full-length common nails. However, energy dissipation of the walls with short nails was significantly more than obtained in similar tests with full-length nails. The majority of the shorter nails withdrew, as opposed to nail fatigue, which occurs with full-length nails. The increased energy dissipation may outweigh the reduction in shear strength in some situations.

Keith and Scaggs (1998) conducted tests to determine the effects of nail penetration and framing species on the current shear wall values given in the codes and used in design. The goal of their research was to make it simpler to effectively modify

the current design values for other framing species and also propose changes to include new minimum fastener penetration.

Dinehart et al. (1999) tested wood-framed shear walls with viscoelastic dampers and compared the results to normal shear walls. The walls with dampers showed a significant increase, as much as 59%, in the amount of energy dissipated when subjected to earthquake loads. The walls also displayed an increase in effective stiffness.

Salenikovich (2000) determined that low wood density of the framing reduced the nailed connection strength when subjected to cyclic loading. As a result, the shear wall strength and toughness was reduced.

1.7 High Aspect Ratio Walls

Dolan (1989) showed that the deflections of high aspect ratio walls are influenced by both flexure and shear, as opposed to longs walls where shear deformation controls. Hold-down connections used to anchor the end studs of the shear walls are also a critical detail for short, high walls. The hold-down connection should remain elastic throughout the loading.

Hanson (1990) tested 1.2m (4 ft) shear walls and 2.4m (8 ft) shear walls under cyclic loading to compare the properties of plywood and OSB sheathing. The loading schedule for the walls was ten cycles at +/-75%, 100%, and 150% of the design load, and five cycles at +/-200% and 300% of design loads. The 1.2m (4 ft) walls were tested at a rate of 0.1 Hz, while the 2.4m (8 ft) walls were tested at a rate of 0.05 Hz. Plywood and OSB performed similarly for the 1.2m (4 ft) shear walls with all walls surviving the loading schedule which was five cycles at +/- 300% of the design load. Hanson concluded that both sheathing materials were equivalent. For the 2.4 x 2.4m (8 x 8ft) shear walls, the OSB and plywood performed similarly up to +/-150% of design load. The OSB walls started to deflect more than the plywood wall at +/-250%. The OSB failed during the first cycle at 300% of the design load while the plywood failed during the third cycle at 300%.

Tall, narrow shear walls were tested under cyclic loads by Commins and Gregg (1994) to determine allowable loads for shear walls with a large aspect ratio of 3.5:1. The walls, which were 3 ½ times as tall as they were wide, were tested at design load for

90 cycles, and then at 140% of design load for another 90 cycles. The authors' hypothesis was that for walls with large aspect ratios, the hold-downs could have large effects on the drift of the wall compared to their minimal effect on square shear walls. They concluded that at the design load, the hold-down load was 121% of the connector's rated design load, and in all cases, the failure mode was fatigue of the nails about 3/8 in. below the surface of the wood stud. All walls recorded a larger drift than the code allowed.

Serrette et al. (1996) tested shear walls with plywood, OSB, and gypsum wallboard sheathing attached to light-weight steel framing. He determined that 4 x 8ft walls have the same shear strength as an 8 x 8ft wall subjected to static loading, provided that the panels have the same orientation to the framing.

Due to the need for a standard test procedure for cyclic loading, the TCCMAR procedure was developed. Commins and Gregg (1996) tested shear walls under this new testing protocol for shear walls 2ft, 4ft, 6ft, and 8ft long with a height of 8 ft. The walls were tested under cyclic loads and then compared to the allowable loads given in the Uniform Building Code (UBC). They determined that the 4ft and 8ft walls performed at 77% while the 2ft walls performed at only 56% of the allowable load given by the UBC. The 8 x 8ft wall behaved the same (on a per foot basis) as two 4 x 8ft walls. All walls failed by fatigue bending of the nails. The hold-downs also contributed to the performance of the shear walls and in particular to the behavior of the sheathing nails. For a stiff hold-down connection, most of the load is carried by the hold-down, which "protects" the plywood nailing. Conversely, with a flexible hold-down connection, much of the uplift was carried by the framing-to-plywood fasteners.

1.8 Test Procedures

Having consistent testing procedures is imperative for comparing values. If one researcher tests walls one way and another tests in a different fashion, it is nearly impossible to compare the results in a consistent manner. Since all testing laboratories are set up differently, it is much harder to replicate the tests performed elsewhere. ASTM standard test methods provide a universal guide that can be followed for a specific test. For shear walls, results from testing a typical-sized wall were used as a basis for

determining the design loads that are used when designing houses or other structures. The first tests that were performed were strictly monotonic and followed ASTM E72. Some people did not like this standard, and a more "real life" monotonic testing procedure was developed with ASTM E564. Then, after researchers questioned the use of a monotonic test to design for earthquake load, the need arose to test walls under seismic loads to understand their performance to this type of reversed, two-way loading. The following section discusses the pros and cons of each testing procedure, along with a comparison of the different tests.

1.8.1 ASTM E72 and ASTM E564

ASTM E72 and ASTM E564 are the two traditional monotonic test procedures used in the United States to evaluate the performance of shear walls. The older of the tests, ASTM E72, covers standard tests of walls, floors, and roof elements. This test is broader based than strictly a racking test (Skaggs and Rose, 1996). ASTM E72 was developed to reference sheathings against an accepted norm of 200mm deep by 25mm thick horizontal siding (Griffiths, 1984). The test method was designed to isolate the effects due to the sheathing panel, and not the behavior of the structural assembly or system.

Some notable features of ASTM E72 are the steel hold-down rods used to prevent the overturning moment and a lateral stop mechanism. Although the steel rod is intended to model the effects of vertical loads that would occur in a structure, many say that the steel rod does not represent true wall behavior and gives unrealistic failure values. Another feature is the stop at the end of the wall. This stop is intended to prevent any lateral slippage of the wall with regards to the test frame. Typically, the test procedure involves incremental loads applied and removed before taking the wall to failure. One problem that can arise from this is the load increments are specified regardless of the sheathing material. A strong material may never reach its design load under this procedure. The factor of safety that is used in design from this test usually averages close to three (Skaggs and Rose, 1996).

Due to the unrealistic failure values from the use of the steel rod, ASTM E72 was replaced with ASTM E564. This test method incorporates a hold-down device that

attaches the bottom of the frame to a rigid base with anchorage connections, which is more realistic of construction practice (Griffiths, 1984). As opposed to the older test method, ASTM E564 was developed to evaluate shear wall assemblies instead of focusing on the behavior of the sheathing. The "stop" is also removed from the ASTM E564, which utilizes the bottom plate bolts to prevent lateral slippage. This test method also provides the option of applying vertical loads. Applying no vertical loads is the most conservative approach because adding in-plane gravity loads would help reduce the uplift and allow the wall to resist more load. Some of the problems with this test method arise from the variation in the hold-down connectors that may be used for this test.

Due to the different boundary conditions incorporated with ASTM E72 and E564, they cannot be directly compared. The hold-down configurations are different, as are the lateral movement restrictions. ASTM E564 is an "assembly" test, and ASTM E72 is a panel test. Neither test gives a safe deflection limit, although it could easily be defined. Although the accepted design values are determined from these tests, it is not very clear how to do so (Skaggs and Rose, 1996).

1.8.2 Cyclic Load Testing

Researchers, engineers, and code officials began to question how well monotonic tests correspond to shear wall behavior when subjected to earthquake loading. After the Northridge Earthquake in 1994, the city of Los Angeles changed its building code to reduce the design load of shear walls by 25% if the design loads were developed from monotonic tests. This was done to achieve conservative design values until the experimental tests could be performed on walls subjected to cyclic loading. However, the problem arose of developing a universally accepted method for testing walls under cyclic loading. This is a hard concept on which to get unanimous agreement because all earthquakes behave differently and having one loading schedule would not be easy to simulate all types of earthquakes.

In 1994, the Structural Engineers Association of Southern California (SEAOSC) developed a cyclic loading schedule that could be used as a standard. The plans were finalized in 1996, but had not yet been submitted to ASTM for consideration (Skaggs and Rose, 1996).

The load cycle used by SEAOSC was based on a procedure developed by the joint U.S. and Japan Technical Coordinating Committee on Masonry Research (TCCMAR). TCCMAR developed the Sequential Phased Displacement (SPD) as a method to compare the parameters of research testing that are not subjected to real-time earthquake loading. The procedure involves a progression of increasing fully reversed cyclic loading displacements until the first major event (FME). This is followed by degradation and stabilization cycles until progressing to the next phase of higher displacement.

The procedure begins with three fully reversing cycles at progressively larger amplitude until the FME is reached. For the shear walls investigated in this study, the FME was taken to be when the wall yielded. After the first major event was reached, the SPD loading procedure was implemented. This consists of phases that have progressively increasing amplitudes. The 1st cycle in a phase consists of a larger displacement proceeded by three degradation cycles. This is followed by three stabilization cycles to complete the phase; then the procedure moves on to the next phase. The loading schedule and waveform graph is shown in Table 1.1 and Figure 1.2.

Table 1.1: The loading Schedule of SPD

Pattern	**Phase**	**Amplitude of Initial Cycle (% FME)**
1	1	25
	2	50
	3	75
2	4	100
	5	125
	6	150
	7	200

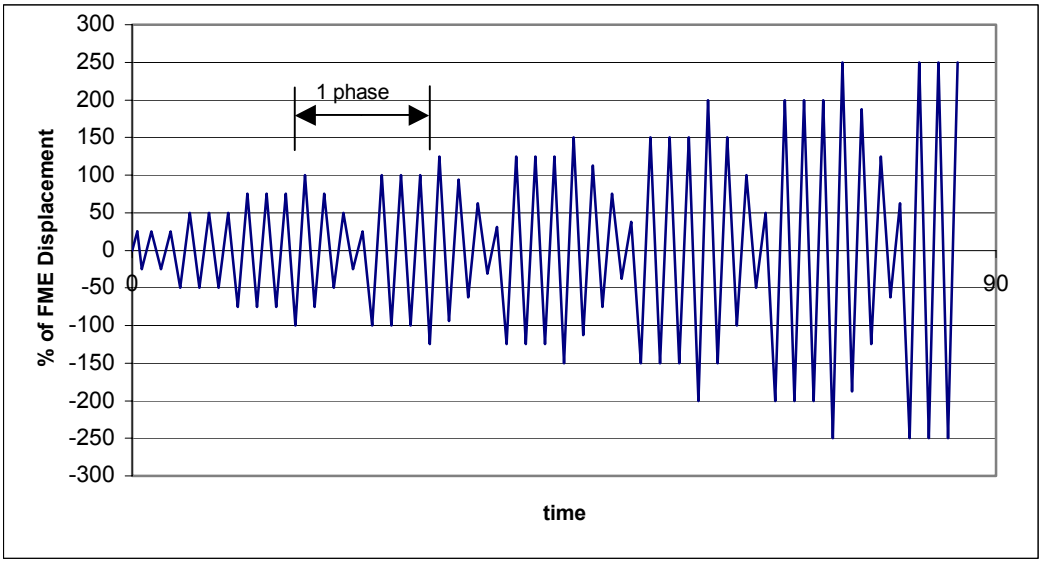

Figure 1.2: Waveform graph of TCCMAR SPD protocol

The SPD procedure more accurately represents an earthquake excitation pattern than the traditional monotonic or simple reversed cyclic loading patterns. Two main factors set the SPD procedure apart: the degradation cycles and the stabilizations cycles.

Degradation cycles are useful to define the lower points within a given hysteretic curve, incorporating the fact that most seismic events contain low energy cycles between major excitations (Porter 1987). They are also included to analyze the effect of systems that develop "slack." A slack system can be observed by watching the load from the load-displacement hysteresis loops approach zero during the degradation cycles. A slack system will lose the ability to dissipate energy at lower displacements. Slack systems do not generally occur in wood structural panel shear walls, but do occur for some bolted connections and some types of brittle systems.

Stabilization cycles are implemented to allow the strength and stiffness reduction to stabilize before increasing the displacement of the next cycle. This means that for a given displacement, the wall incurs as much damage as possible before moving on to a larger displacement. Stabilized curves should see degradation between successive cycles of no more than five percent before moving to the next displacement phase.

Another feature of the SPD procedure proposed by SEAOSC is the frequency at which the test should be performed. Researchers proposed that a testing frequency of 0.2–1.0 Hz should be used, which is lower than that typically observed during

earthquakes. This frequency was implemented because a lower one may limit the inertial effects that might occur during load tests. Also, a lower frequency is easier to obtain when using an effective force testing system. A force testing system, like the one used for this study, simulates dynamic loads of earthquakes by applying an "effective force to the wall." This system is used in lieu of shake-table testing.

1.8.3 Problems with Cyclic Test Procedure

One significant observation from tests performed using 10d common nails was that the nails around the perimeter exhibited fatigue. Because a majority of the cycles are above the FME, the wall damage is dependant on displacement and the number of cycles performed above FME. This indicates that the performance of the wall will be dependant on the load history. Nail fatigue was not observed in damaged shear walls from the Northridge earthquake. This finding is inconsistent with the SEAOSC procedure in which nail fatigue is often observed (Skaggs and Rose 1996).

1.8.4 Similar Testing Procedures

The modified TCCMAR procedure eliminates the degradation cycles that occur after FME. When compared to the original TCCMAR procedure, the deleted degradation cycles did not significantly affect the results of wood structural panel shear wall tests. However, there is indication that applying degradation cycles when using steel framing may provide useful information (Rose 1998).

1.8.5 ASTM E2126

The SPD procedure that was developed by TCCMAR and used by many researchers was adopted as ASTM standard test E2126. The basic concept of the testing protocol is similar to the SPD procedure with a few changes. ASTM E2126 uses an FME and the ductility ratio of the specimen. Instead of basing all of the initial displacements from the FME as the SPD procedure implies, the new standard uses the ductility factor times a constant to be used as a percentage of the FME.

1.9 Summary

A review of previous shear wall studies and testing procedures has been presented. A majority of the tests have involved either plywood or OSB, which are the two most widely used sheathing materials. The remainder of the thesis will focus on the use of other sheathing materials such as fiberboard and hardboard and study its effects under monotonic and cyclic loading. The contribution of gypsum wallboard to the racking resistance of shear walls will be studied. The effects of using hold-down anchors versus no hold-down anchors on walls with high aspect ratios also needs further investigation and will be examined in the following chapters.

Due to the discrepancy found when using ASTM E72, all of the monotonic tests were performed according to ASTM E564. ASTM E564 was developed specifically for determining the shear resistance of shear walls. The recently adopted testing standard for cyclic loading, ASTM E2126, was used for all of the cyclic tests in this study.

Chapter 2: Test Specimens, Procedures, and Definitions

2.1 Introduction

Forty-five walls subjected to monotonic and cyclic loading were tested during this study. A list of tests and test numbering is displayed in Table 2.1. All walls were 1.2 x 2.4m (4 x 8ft) with various sheathing materials. The sheathing materials examined are oriented strand board (OSB), hardboard, fiberboard, and gypsum wallboard (GWB). Four-foot walls were chosen because this is the minimum width for eight-foot high walls allowed by the U.S. Building Codes to resist wind or seismic loads. Also, this is the most typical width of fully sheathed wall segments used to fill the space between windows and doors in residential construction.

Table 2.1: Test configurations and test number

Material	Monotonic	Cyclic w/ Hold-down	Cyclic w/o Hold-down
OSB	1, 1a, 2, 2a	10 10a	17 18
Hardboard	3, 3a, 4	11a 12	19 20
Fiberboard	5 6	13 14a	21 22
Gypsum	7 8	15 16	23 24
OSB/GWB	25 26	31 32	37 38
Hardboard/GWB	27 28	33 34	39 40
Fiberboard/GWB	29 30	35 36	41 42

This study assumes that the sheathing material, sheathing nail, amount of overturning restraint, and type of loading will most influence the performance of the shear walls. This study also gathers baseline information on the performance of hardboard and fiberboard sheathing.

This chapter describes the test specimens and testing procedures. A discussion of the materials, construction details, instrumentation, and data acquisition system is included. This chapter also identifies the important shear wall parameters that define the behavior of the specimens.

When evaluating shear walls, the strength, stiffness, deformation characteristics, energy dissipation, and damping are the key parameters that are investigated. Nearly all of these characteristics can be determined from the load-displacement curve that is generated for each test. Ductility, yield load, and yield displacement are other properties that are of interest when evaluating shear walls. Many methods have been developed to determine how and when these properties can be calculated, and each method has its own valid base of reasoning. All wall parameters calculated in this study follow the definitions set forth by ASTM E2126 and E564.

2.2 Materials

This study incorporates several different sheathing materials. Although plywood and oriented strand board (OSB) are most widely used in construction, there is a desire to understand the performance of other sheathing materials such as fiberboard and hardboard. The contribution of gypsum wallboard acting alone or on the opposite side of the wall from another sheathing material is also investigated in this study. This section defines the sheathing panels used, which consist of OSB, hardboard, fiberboard, and gypsum. All sheathing panels were 1.2 x 2.4m (4 by 8ft). Also provided is a description of the framing materials and nails used in the wall specimens. All sheathing panels, framing materials, and nails were stored inside the laboratory under a controlled environment until ready to be tested. A list of the sheathing materials with their appropriate nail size and nailing schedule is given in Table 2.2.

Table 2.2: Sheathing materials and nailing schedule

Sheathing		**Nails**		
		Type	**Nail Spacing (o.c.)**	
Material	**Thickness**		**Edge**	**Field**
OSB	11mm (7/16 in.) per US VPA DOC PS-2	**8d common** (ϕ3.33mm x 63.5mm long) (ϕ0.131" x 2 ½" long)	152mm (6in.)	305mm (12in.)
Hardboard	9mm (3/8 in.) per ANSI/AHA 135.4 and 135.6	**6d box** (ϕ2.5 x 51mm long x 6.8mmϕ head) (ϕ0.099" x 2" long x 0.266"ϕ head)	102mm (4in.)	203mm (8in.)
Fiberboard	12mm (½ in.) per ASTM C209	**11ga. Galv. roofing nail** (ϕ3 x 38mm long x ϕ9.5mm head) (ϕ0.12" x 1 ½"long x ϕ3/8" head)	102mm (4in.)	152mm (6in.)
Gypsum (GWB)	12mm (½ in.) per ASTM C36	**11ga. Galv. roofing nail** (ϕ3 x 38mm long x ϕ9.5mm head) (0.12"ϕ x 1 ½"long x 3/8"ϕ head)	178mm (7in.)	406mm (16in.)

2.2.1 Sheathing Panels

Oriented strand board (OSB) has been the focus of many research projects. Therefore, in this study, it is used for comparison to the other sheathing materials. OSB is a nonveneer panel manufactured from reconstituted wood strands or wafers. The strand-like or wafer-like wood particles are compressed and bonded with phenolic resin. The wood strands are directionally oriented in a manner to provide strength in the desired direction (Breyer et al. 1999). The OSB panels used for these tests were 11mm (7/16 in.) thick, purchased from a local lumberyard.

The hardboard was 9mm (3/8 in.) sheathing manufactured and delivered by ABTCO Hardboard Siding. Hardboard is a panel manufactured primarily from inter-felted lingo-cellulosic fibers, which are consolidated under heat and pressure in a hot-press to a density of 500 kg/m^3 (31 lbs/ft^3) or greater. Other materials can be added to improve certain properties such as stiffness, hardness, finishing properties, resistance to abrasion and moisture, as well as to increase the strength and durability of the product. Since hardboard is a wood-base material, its moisture content will vary with environmental humidity conditions. The moisture content of hardboard should not be less than two percent or more than 9 percent (AHA 1995).

According to ASTM C208 (2001), fiberboard structural panels are a fibrous-felted homogeneous panel made from lingo-cellulosic fibers, usually wood or cane. It has a density of less than 497 kg/m^3 (31 lbs/ft^3), but more than 160 kg/m^3 (10 lbs/ft^3). Fiberboard is characterized by an integral bond that is produced by interfelting of the fibers, and has not been consolidated under heat and pressure in a separate manufacturing stage. Other materials may be added during the manufacturing process to improve certain properties of the fiberboard. The fiberboard used for this study was 12mm (½ in.) panels produced by Georgia Pacific.

The fourth sheathing material used was gypsum wallboard (GWB). GWB is also commonly referred to as drywall or sheet rock. The performance of gypsum in this study was investigated by itself and as a sheathing material added on the interior side of the wall. Gypsum wallboard consists of a gypsum plaster core, covered on both surfaces with a paper veneer. The plaster core is brittle in nature, but the paper veneer provides strength and stiffness to resist racking forces (Wolfe 1983).

2.2.2 Sheathing Nails

Past research projects have shown that nail load-slip behavior is the single most important parameter of shear walls. The sheathing-to-framing connection is where shear walls fail when subjected to lateral forces. Rarely does the sheathing fracture or the framing elements fail when testing a shear wall. Therefore, the type of nail, nail spacing, and nail edge distance are very important when constructing shear walls. In experimental testing, the walls should be as close to common construction practice as possible, enabling the results to be accurately compared to "real life" applications. The nail schedule used for this study followed industry standards and is given in Table 2.2. The edge distance provided for this study was 19mm (¾ in.), which is ideal for nailing into a 38mm (1½ in.) wide stud. An edge distance of 9mm (3/8 in.) is often used when two sheathing panels are attached to one stud, but the walls investigated in this test only had one sheathing panel, enabling a 19mm (¾ in.) edge distance around the perimeter of the wall.

All nails were hand driven except for when OSB panels were used. The OSB panels used 3.33mm x 63.5mm (0.131 in. diameter x 2 ½ in. long) 8d common nails, with

152mm (6 in.) o.c. spacing around the perimeter and 305mm (12 in.) o.c. field spacing. The 8d common nails were full headed, pneumatically driven, SENCO nails purchased from a local hardware store. The hardboard panels called for 2.5mm x 51mm long x 6.8mm head (0.099 in. diameter x 2 in. long x 0.266 in. diameter head) 6d box nails with 102mm (4 in.) o.c. edge spacing and 203mm (8 in.) o.c. field spacing. The GWB and fiberboard sheathing used galvanized roofing nails 3mm x 38mm long x ϕ9.5mm head (0.120 in. diameter x 1 ½ in. long x 3/8 in. head diameter). The nail schedule of the gypsum panels consisted of 178mm (7 in.) o.c. edge spacing and 406mm (16 in.) o.c. field spacing. The nail schedule of the fiberboard panels was 102mm (4 in.) o.c. edge spacing and 152mm (6 in.) o.c. field spacing.

2.2.3 Framing Members

All framing members used to construct the walls were stud or better Spruce-Pine-Fir (SPF). The dimensions were 38 x 89 mm (1.5 in. x 3.5 in. nominal dimensions). All framing materials were purchased from a local hardware store and kept in the laboratory under a controlled environment. The moisture content of the studs ranged from 12-15%. The double top plates and the bottom plates were cut to four feet, and the studs were cut to 2324mm (91 ½ in.) so that the total height of the wall was eight feet as shown in Figure 2.1. The framing components and materials are listed in Table 2.3.

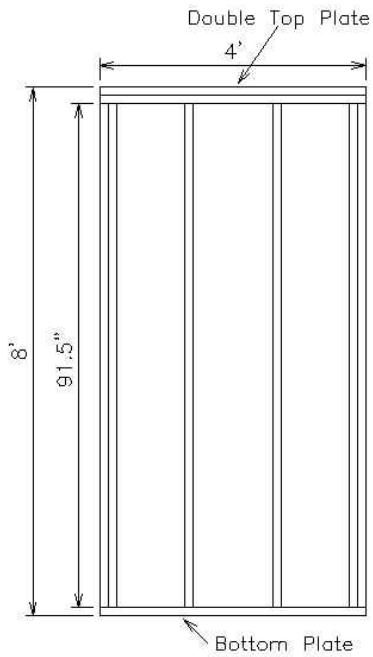

Figure 2.1: Shear wall dimensions

Table 2.3: Framing components and materials

Component	Fabrication and Material
Framing:	Stud, Spruce-Pine-Fir, 38 x 89 mm (1.5 x 3.5 in.) (nom.) @ 406mm (16 in.)o.c.
	Connection:
Plate to Stud	(2) 16d common per foot
Plate to Plate	(2) 16d common per foot
Stud to Stud	(2) 16d common each end
Sheathing:	
Exterior	OSB, Hardboard, Fiberboard, or Gypsum
Interior	Some walls sheathed with Gypsum on Interior
Tie Down:	USP HTT22, nailed to end studs with (32) 16d (ϕ3.8 x 82.6mm) sinker nails, 15.9mm diameter (ϕ5/8") bolt to connect to foundation
Anchor Bolts:	16mm ϕ (5/8in. diameter) A307 bolt with 63.5 x 63.5 x 6.35mm (2.5 in. square x 1/4 in.) steel plate washers.

2.3 Fabrication of Walls

All walls were constructed and stored inside the laboratory until it was time to test. The walls were constructed using a steel frame that was welded to the desired

dimensions. The frame enabled a square wall that could easily be nailed together with the correct spacing. Pictures of the test frame are shown in Figures 2.2 and 2.3.

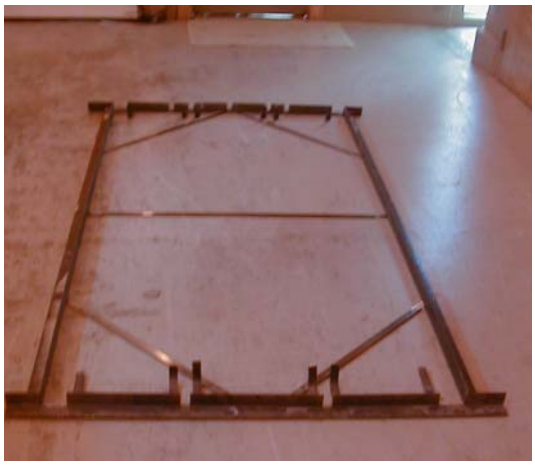

Figure 2.2: Construction frame

Figure 2.3: Frame with wall in place

A typical shear wall is constructed as shown in Figure 2.4. The end studs were put together using two 16d common nails spaced every foot. For the walls that included hold-downs, the hold-downs were installed on the end studs before being placed in the test frame to avoid awkward nailing positions. The hold-downs were placed about 38 to 51mm (1 ½ to 2 in.) from the bottom of the end studs as shown in Figure 2.5. Hold-downs were connected with thirty-two 16d sinker nails that were 72mm (3 in.) long. End studs and intermediate studs were then placed into the test frame and nailed to the top and bottom plate. The stud-to-plate connection consisted of two 16d common nails on each end. The top plate consisted of two framing members that were connected with two 16d common nails every foot. Holes in the top and bottom plate used to connect the wall to the testing apparatus were predrilled to avoid having to drill the holes after construction was complete. A template was used that matched the holes on the test frame, which helped align the wall and make installation easier.

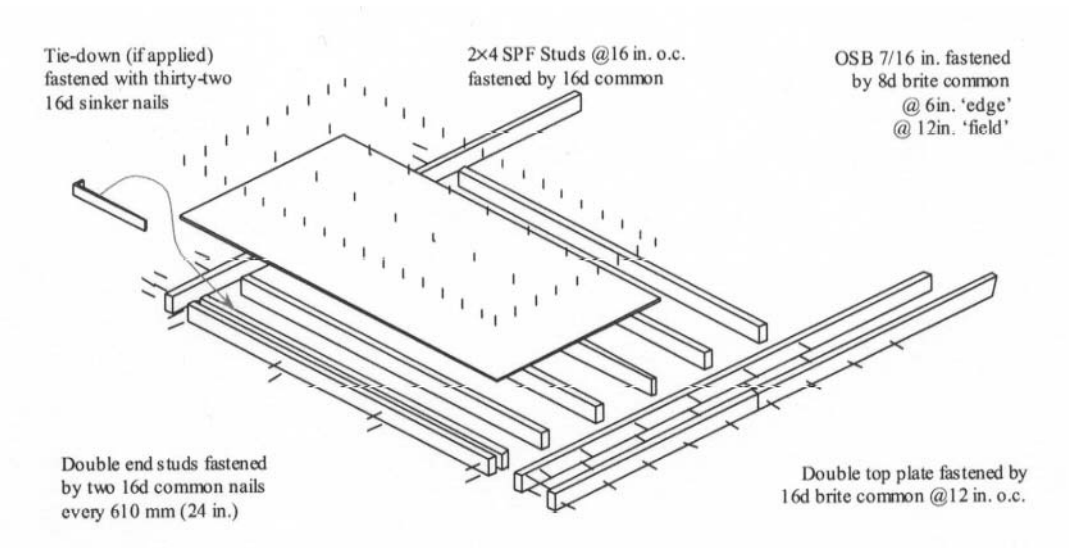

Figure 2.4: Typical construction of a shear wall (Courtesy of Heine 1997)

Figure 2.5: Hold-down connector

After the frame was constructed, the sheathing panel was attached to the frame. All of the sheathing nail positions were premarked at the ¾" edge spacing for consistency. The nails were driven flush with the sheathing material as specified. The sheathing was placed on the bottom side of the framing in regards to how the walls were placed in the testing apparatus. This method allowed for easy installation of the anchor bolts; it also allowed for the sheathing to fall off during the tests if it became separated from the framing. For the walls sheathed with dissimilar materials on opposite sides of

the wall, the gypsum was placed on top and nailed to the framing after the wall was bolted into place.

2.4 Test Setup

The walls were tested in a horizontal position as shown in Figure 2.6. No dead load was applied in this setup, which is a conservative representation of a typical shear wall. The bottom plate of the wall was attached to a 76 x127mm (3 x 5 in.) steel beam with 16mm (5/8 in.) anchor bolts spaced at 610mm (24 in.) on center. For the walls with hold-downs, 16mm (5/8 in.) bolts were used to attach the hold-down to the steel beam. All holes in the bottom plate were drilled 1.5mm (1/16 in.) larger than the bolt diameter. To avoid interference of the sheathing with the support, the narrow face of the beam 76mm (3 in.) was in contact with the bottom plate 89mm (3.5 in.). This arrangement allowed the sheathing to move over the top of the steel beam without meeting any resistance, which would add false loads on the wall. Another 76 x 127mm (3 x 5 in.) steel beam was attached to the top plate with 16mm (5/8 in.) bolts spaced 610mm (24 in.) on center. The top steel beam was attached to the programmable hydraulic actuator, which provided the racking force to the wall.

Figure 2.6: 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 2.7. The hinges are present to release any moment that could be applied from the weight of the actuator. 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.

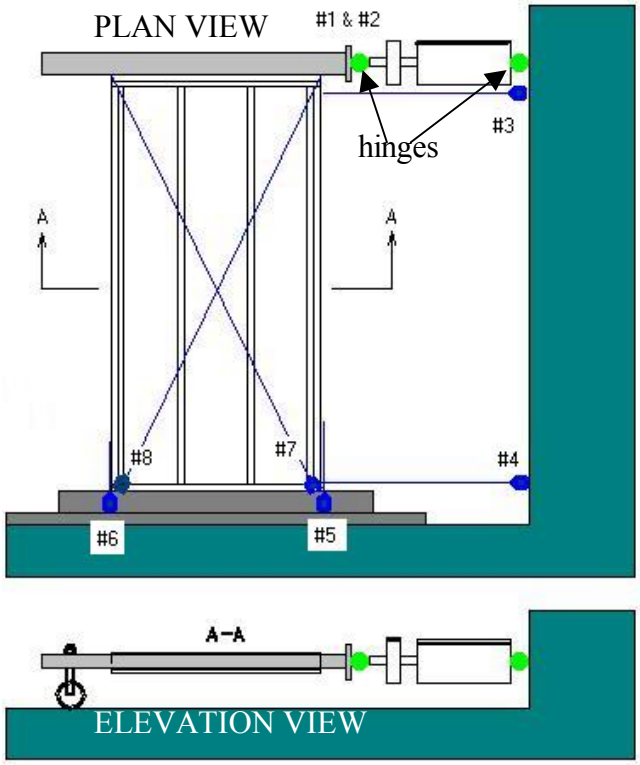

Figure 2.7: Plan and elevation view with LVDT locations

Table 2.4: Channel numbers with appropriate measurement

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

2.5 Instrumentation and Data Acquisition

Instrumentation and recording of data are crucial for any testing. For a researcher to duplicate or verify someone else's work, the details of the tests must be properly specified. In this investigation, eight channels of the data acquisition system were used to take readings on the walls. The channel number and its corresponding measurement are given in Table 2.4. An internal linear variable differential transducer, LVDT, and the 55 Kip load cell in the actuator took measurements at the top of the wall. Resistance potentiometers (string pots) measured the deflection at critical locations of the walls.

The eight channels of the data acquisition system used for this wall are shown in Figure 2.7. 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) verified the displacement from channel #1. Channel #4 measured the horizontal displacement of the bottom plate. Any slippage of the bottom plate had to be subtracted from the top displacement in order to determine 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. When the top of the wall is displaced, it causes an overturning moment at the base that causes the wall to lift off the foundation and produces tension or compression in the bottom plate, resulting in some deflection. Uplift from the walls with hold-downs was minimal, but when hold-downs were not present, the resulting uplift was quite large when the end studs pulled away from the bottom plate. This measurement is necessary when considering rigid body motion of the structure.

Channels #7 and #8 measured the diagonal displacement of the walls between the top and bottom plates. The diagonal displacements were taken to obtain information on the shear deformation as discussed in ASTM E564.

2.6 Testing Procedures

All walls were either tested using ASTM E564 for monotonic loading or ASTM E2126 for cyclic loading. All monotonic walls were tested with hold-downs, but half of the cyclic walls were tested with hold-downs and half without hold-downs.

2.6.1 Monotonic Testing

All monotonic tests were in accordance with ASTM E564. 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 152mm (6 in.). Unlike the specifications in ASTM E72 and E564, the unloading phases were omitted as the load was applied continuously until failure. This procedure was done because of the limited information that the unloading phases provide. For the data collection, ten readings per second were recorded during the test.

2.6.2 Cyclic Testing

All cyclic tests were in accordance with ASTM E2126 Method A. ASTM E2126 follows closely the SPD procedure developed by TCCMAR (Porter 1987) with a few changes. The SPD procedure is based solely on the FME, while ASTM E2126 takes into account the FME and ductility ratio when determining the initial amplitude of each phase in the protocol.

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 FME is defined as the displacement at which the structure starts to deform inelastically (anticipated yield displacement). The ductility ratio (displacement at the failure load divided by the displacement at the yield load) is represented by the following equation.

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

The loading schedule and waveform pattern for a typical wall are shown in Table 2.5 and Figure 2.8. FME and the ductility ratio values (μ) for each different sheathing material are listed in Table 2.6. From the loading schedule defined by ASTM E2126, and shown in Table 2.5, 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 situation occurred, the phase was not included when forming the envelope curves and only increasing displacement amplitudes were included.

Table 2.5: Amplitude of initial cycle

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

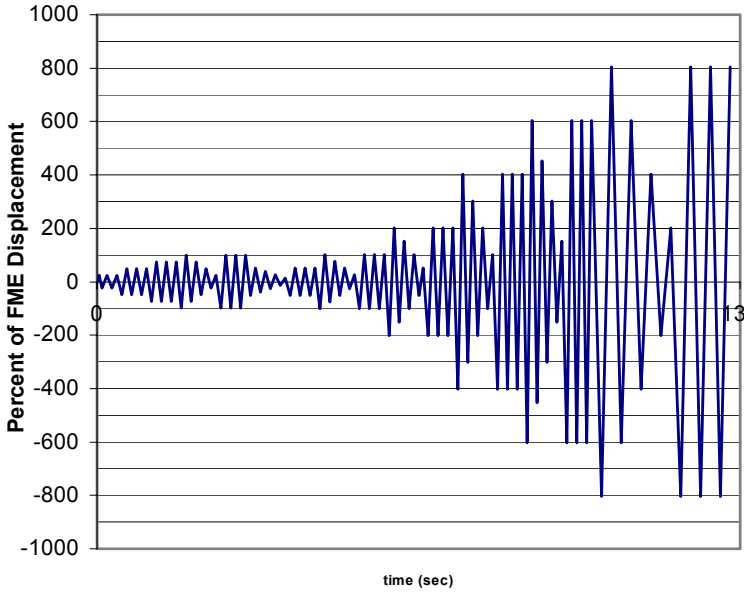

Figure 2.8: ASTM E2126 waveform pattern

Table 2.6: FME and ductility ratio values (μ) for wall specimens

Sheathing Material	FME (in.)	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 80μ. At this cycle, the frequency was changed to 0.25 Hz. This change was made 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 Section 2.8. 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. These values are only 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 gypsum 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 their corresponding anchored walls. This was done for comparison purposes and because all of the monotonic tests were performed with hold-downs.

Shear Wall Property Definitions

2.7 Load-Displacement Curve

For every test, a load-displacement curve was produced from the data obtained by channels #1 and #2. Nearly every parameter of shear walls can be obtained from this

graph. 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 0.75 mm) so it was not subtracted from the total wall displacement.

The load-displacement graph for monotonic tests as illustrated in Figure 2.9 is always positive and produces a curved line characteristic of its one directional loading. Cyclic tests produce a load-displacement graph similar to Figure 2.10. The graph 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 the curve from the monotonic load-displacement graph. The difference between these curves allows quantifying the strength and stiffness degradation from subjecting the shear wall to fully reversed loading cycles. Because the cyclic tests include positive and negative sides, the average of the absolute values are used in the analysis of the walls. The purpose of positive and negative cycles is based on the assumption that a structure must be able to resist fully reversed seismic excitations of equally large amplitudes.

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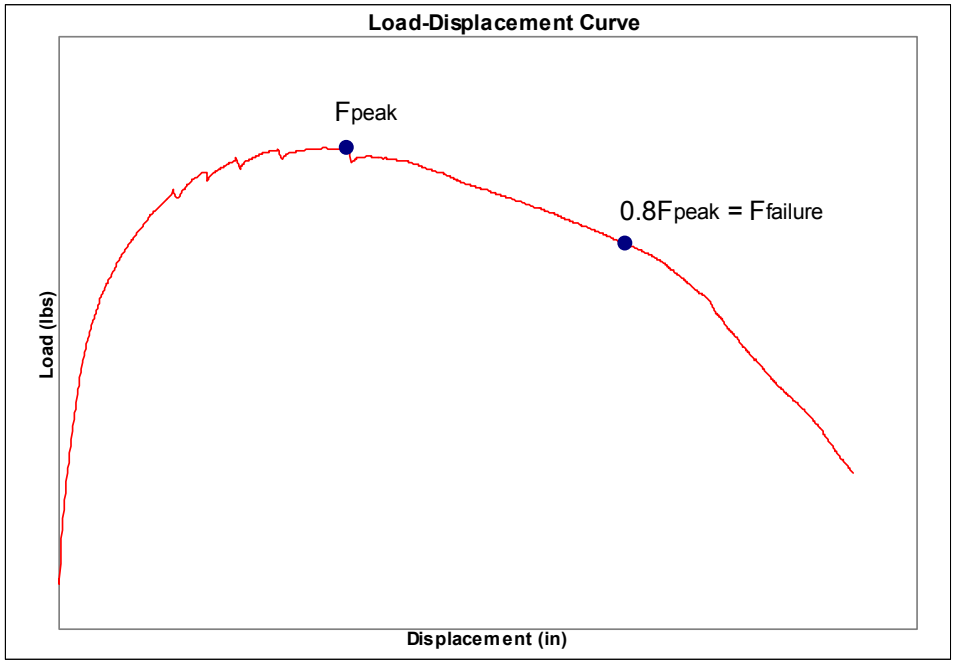

Figure 2.9: Typical monotonic load-displacement curve

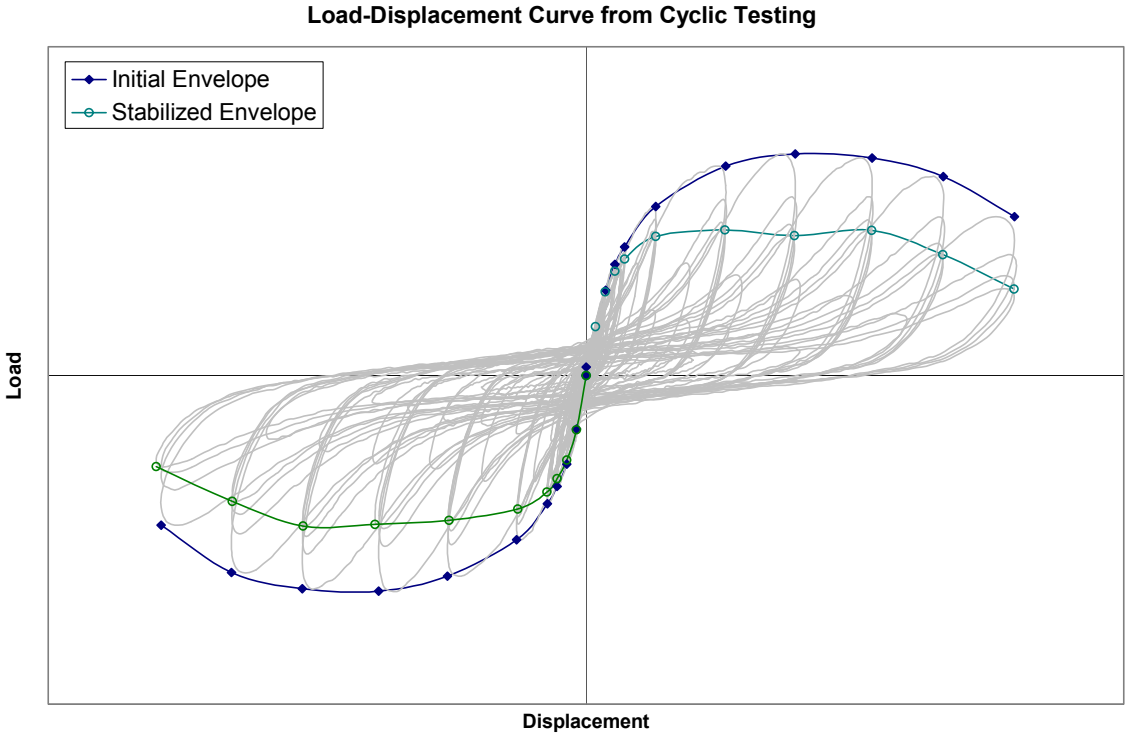

Figure 2.10: Typical hysteretic loops when subjected to cyclic loading

The stabilized envelope curve connects all of the peak loads from the last cycle of each loading phase in the loading procedure. The purpose of using stabilization cycles in the loading procedure is discussed in Chapter One, Section 1.8.2.

2.7.1 Wall Capacity

The wall capacity, 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.

2.7.2 Wall Failure

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) on the descending portion of the load-displacement curve. 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.

The failure displacement is recorded and used to determine the ductility of the structure. The more a structure can deflect before failure and the more load it can resist at failure are important to the integrity of the structure. It is crucial that a shear wall be able to deflect by a significant amount to withstand the ground motions produced by an earthquake.

2.7.3 Energy Dissipation

A structure must be able to sustain large deformations and 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. For cyclic tests, the total energy dissipated is the area under the envelope curve as shown in Figure 2.10. This 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.

2.8 EEEP Parameters

Light-frame wood construction has a significantly different load-displacement reaction than other building materials, such as steel, which exhibit a nearly perfect elastic-plastic response when loaded as a coupon. Light-frame wood construction does not have a distinct yield load, and the proportional limit cannot be definitely set. Several definitions have been 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 as illustrated in Figure 2.11. An EEEP curve is a perfectly elastic plastic representation of the actual response of the specimen. The EEEP curve encompasses the same amount of area as the actual load-displacement curve from the origin to the ultimate displacement. This area is a measure of the toughness of the system. Toughness is the energy that is needed to 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 structure. 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.

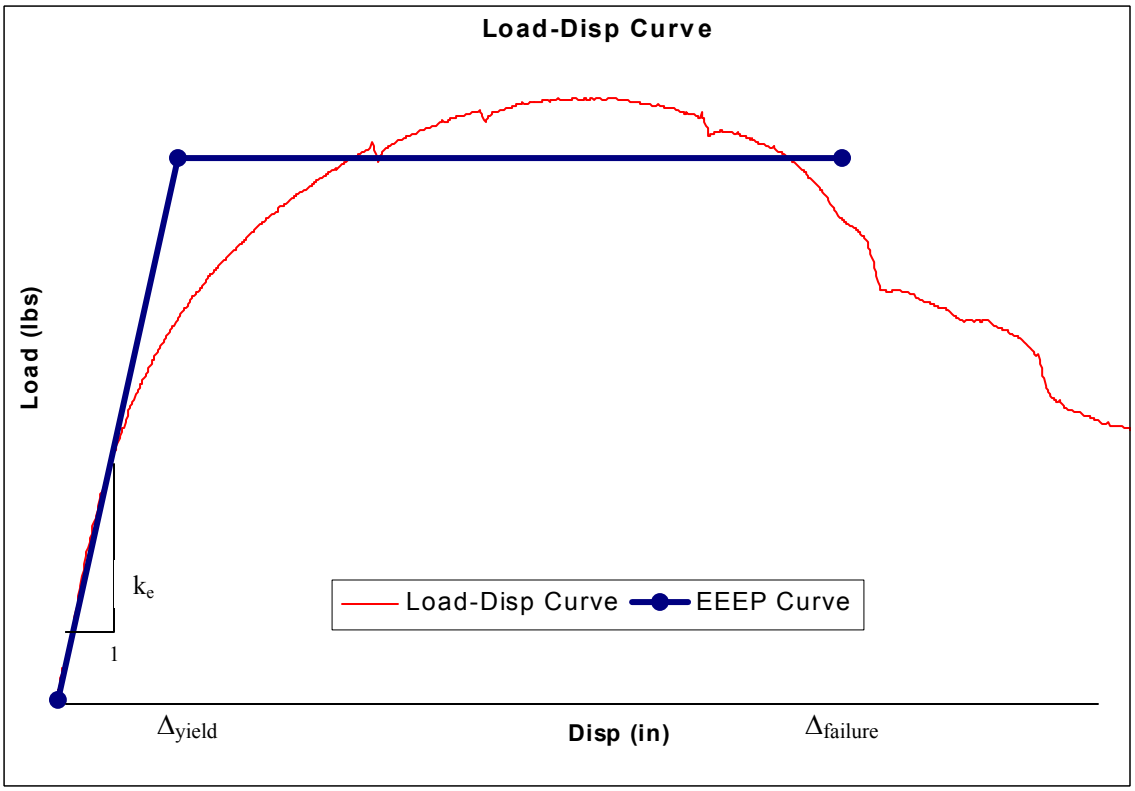

Figure 2.11: EEEP curve with parameters

2.8.1 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% 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. It is a compromise between the ASTM and CEN standards. The elastic stiffness is a good representation of the stiffness that a wall would exhibit when subjected to low to moderate displacements (Salenikovich 2000).

2.8.2 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 as illustrated in Figure 2.11. 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 be calculated as follows:

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

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 following relationship:

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

2.8.3 Ductility

Ductility is an important feature of a structural system, which enables it to yield and deform inelastically without failure. The ability of walls to bend but not break is crucial when subjected to the sudden and powerful motions of earthquakes. Several methods have been proposed 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 assumes 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 structure's 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, a large margin for error and inconsistent results exists.

2.9 Earthquake Performance Indicators

2.9.1 Damping

Damping is the process by which free vibration of a structure steadily diminishes in amplitude. Often times the energy of the vibrating system is dissipated by various mechanisms. In a typical structure, some common mechanisms that contribute to energy dissipation are 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 structure to an equivalent viscous system. The energy dissipated by the actual structure 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 2.12.

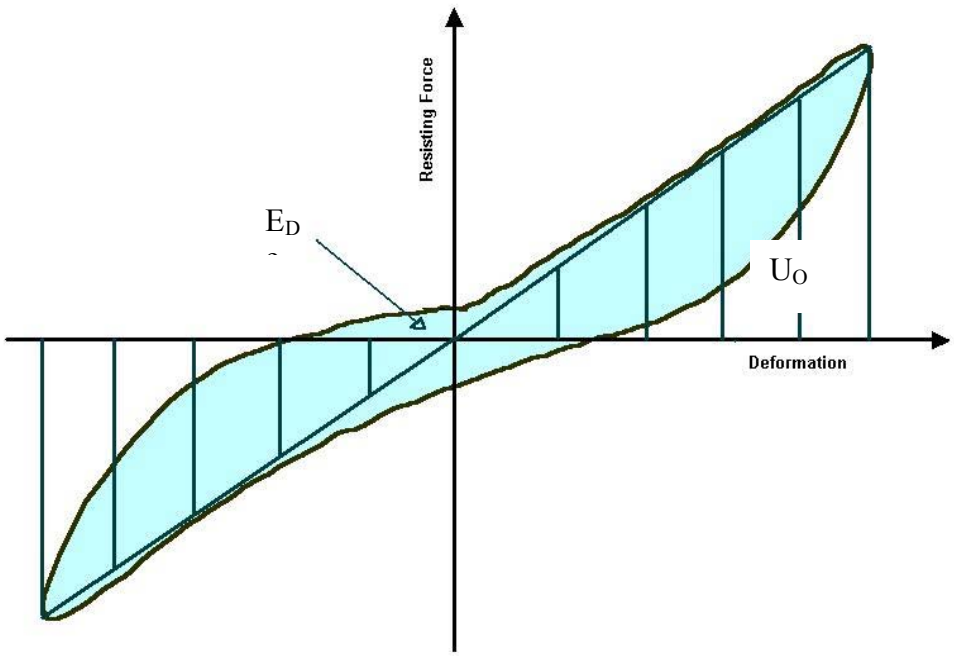

Figure 2.12: Typical stabilization loop used to determine damping ratio

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
E_D = hysteretic energy (area of load-disp. curve)
U_o = Strain Energy (area of triangle)

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. To be as accurate as possible, the areas 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 walls in this study.

2.9.2 Cyclic Stiffness

The average cyclic stiffness, k_c, is determined by calculating the average slope of the maximum positive and negative load values for the initial cycle 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.

2.10 Summary

This chapter provides a description of the test specimens and testing procedures. A discussion of the materials, construction details, instrumentation, and data acquisition system is included. This chapter also identifies and defines the important shear wall parameters that define the behavior of the specimens. When evaluating shear walls, the strength, stiffness, deformation characteristics, energy dissipation, and damping are the key parameters. Ductility, yield load, and yield displacement are other properties that are of interest when evaluating shear walls.

Chapter 3: Monotonic Test Results

3.1 General

A total of seventeen monotonic tests were performed for this study. All specimens were tested according to ASTM E564-95 (2000), which requires the use of a hold-down devise. Hold-downs are attached to the end studs by nails and bolted to the foundation to resist the overturning moment produced by the racking force. USP HTT22 hold-down connectors were used for all tests. All walls were 1.2 x 2.4m (4 x 8ft). This size was chosen because it is the minimum length allowed for wood structural panels to resist earthquake loads without further reduction in design values. It was shown by Salenikovich (2000) that walls 1.2m (4ft) and longer developed the same strength, on a unit length basis, provided the wood density of the studs was equal.

Four walls were sheathed with OSB, three were sheathed with hardboard, and only two were tested with every other sheathing configuration as listed in Figure 3.1. 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 in-plane gravity loads to the structure after the 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.

Sheathing Material	Test Number		Sheathing Material	Test Number
OSB	1, 1a, 2, 2a		OSB/GWB	25, 26
Hardboard	3, 3a, 4		Hardboard/GWB	27, 28
Fiberboard	5, 6		Fiberboard/GWB	29, 30
Gypsum	7, 8			

Figure 3.1: Monotonic sheathing configurations with test numbers

The complete results for each monotonic test can be found in Appendix A. A description of the test observations and mode of failure are given for each specimen in

Appendix A. A listing of every desired shear wall parameter is given in tabular form. Graphs of the load-displacement curve are also provided in Appendix A.

The main focus of this chapter is comparing the sheathing materials and investigating the contribution of gypsum wallboard (GWB) when subjected to monotonic loading. The parameters investigated include ultimate load, elastic stiffness, yield load and displacement, failure displacement, ductility, and energy dissipation.

3.2 Comparison of sheathing materials

3.2.1 Load-Displacement Relationship

A typical load-displacement graph for each material tested is shown in Figure 3.2. The two data sets for each sheathing material were averaged to determine the load-displacement values. The graph reveals that the performance of the wall is dependant on the sheathing material and its corresponding nailing schedule. It shows that each sheathing material reaches its maximum strength at a different load and fails at a different load and displacement. Therefore, it is important to look at several different parameters when analyzing the various sheathing materials.

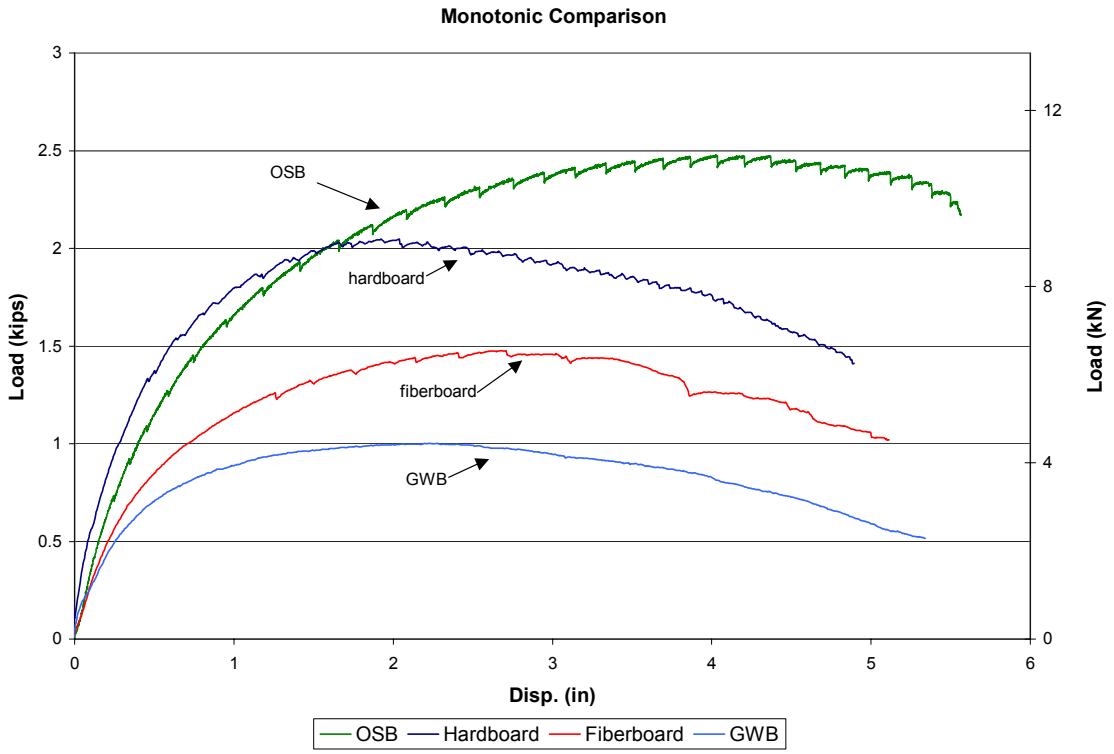

Figure 3.2: Load-Displacement graph of monotonic tests sheathed on one side

3.2.2 Ultimate Load

Ultimate load is taken at the largest point on the load-displacement graph. Typically, allowable stress design values are determined from the ultimate load with a factor of safety applied to give a safe design value. A list of the ultimate load for each sheathing material is given in Table 3.1 and a graphical representation is shown in Figure 3.3. It was observed that OSB achieved the maximum load at an average of 11.16 kN (2.51 kips). The next strongest sheathing material was hardboard, which was able to resist 9.26 kN (2.08 kips). Fiberboard resisted 6.75 kN (1.52 kips), while gypsum wallboard resisted an average maximum load of 4.46 kN (1.0 kips). For each set of tests, the peak load values were in close agreement. The coefficient of variation (COV) was 7.5%, 4.7%, 3.9%, and 5.8% for OSB, hardboard, fiberboard, and gypsum, respectively.

Table 3.1: Monotonic peak load values for sheathing materials

OSB	Load (kN)	Load (kips)	Unit Load (kN/m)	Unit Load (kip/ft)
Test 1	10.85	2.44	8.90	0.61
Test 2	11.68	2.63	9.58	0.66
Test 1a	10.14	2.28	8.32	0.57
Test 2a	11.99	2.70	9.83	0.67
Average	**11.16**	**2.51**	**9.16**	**0.63**
Hardboard				
Test 3	9.52	2.14	7.81	0.54
Test 4	8.76	1.97	7.19	0.49
Test 3a	9.51	2.14	7.80	0.53
Average	**9.26**	**2.08**	**7.60**	**0.52**
Fiberboard				
Test 5	6.94	1.56	5.69	0.39
Test 6	6.57	1.48	5.39	0.37
Average	**6.75**	**1.52**	**5.54**	**0.38**
Gypsum				
Test 7	4.28	0.96	3.51	0.24
Test 8	4.64	1.04	3.81	0.26
Average	**4.46**	**1.00**	**3.66**	**0.25**

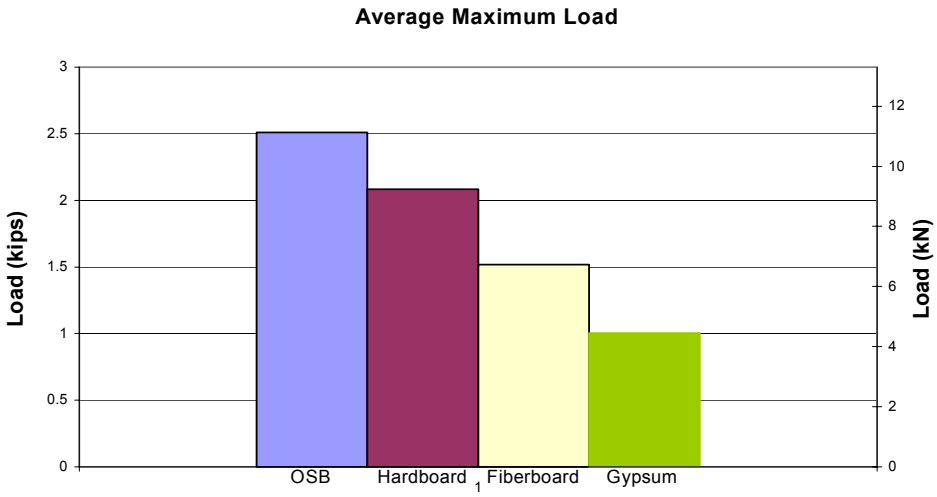

Figure 3.3: Graph of monotonic peak loads

3.2.3 Yield Load and Displacement

Yield load determined for the walls in this study was an approximation of the first major event, which is the theoretical load and displacement at which the structure starts to deform inelastically. Because wood does not behave in linear elastic manner, the yield load is only an approximation determined from Section 2.8.2. The yield load is a function of the elastic stiffness, failure displacement, and the energy dissipated.

As defined in Chapter Two, the elastic stiffness is the slope of the secant line that passes through the origin and the load that is equal to $0.4F_{peak}$. There was 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 structure. An initial load was sometimes inadvertently applied to a wall when it was attached to the test frame and the bolt holes did not match perfectly. This was actual load on the structure, so it could not be neglected. The initial load caused the elastic stiffness to increase because the wall reached a higher load than a wall without an initial load at a given displacement. An example of this issue is shown in Table 3.2. As the initial load increased, the elastic stiffness also increased.

Table 3.2: Effects of initial load on the elastic stiffness

Hardboard	Elastic Stiffness		Initial Load		COV
	(kN/mm)	(kip/in)	(kN)	(lb)	
Test 3	0.8	4.59	0.93	210	13%
Test 3a	0.62	3.57	0.85	19	
Test 4	0.77	4.39	0.36	80	

As expected, gypsum and fiberboard panels had the lowest elastic stiffness. Gypsum wallboard had an elastic stiffness of 0.39 kN/mm (2.22 kip/in), while fiberboard had an elastic stiffness of 0.39 kN/mm (2.18 kip/in). Due to the large variation in elastic stiffness of OSB and hardboard, it is not conclusive which material has a higher elastic stiffness. As determined from the tests, OSB-sheathed walls had an average elastic stiffness of 0.76 kN/mm (4.31 kip/in), and the hardboard-sheathed walls had an elastic stiffness of 0.73 kN/mm (4.18 kip/in), but the COV was 69% and 13%, respectively.

The yield load of the specimens followed the same pattern as the peak load with OSB achieving the highest, followed by hardboard, fiberboard, and gypsum. The average yield loads are shown in Table 3.3. The yield displacement, which is determined directly by the elastic stiffness and the yield load, is also shown in Table 3.3.

Table 3.3: Monotonic yield load and displacement values

Material	Yield Load		Yield Displacement	
	(kN)	(kip)	(mm)	(in.)
OSB	9.96	2.24	17.5	0.69
Hardboard	8.4	1.89	11.6	0.46
Fiberboard	6.0	1.36	15.9	0.63
Gypsum	4.1	0.92	10.5	0.42

3.2.4 Failure Capacity and Displacement

As defined in Chapter Two, the failure load was taken to be $0.8F_{peak}$. Because this is obviously based on the maximum load, the walls with the highest strength will also have the largest load capacity at failure. However, the displacement capacity of a

structure is an important parameter to investigate. The ability of a structure to dissipate more energy results from it being able to deform without failing.

From Table 3.4, it can be seen that the OSB walls were able to withstand the largest force and displacement at failure when subjected to monotonic loading. Although hardboard had more strength than fiberboard, the displacement at failure was nearly the same. Gypsum was the weakest material and also failed at the lowest displacement.

Table 3.4: Failure load and displacement values

	Failure Load		**Failure Displacement**	
Material	**(kN)**	**(kips)**	**(mm)**	**(in.)**
OSB	8.94	2.01	142	5.6
Hardboard	7.38	1.66	117	4.6
Fiberboard	5.38	1.21	117	4.6
Gypsum	3.56	0.8	104	4.1

The failure mode of the walls typically involved the sheathing nails either pulling out of the framing or tearing through the sheathing along the bottom plate. Although the nails in the hardboard panels were spaced at 102mm (4 in.) on center, the material failed at 25mm (1 in.) less than OSB panels. When testing hardboard-sheathed walls, the ϕ2.5 x 51 long x 6.8mmϕ head, 6d box nails were observed to withdrawal from the framing with relative ease, partly due to the low lateral stiffness of the nails. When the pullout became significant, the sheathing panel could no longer resist the shear forces and the wall failed. OSB-sheathed walls were observed to experience withdrawal from the framing and tear-through of the sheathing. However, this did not occur until a larger displacement, which enabled the wall to resist a higher load through a larger displacement.

Fiberboard and gypsum walls were connected using the same nails, but fiberboard had a perimeter spacing of 102mm (4 in.), while gypsum had a perimeter spacing of 178mm (7 in.). Sheathing nails in the gypsum panels were observed to tear through the panels with ease. This not only drastically reduced gypsum's ability to resist shear but also enabled the wall to fail at a smaller displacement. Nails in the fiberboard walls would also tear through the sheathing with ease, but fiberboard walls were able to resist more shear through a larger displacement than gypsum walls. Due to the different nail

spacing, it is uncertain whether fiberboard panels have a larger displacement capacity than gypsum panels.

3.2.5 Ductility

Ductility values alone do not provide much insight into the performance of the walls. Ductility is a function of the elastic stiffness, yield displacement, and failure displacement. As discussed before, the elastic stiffness can vary with the amount of initial load, which affects the yield point and in turn the ductility. The ductility factor used for this study is defined as the failure displacement divided by the yield displacement. The ductility values are listed in Table 3.5.

Table 3.5: Ductility ratios of sheathing materials

Material	**Ductility, μ**	**COV (%)**
OSB	7.15	40.5
Hardboard	8.7	22.8
Fiberboard	7.3	15.5
Gypsum	10	12.4

Ductility factors varied from 10.9 to 5.1. Gypsum wallboard was the most ductile sheathing material with a ductility factor of 10. However, gypsum did not resist much load, so the yield load and displacement were small. Since the gypsum was able to maintain its minimal load through a large displacement, the ductility ratio was large.

Hardboard-sheathed walls exhibited a ductility factor of 8.7, while OSB-sheathed walls had a ductility factor of 7.2. The elastic stiffness of hardboard walls was greater than that of OSB walls, so the yield displacement was smaller than the OSB walls. Even though the failure displacement of OSB was 25mm (1 in.) larger than fiberboard, the ductility factor of fiberboard was larger since the yield displacement was significantly smaller.

It should be noted that walls in which failure was inhibited by the steel guide piece were not included (Tests 1 and 2), because these tests gave unrealistic failure

displacements. Although the calculated ductility ratios depend on several parameters, it can still be used to generalize the ductility of the sheathing materials.

3.2.6 Work to Failure

The amount of work a structure can supply is very important when considering a material for use as a shear wall. The lateral forces that are exerted on a structure and transferred to the shear wall produce large amounts of energy that must be absorbed in order to avoid failure. The energy dissipated during monotonic tests is calculated by determining the area under the load-displacement graph. 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 $0.8F_{peak}$.

The amount of work to failure and its corresponding coefficient of variation are given in Table 3.6. OSB-sheathed walls were able to dissipate the most energy of the sheathing materials, followed by hardboard, fiberboard, and gypsum, respectively. The energy dissipated by the OSB and gypsum sheathed walls yielded consistent results, while the response of the hardboard and fiberboard tests were erratic after reaching peak load, and the amount of energy dissipated was inconsistent. On average, the OSB dissipated 1.31 kN-m (0.97 kip-ft) of energy with a coefficient of variation equal to 9.5%. Gypsum sheathed walls dissipated 0.41 kN-m (0.3 kip-ft) of energy with a COV of 3.8%.

Table 3.6: Energy dissipation of sheathing materials

Material	Energy Dissipated (kN-m)	Energy Dissipated (kip-ft)	COV
OSB	1.31	0.97	9.5 %
Hardboard	0.95	0.7	38.2 %
Fiberboard	0.66	0.49	32.2 %
Gypsum	0.4	0.3	3.8 %

From hardboard tests (Tests 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, Test 3a was conducted using hardboard and the energy dissipated was calculated to be 1.14 kN-m (0.84 kip-ft). Combining the three tests together, the coefficient of variation was 38.2%. Wall 4 behaved as the other two walls

until reaching peak load. After peak load, Wall 4 drastically reduced in strength, which resulted in minimal energy dissipation. Wall 4 failed at 74mm (2.9 in.), while Walls 3 and 3a failed at 140mm (5.5 in.). The cause of Wall 4 to fail suddenly is unknown, but one possibility is weak framing members, which enabled the sheathing nails to pull out of the framing much easier than in the other two walls.

A similar occurrence was observed for the fiberboard sheathed walls (Tests 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, which equates to a COV of 32.2%. Although both walls nearly reached the same maximum load and elastic stiffness, Wall 6 reached maximum load at 66.3mm (2.61 in.), while Wall 5 reached maximum load at 107mm (4.2 in). Wall 6 failed at 94mm (3.7 in.), while Wall 5 failed at 140mm (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 may be an issue with the variability of fiberboard sheathing panels or the framing members. As shown in Figure 3.4a, Walls 5 and 6 perform similarly at the beginning, but Wall 6 fails much more rapidly, which reduces its ability to dissipate energy. The similarity of gypsum wallboard is shown in Figure 3.4b, which peaks at the same displacement and follows the same curve until failure. This could indicate that gypsum is a more uniform product.

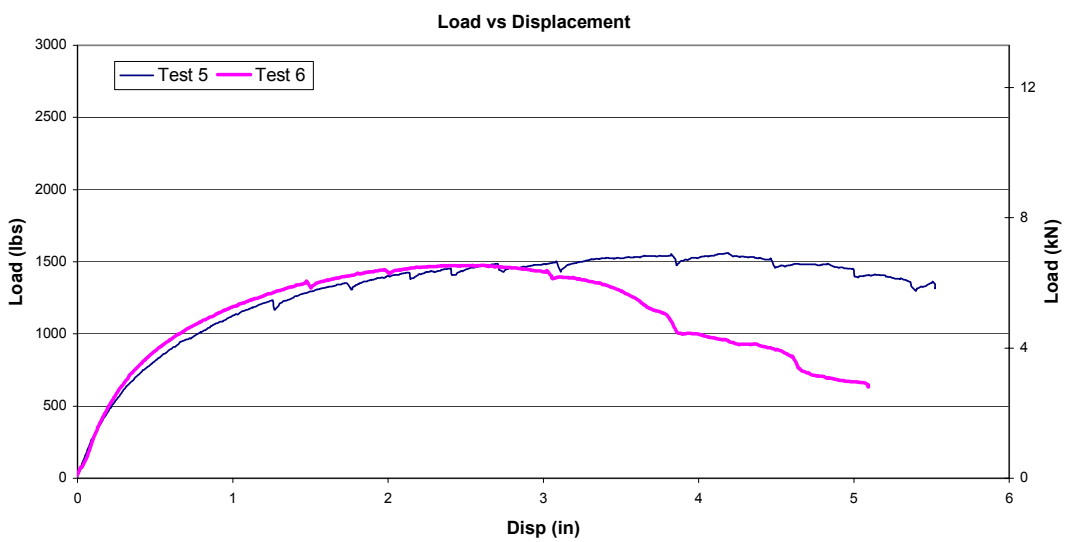

Figure 3.4a: Fiberboard work to failure

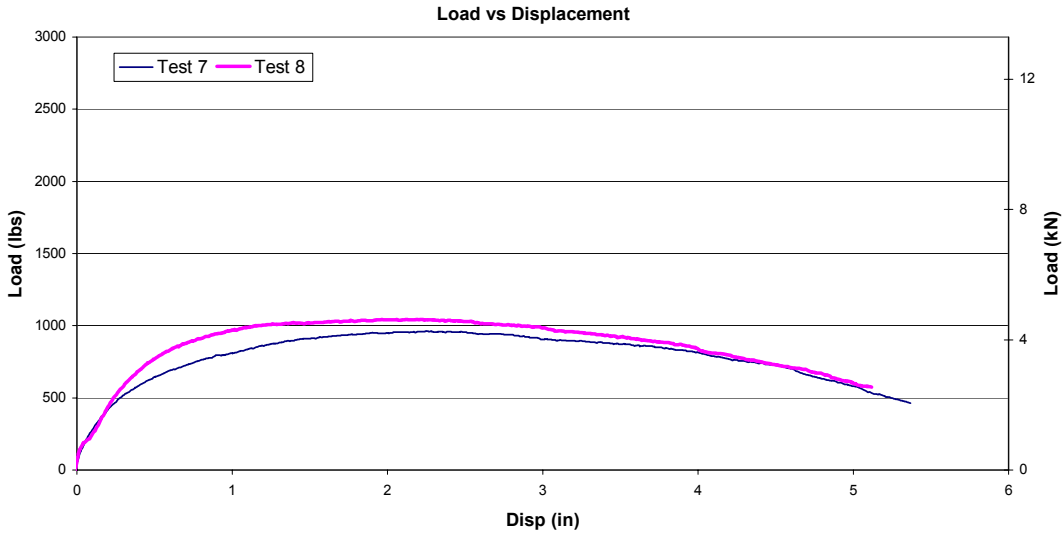

Figure 3.4b: Gypsum work to failure

3.2.7 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). The 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. Because different nails and nail spacing were used for the sheathing panels, interaction of the nails with the sheathing and framing produced different wall behavior and modes of failure.

3.2.7.1 OSB Walls

All of the OSB-sheathed walls were connected to the framing with ϕ3.33mm x 63.5mm long, 8d common nails that were pneumatically driven. Perimeter spacing was 152mm (6in.) and field spacing was 305mm (12in.). Although all of the monotonic tests used overturning restraints in the form of hold-downs, a majority of the damage to the shear wall was observed on the bottom plate as pictured in Figure 3.5. As shown in Figure 3.6, some sheathing nails pulled out of the framing, while other nails tore through the sheathing. As the sheathing nails became separated, the right end stud pulled away from the top plate as seen in Figure 3.7. Also, after the sheathing nails came out of the

bottom plate, the same occurred on the end studs starting at the bottom and working towards the top. The general mode of failure for OSB walls was the sheathing nails pulling out of the framing or tearing through the sheathing along the bottom plate, allowing the end stud to separate from the top plate.

Figure 3.5: Unzipping of nails on bottom plate of OSB

Figure 3.6: Nail pullout and tear-through on bottom plate of OSB

Figure 3.7: End stud separation at top of wall for OSB wall

3.2.7.2 Hardboard Walls

All hardboard-sheathed walls were connected to the framing with ϕ2.5mm x 51mm long x ϕ6.8mm head, 6d box nails. Perimeter spacing was 102mm (4in.) and field spacing was 203mm (8in.). Whereas OSB nails either pulled out of the framing or tore through the sheathing, the sheathing nails used for hardboard always pulled out of the framing. The nails never damaged the hardboard sheathing as can be seen in the pictures. This could be contributed either to the smaller diameter and shorter nail used for hardboard panels when compared to the OSB panels, or to the fact that hardboard sheathing is denser than OSB, as a result preventing a tear-out failure.

As shown in Figures 3.8 and 3.9, the nails along the bottom plate were observed to pull out first. After the sheathing nails pulled out of the bottom plate, the same occurred on the end studs starting at the bottom and working towards the top. When the sheathing nails pulled out of the framing, the right end stud pulled away from the top plate, as pictured in Figure 3.10. The general mode of failure for hardboard walls was the sheathing nails pulling out of the framing on the bottom plate, allowing the end stud near the actuator to separate from the top plate.

Figure 3.8: Nail pullout along bottom plate of hardboard sheathing

Figure 3.9: Nail pullout in hardboard panels

Figure 3.10: End stud separation at top of wall for hardboard sheathing

3.2.7.3 Fiberboard Walls

All of the fiberboard-sheathed walls were connected to the framing with ϕ3mm x 38mm long x ϕ9.5mm head, galvanized roofing nails. Perimeter spacing was 102mm (4in.), while the field spacing was 152mm (6in.). Fiberboard is a much weaker material than OSB and hardboard, so the sheathing nails could easily tear through the sheathing as shown in Figure 3.11. However, due to the large head diameter of the nail, the nails were also observed to pull out of the framing in some cases as demonstrated in Figure 3.12. Fiberboard nails pulled out of the framing or tore through the sheathing around the entire perimeter of the wall. Some of the sheathing nails completely pulled out of the frame and lay on the ground after the test. There was some minor separation of the end stud from the top plate, but not as drastic as observed in OSB and hardboard walls. The general mode of failure for fiberboard walls was the sheathing nails completely pulling out of the framing or tearing through the sheathing along the perimeter of the wall.

Figure 3.11: Nail tear-through of fiberboard sheathing

Figure 3.12: Sheathing-to-framing separation of fiberboard sheathing

3.2.7.4 Gypsum walls

All gypsum-sheathed walls were connected to the framing with ϕ3mm x 38mm long x ϕ9.5mm head, galvanized roofing nails. Perimeter spacing was 178mm (7in.), while the field spacing was 406mm (16in.). As expected, gypsum was the weakest sheathing material allowing the nail head to easily penetrate the outer layer and sink through the panel as shown in Figures 3.13 and 3.14. The nail tear-through started along the bottom plate and continued around the perimeter of the wall. Furthermore, the sheathing completely fell off of the frame during the test. The general mode of failure for gypsum walls was the sheathing nail heads sinking and tearing through the sheathing along the perimeter of the wall.

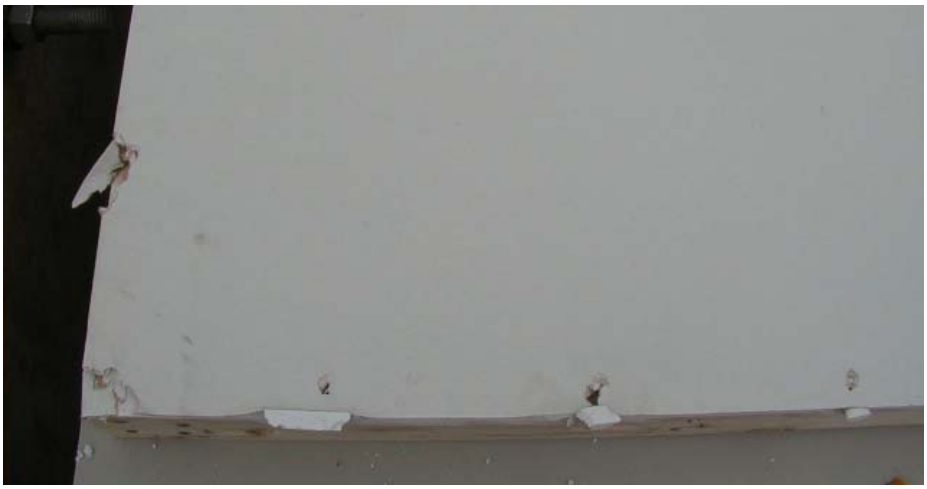

Figure 3.13: Nail tear-through in the gypsum panels

Figure 3.14: Nail tear-through on the top plate of gypsum sheathing

3.3 Contribution of gypsum wallboard

The contribution of gypsum wallboard subjected to monotonic loading is investigated in this section. Two walls were tested with only one panel of gypsum, and two walls of each of the other sheathing materials were tested with OSB, hardboard, and fiberboard on the exterior of the wall and gypsum sheathing on the interior of the wall. The purpose was to determine how the racking resistance of walls using dissimilar sheathing materials should be calculated.

3.3.1 Ultimate Load

The ultimate load of walls with dissimilar sheathing materials on opposite sides was compared to walls sheathed on only one side to determine the contribution of gypsum to the maximum strength of the structure. The contribution of gypsum when used in combination with OSB was 2.36 kN (0.53 kips). When sheathed with hardboard and fiberboard, the contribution of gypsum was 3.74 kN (0.84 kips) and 3.5 kN (0.79 kips), respectively. When the contribution of gypsum is averaged for the three sheathing materials, the resulting strength was 3.2 kN (0.72 kips). The ultimate load of the walls tested only with gypsum was 4.45 kN (1.0 kips). Although small, the shear resistance of the framing material is included in the 4.45 kN (1.0 kips) value for gypsum. Therefore, it appears that for monotonic tests the strength of gypsum contributes, but is not additive to the ultimate strength of the wall providing 3.2 kN (0.72 kips) in addition to the exterior

sheathing panel. When used in combination with OSB, the contribution of gypsum was much smaller than when used in combination with hardboard and fiberboard. The reason for this is unknown, but more tests would have to be conducted to fully understand this interaction.

Table 3.7: Contribution of gypsum to peak load

Material	Average Peak Load (kN)	Average Peak Load (kips)	**Contribution of Gypsum**	
OSB	11.16	2.51	**0.53 (kip)**	**0.13 (kip/ft)**
OSB/GWB	13.52	3.04	**2.36 (kN)**	**1.94 (kN/m)**
Hardboard	9.26	2.08	**0.84 (kip)**	**0.21 (kip/ft)**
Hardboard/GWB	13.0	2.92	**3.74 (kN)**	**3.07 (kN/m)**
Fiberboard	6.75	1.52	**0.79 (kip)**	**0.2 (kip/ft)**
Fiberboard/GWB	10.3	2.31	**3.5 (kN)**	**2.87 (kN/m)**
Only GWB	**4.45**	**1.0**	**1.0 (kip)**	**0.25 (kip/ft)**
			4.45 (kN)	**3.65 (kN/m)**

3.3.2 Yield Load and Displacement

The yield load was determined in the same manner as described in Section 2.8.2. Yield load is function of the elastic stiffness, failure displacement, and energy dissipation. Elastic stiffness is the slope of the secant line that passes through the origin and the point on the load-displacement curve that is equal to $0.4F_{peak}$. Similar to the monotonic tests sheathed on only one side, there was a wide variation in the elastic stiffness observed in the tests. This was due in part to the initial load that was put on the structure. An initial load was sometimes applied to a wall when it was attached to the test frame and the boltholes did not match perfectly. This was actual load on the structure, so it could not be neglected. An initial load caused the elastic stiffness to increase because the wall reached a higher load than a wall without an initial load at a given displacement. However, the general trend that can be seen in Figure 3.15 is that the elastic stiffness of the walls sheathed on both sides is greater than the single-sheathed walls.

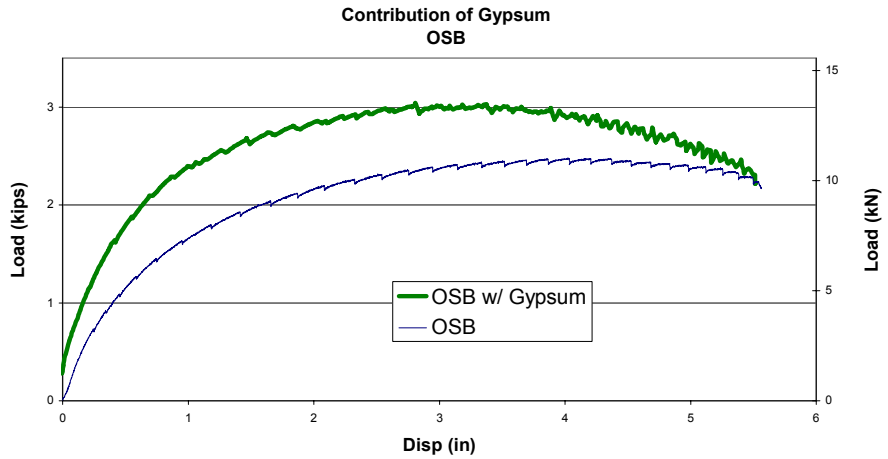

Figure 3.15a: Contribution of gypsum subjected to monotonic loading (OSB)

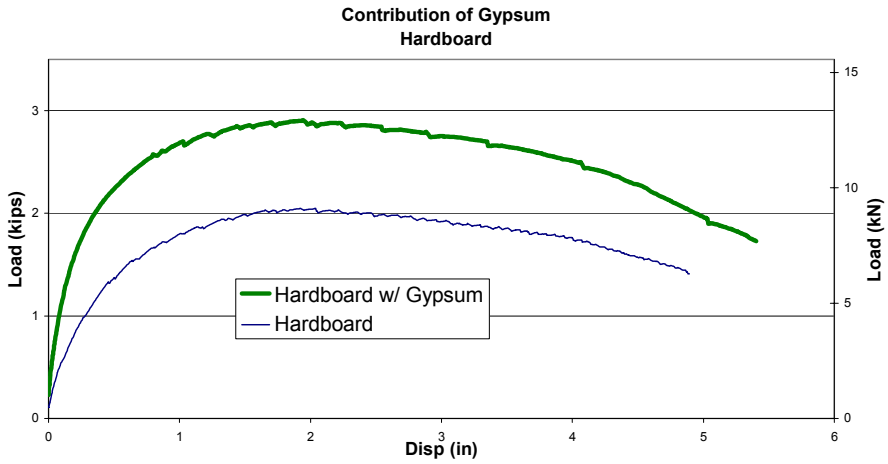

Figure 3.15b: Contribution of gypsum subjected to monotonic loading (hardboard)

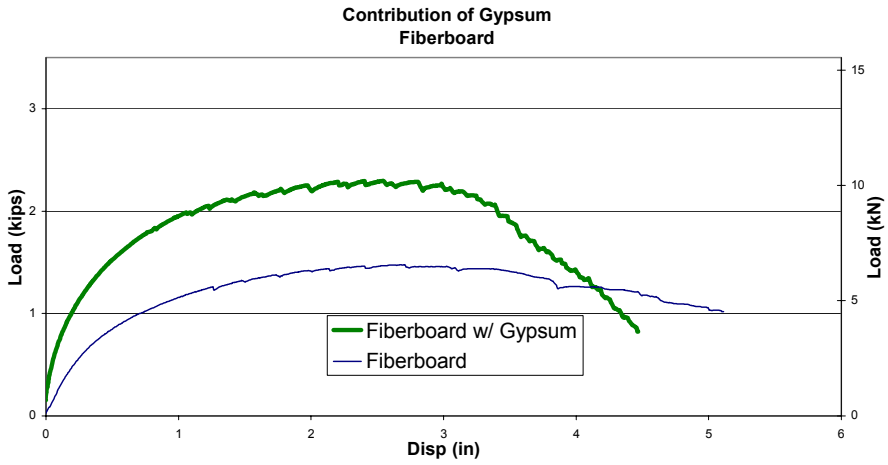

Figure 3.15c: Contribution of gypsum subjected to monotonic loading (fiberboard)

The displacement at failure when using gypsum actually decreased (an average of 13.5mm, 0.53 in.) when compared to its corresponding wall without gypsum.

Displacement capacity of the fiberboard-sheathed walls decreased by 25mm (1 in.) when gypsum was included. The more rapid reduction of strength after capacity is reached in fiberboard walls with gypsum can be seen in Figure 3.15.

The yield load of the walls with gypsum was higher than their corresponding wall without gypsum because the increased peak load directly influences the yield load. The yield load for all of the tests had a coefficient of variation less than 6%, but as expected, the COV for the yield displacement was very high.

3.3.3 Ductility

Ductility values alone do not provide much insight into the performance of the walls. Ductility is a function of elastic stiffness, yield displacement, and failure displacement. As discussed before, elastic stiffness can vary with the amount of initial load, which affects the yield point and in turn ductility.

The ductility of gypsum by itself was large when compared to the other sheathing materials. When included with another sheathing material, the ductility factor increased by a substantial amount. This can be attributed to the increase in elastic stiffness, which decreased the yield displacement. When gypsum was included, the failure displacement actually decreased on average by 13mm (0.5 in.). The COV is very large, which shows the results are not consistent. Although more tests would be needed to accurately measure the ductility, the addition of gypsum tends to increase the ductility of the system as shown in Table 3.8.

Table 3.8: Contribution of gypsum to ductility

Material	**Ductility, μ**	**COV (%)**	**% Increase**
OSB OSB/GWB	7.2 10.3	41 37	43%
Hardboard Hardboard/GWB	8.7 18.5	23 25	112%
Fiberboard Fiberboard/GWB	7.3 11.0	16 42	51%
Gypsum	10.0	12	0%

3.3.4 Work to Failure

Work to failure values for walls that were sheathed with dissimilar materials on opposite sides of the wall is given in Table 5.8. There was a small increase in the work to failure when gypsum was included in the specimens. Single-sheathed gypsum walls dissipated 0.4 kN-m (0.3 k-ft) of energy, while gypsum sheathing only contributed an average of 0.21 kN-m (0.15 k-ft) of energy when combined with a structural sheathing material. Although only half of the energy of gypsum was dissipated when used with a structural sheathing material, it can still be concluded that gypsum contributes to the energy dissipating capacity.

Table 3.9: Contribution of gypsum to energy dissipation

Material	Energy Dissipated (kN-m)	Energy Dissipated (kip-ft)	**Contribution Of Gypsum**
OSB OSB/GWB	1.31 1.49	0.97 1.1	**0.13 (k-ft)** **0.18 (kN-m)**
Hardboard Hardboard/GWB	0.95 1.25	0.7 0.92	**0.22 (k-ft)** **0.3 (kN-m)**
Fiberboard Fiberboard/GWB	0.66 0.81	0.49 0.6	**0.11 (k-ft)** **0.15 (kN-m)**
Gypsum	**0.4**	**0.3**	

3.3.5 Sheathing Nail Behavior and Mode of Failure

For the walls sheathed on both sides, the general observations from the tests were similar to when the walls were tested with only one sheathing material (Refer to section 3.2.7 to understand the general behavior of the sheathing panels). Gypsum panels were always the first to fail due to the ease of which the nails could tear through the sheathing. This is why the gypsum increased the elastic stiffness, but the failure displacement was about the same as the exterior sheathing panel. The racking displacement observed during monotonic tests is demonstrated in Figure 3.16. The interaction of fiberboard and gypsum panels is shown in Figures 3.17 and 3.18. Nails in the fiberboard panels pulled

out of the framing, while the gypsum nail heads sink and tear through the sheathing panel.

Figure 3.16: Racking displacement of sheathing panels

Figure 3.17: Nail pullout and tear-through of fiberboard and gypsum

Figure 3.18: Fiberboard and gypsum interaction

3.4 Summary

Seventeen walls were tested under monotonic loading using ASTM E564. All walls used hold-downs connectors as the overturning restraint. In summary, the effects of using various sheathing materials on light-framed shear walls subjected to monotonic loading are as follows:

(1) OSB was the strongest material based on ultimate strength (11.16 kN, 2.51 kips). Hardboard had an average strength of 9.26 kN (2.08 kips), while fiberboard's strength was 6.75 kN (1.52 kips). As expected, gypsum was the weakest material with an average maximum strength of 4.45 kN (1.00 kips).

(2) Average values calculated at the critical loads were in good agreement with each other, but the corresponding displacements had large variations. Initial loads applied to the structure are one reason for the variation, but another reason may be the nature of the materials. More tests should be performed to better estimate the performance of these walls.

(3) Some walls had an obvious failure pattern, while some walls reached their peak load and then maintained a load above $0.8F_{peak}$ through a substantial displacement. It could not be determined from two tests why this occurred.

(4) The contribution of gypsum wallboard to monotonic tests was shown to increase the overall strength, elastic stiffness, and work to failure of the structure. The average additional strength provided when gypsum was included was 3.2 kN (0.72 kips). The strength of a wall sheathed only with gypsum was 4.45 kN (1.00 kips). Therefore, gypsum should be considered to supply a substantial amount of shear resistance when subjected to monotonic loading but is not linearly additive.

Chapter 4: Cyclic Test Results

4.1 General

A total of 28 cyclic tests were conducted for this study. All walls were tested using the new ASTM cyclic standard E2126 as described in Chapter 2. USP HTT22 hold-down connectors were used for half of the tests, while the other half were tested without hold-down connectors. In addition to the use of hold-downs, the four sheathing materials are compared, and the contribution of gypsum wallboard is discussed in this chapter. The relationship between the initial and stabilized envelope curves is also of interest to understand the performance of shear walls. The data analysis involves performance indicators such as capacity, yield strength, failure capacity, hysteretic energy, elastic and cyclic stiffness, damping, and ductility. A list of the test configurations and test numbers conducted for the cyclic tests are given in Table 4.1.

Table 4.1: Number of cyclic tests with test numbering

	Test Number	
Sheathing Material	**Hold-down**	**No Hold-down**
OSB	10 10a	17 18
Hardboard	11 12	19 20
Fiberboard	13 14a	21 22
Gypsum	15 16	23 24
OSB/GWB	31 32	37 38
Hardboard/GWB	33 34	39 40
Fiberboard/GWB	35 36	41 42

All of the walls were 1.2 x 2.4m (4 x 8ft). This size was chosen because it is the minimum length allowed for wood structural panels to resist earthquake loads without further reduction in design values. Salenikovich (2000) showed that all walls 1.2m (4ft) and longer developed the same strength, on a unit length basis, provided the wood density of the studs was equal.

The 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-downs are given in Appendix B, while the individual specimen results of the cyclic tests without hold-downs

are given in Appendix C. In Appendix B and C, a description of the test observations is given for each test, along with the mode of failure. A listing of shear wall parameters is given in tabular form. Graphs of the load-displacement curves are also provided in Appendix B and C.

According to ASTM E2126 (Method A), it is possible to have phases where the initial amplitude does not increase. If the ductility factor is less than 20, then the 5th phase in the loading protocol will be less than the previous phase. Regardless, the loading protocol was followed exactly as described in the standard. However, when the envelope curves and other graphs are displayed in this chapter, the phases with decreasing amplitude are not included. This was done because incorporating decreasing displacement phases does not appear to provide beneficial information into the performance of the wall.

4.2 Comparison of Sheathing Materials with Hold-downs

4.2.1 Load-Displacement Relationship

A typical load-displacement curve is shown in Figure 4.1 for a cyclic test with hold-downs. The average load-displacement graph for each material is shown in Figure 4.2. The cyclic tests were fully reversed, so the absolute values of the negative and positive cycles were averaged to determine the initial envelope curve. The initial envelope curve for the two data sets for each sheathing material was then averaged to determine the overall load-displacement values. From Figure 4.2, it is obvious that the performance of the wall depends on the sheathing material. Each sheathing material reaches its maximum strength at a different load and fails at a different load and displacement. Therefore, it is important to look at several different parameters when analyzing the various sheathing materials.

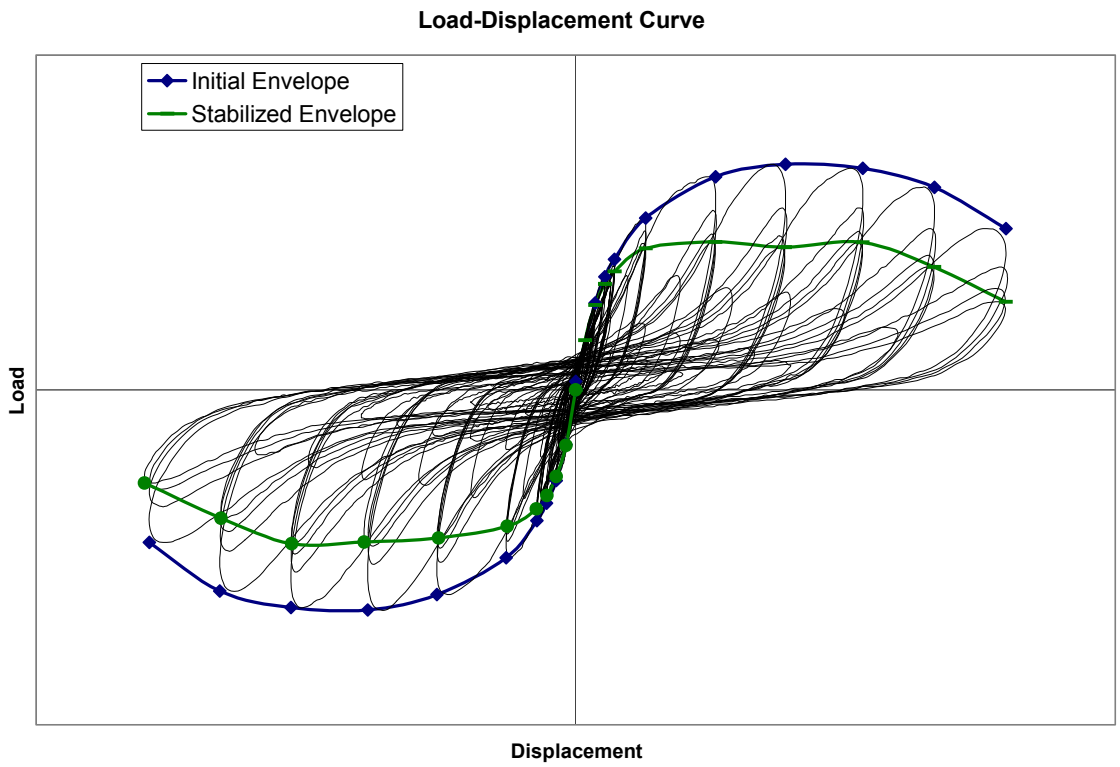

Figure 4.1: Typical load-displacement curve subjected to cyclic loading

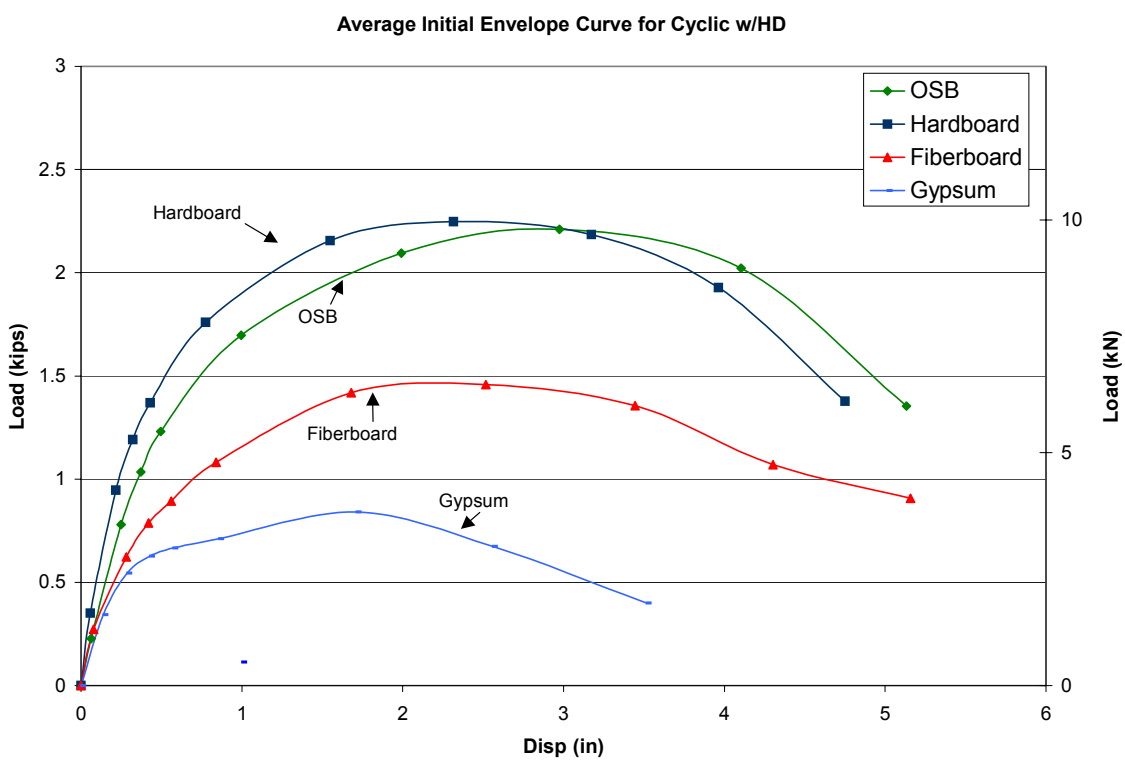

Figure 4.2: Average initial envelope curve

Hardboard sheathing panels were able to resist the largest load of the four sheathing materials. Its strength was 10.0 kN (2.25 kips), while OSB panels had an average maximum strength of 9.83 kN (2.21 kips). Fiberboard's strength was 6.48 kN (1.46 kips), and gypsum could only resist 3.74 kN (0.84 kips). The elastic stiffness of the initial envelope curves followed the same pattern. Hardboard had an elastic stiffness of 0.92 kN/mm (5.27 kip/in). OSB had an elastic stiffness of 0.66 kN/mm (3.76 kip/in). Fiberboard and gypsum had an elastic stiffness of 0.51 kN/mm (2.91 kip/in) and 0.45 kN/mm (2.56 kip/in), respectively.

In addition to the sheathing material, the nail size and perimeter spacing influenced the performance of the wall. The peak load of OSB and hardboard was similar, but the nails and nail spacing were not. The OSB panels were fastened using ϕ3.33 x 63.5mm long, 8d nails with 152mm (6 in.) perimeter spacing, while hardboard panels were fastened using ϕ2.5 x 51mm long x 6.8mm head, 6d nails with 102mm (4 in.) perimeter spacing.

As described in Chapter One, stabilization cycles were implemented to allow the strength and stiffness reduction to stabilize before increasing the displacement of the next cycle. Stabilized envelope curves were formed using the same procedure as the initial envelope curve. The average stabilized envelope curves for the four sheathing materials are shown in Figure 4.3. Peak load values, yield load values, and the ratio of initial and stabilized curves are displayed in Table 4.2. The peak load and yield load of the stabilized response was typically 60-70% of the initial response. This means that the maximum strength degradation was 30-40% when subjected to continuous cycles at the same displacement. The elastic stiffness of the two curves was similar because the load and displacement at 40% of maximum load was the same. At this point, the wall had not yielded, so there was very little strength degradation during the stabilized cycles at low load levels.

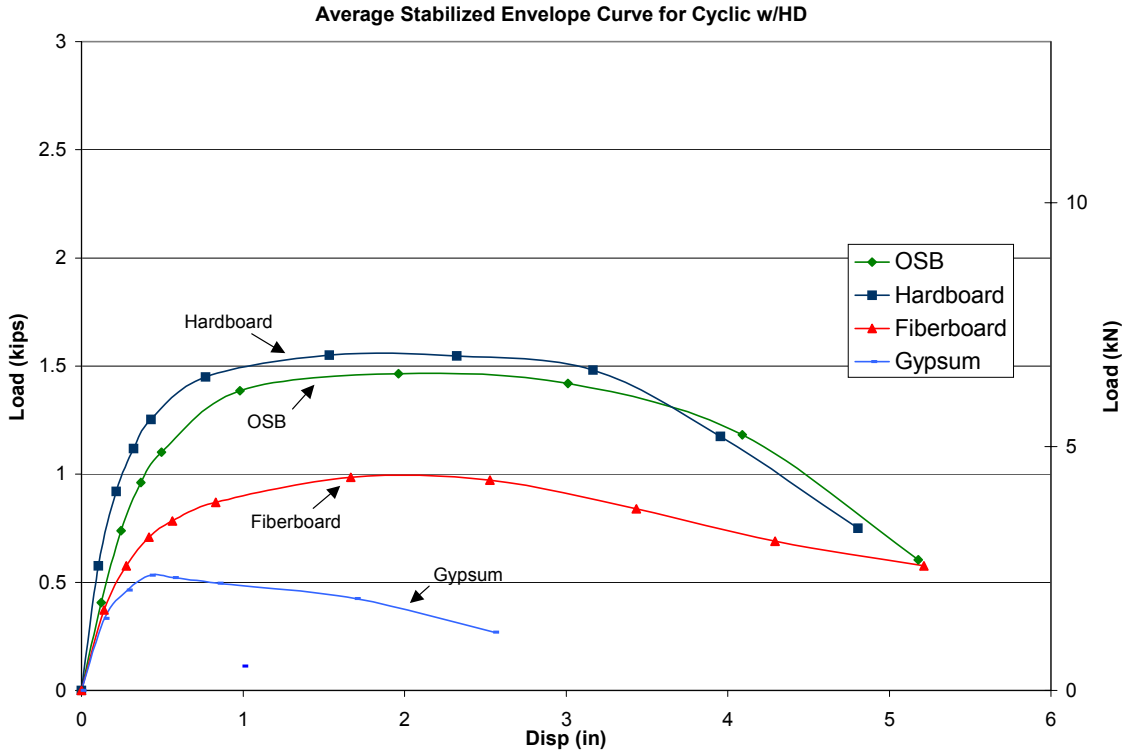

Figure 4.3: Average stabilized envelope curve

Table 4.2: Comparison of the initial to stabilized values

	SHEATHING MATERIAL							
	OSB		Hardboard		Fiberboard		Gypsum	
	(kN)	(kips)	(kN)	(kips)	(kN)	(kips)	(kN)	(kips)
Capacity:								
Initial cycle	9.83	2.21	10.0	2.25	6.48	1.48	3.74	0.84
Stabilized cycle	6.58	1.48	7.06	1.59	4.48	1.01	2.39	0.54
Stabilized/Initial	67%		71%		69%		64%	
F_{yield}:								
Initial cycle	8.72	1.96	8.95	2.01	5.68	1.28	3.07	0.69
Stabilized cycle	6.00	1.35	6.39	1.44	3.97	0.89	1.75	0.39
Stabilized/Initial	69%		72%		70%		57%	

4.2.2 Failure Capacity and Displacement

As defined in Chapter Two, the failure load was taken to be $0.8F_{peak}$. Since this is obviously based on the maximum load, the walls with the highest strength will also have the largest load capacity at failure. Displacement at failure is an important parameter to investigate and is not a function of the ultimate load. The ability of a structure to dissipate more energy results from it being able to deform without failing.

As shown in Table 4.3, OSB and hardboard walls were able to withstand the largest force at failure when subjected to cyclic loading. The failure displacement of hardboard walls was not significantly smaller (11%) than OSB walls at failure. Although fiberboard was weaker than OSB and hardboard, fiberboard failed at a similar displacement as the other two sheathing materials. Gypsum was the weakest material and failed at a displacement of 41mm (1.62 in.) less than any other sheathing material.

Table 4.3: Failure load and displacement of cyclic tests

Material	Failure Load		Displacement at Failure	
	(kN)	(kips)	(mm)	(in.)
OSB	7.87	1.77	116	4.58
Hardboard	8.01	1.80	106	4.17
Fiberboard	5.20	1.17	109	4.29
Gypsum	2.98	0.67	65	2.55

The failure mode of the walls typically involved the sheathing nails either pulling out of the framing or tearing through the sheathing. Although the nails in the hardboard panels were spaced at four inches on center, the material failed at a displacement 25mm (1 in.) less than OSB walls. When hardboard-sheathed walls were tested, the nails were observed to pull out of the framing with relative ease. Although the pullout started along the bottom plate, the nails also pulled out along the end studs during the latter part of the testing. When the pullout became significant, the sheathing panel could no longer resist the shear forces, and the wall failed. OSB-sheathed walls were observed to experience pullout and tear-through. However, this did not occur until a larger displacement, which enabled the wall to resist a higher load through a larger displacement.

4.2.3 Ductility

Using the load-displacement graph, the ductility factor was taken directly from the EEEP curve shown in Figure 4.4. The EEEP curve, defined in Chapter Two, was formed using the average initial envelope curve. The ductility factor used for this study is defined by the displacement at failure divided by the yield displacement. The ductility factor of the four sheathing materials varied from 7.6 to 10.4 as listed in Table 4.4. Hardboard panels were the most ductile. Fiberboard and gypsum had nearly the same

ductility factor of 9.5. OSB was the least ductile with a ductility factor of 7.7. As shown in Table 4.4, the coefficient of variation was large, but because only two tests were performed for each sheathing material the variability cannot be investigated. However, the ductility factor does give a good indication of the general behavior of the material.

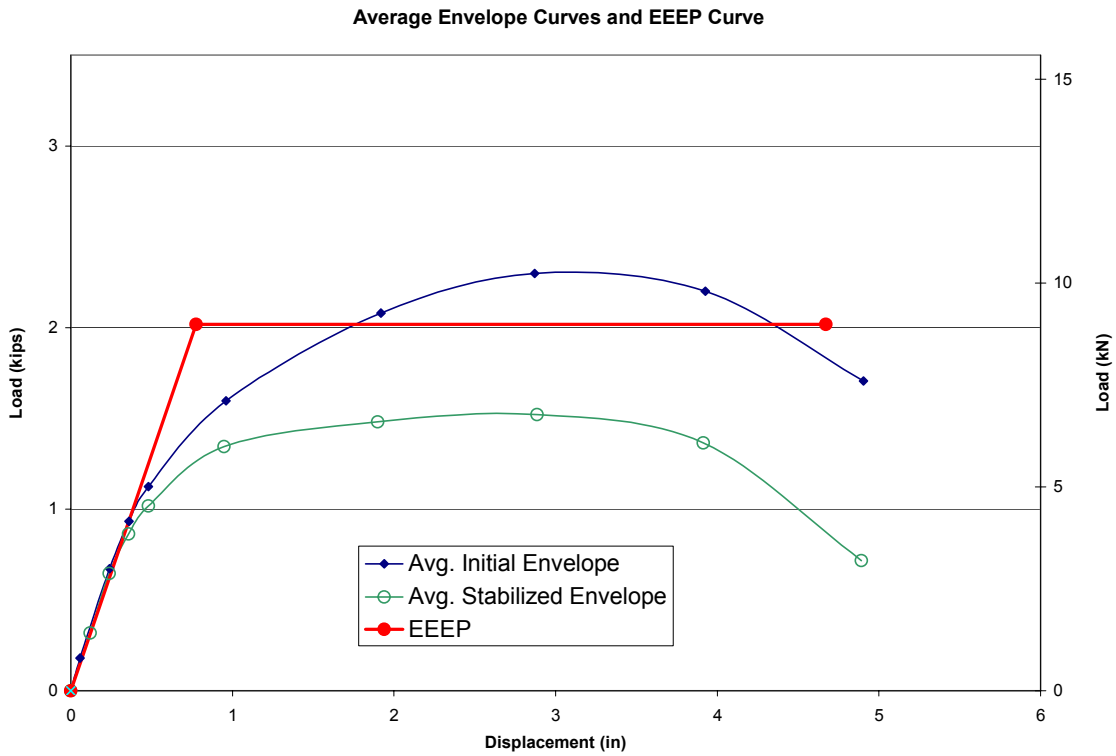

Figure 4.4: Typical EEEP curve for cyclic test with hold-downs

Table 4.4: Ductility factors for cyclic tests

Material	Ductility, μ	COV (%)
OSB	7.66	30.0
Hardboard	10.41	2.7
Fiberboard	9.42	26.7
Gypsum	8.94	14.9

4.2.4 Hysteretic Energy

An important performance indicator of a shear wall is its ability to dissipate energy when subjected to ground motions produced by earthquakes. 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. When tested using the sequential phased displacement procedure, the critical cycles are the initial cycle and final stabilized cycle. The plot in Figure 4.5 displays the amount of hysteretic energy of each sheathing material during the initial and stabilized cycle of each phase.

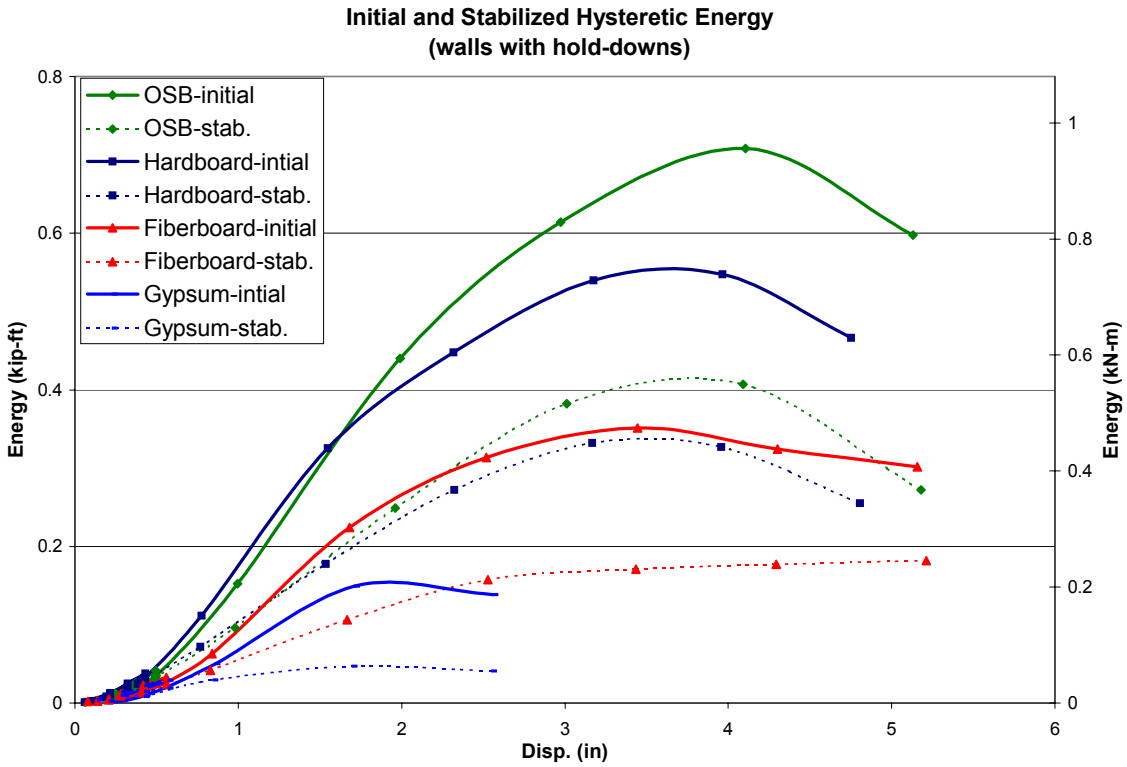

Figure 4.5: Initial and stabilized hysteretic energy versus displacement

Plotting hysteretic energy versus displacement at a given cycle enables one to compare the energy dissipating capacity of the sheathing materials. As expected, OSB and hardboard panels dissipated the largest amount of energy. Both materials behaved the same until 38mm (1.5 in.) displacement. At this displacement, the hysteretic energy of hardboard walls leveled off, but OSB walls dissipated much more energy. When considering peak load, hardboard panels were slightly stronger. However, as shown in Figure 4.5, OSB panels were much tougher, meaning that they absorbed more energy at a given displacement. Toughness of a material is crucial when considering the ground motions produced by earthquakes.

The reduction in energy dissipated observed in the hardboard panels can be attributed to nail behavior. Nails in the hardboard panels easily pulled out of the framing.

When this occurred, low lateral stiffness of the nails enabled them to bend, which reduced the friction between the framing and the sheathing. Nails in the OSB panels did not pull out as easily, which enabled the friction to continue to dissipate more energy at a larger displacement than fiberboard.

The toughness of fiberboard panels was significantly smaller than OSB and hardboard panels but was much larger than gypsum panels. The nails used to connect fiberboard and gypsum walls easily tore through the sheathing, which restricted the amount of energy that could be dissipated. The nailing schedule of fiberboard panels was much denser than gypsum panels, which has a direct influence on the energy dissipating capacity.

There was also a large reduction in hysteretic energy between the initial and stabilized cycles. 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.

4.2.5 Equivalent Viscous Damping

The equivalent viscous damping ratio, EVDR, is only an approximation because the walls were 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 timber structures to structures with other materials when considering the equivalent viscous damping beyond the elastic limit (Lowe and Edwards 1984; Polensek 1988; Foliente and Zacher 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. As shown in Figures 4.6 and 4.7, the EVDR was the smallest during elastic cycles. During the initial cycles, the EVDR increased until a displacement of about 44.5mm (1.75 in.). After reaching maximum load, damping progressively decreased until failure. During the stabilized cycles, the EVDR was the smallest during elastic cycles, then

increased to a maximum at 44.5mm (1.75 in.), but maintained at that level until failure. The OSB displayed the largest damping ratio of the sheathing materials.

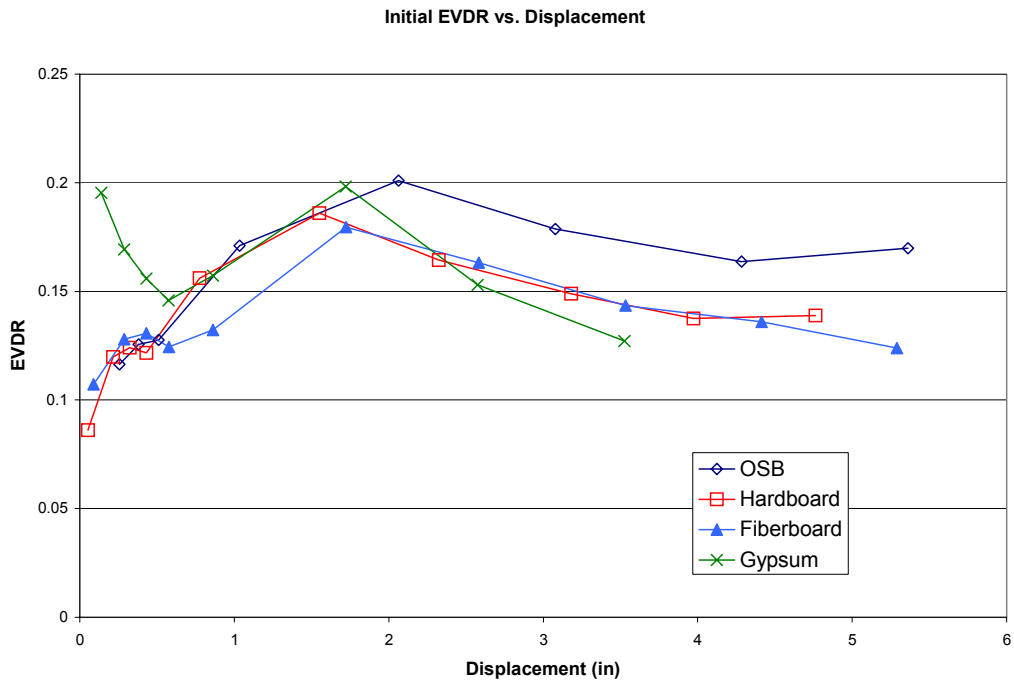

Figure 4.6: Initial EVDR of sheathing panels

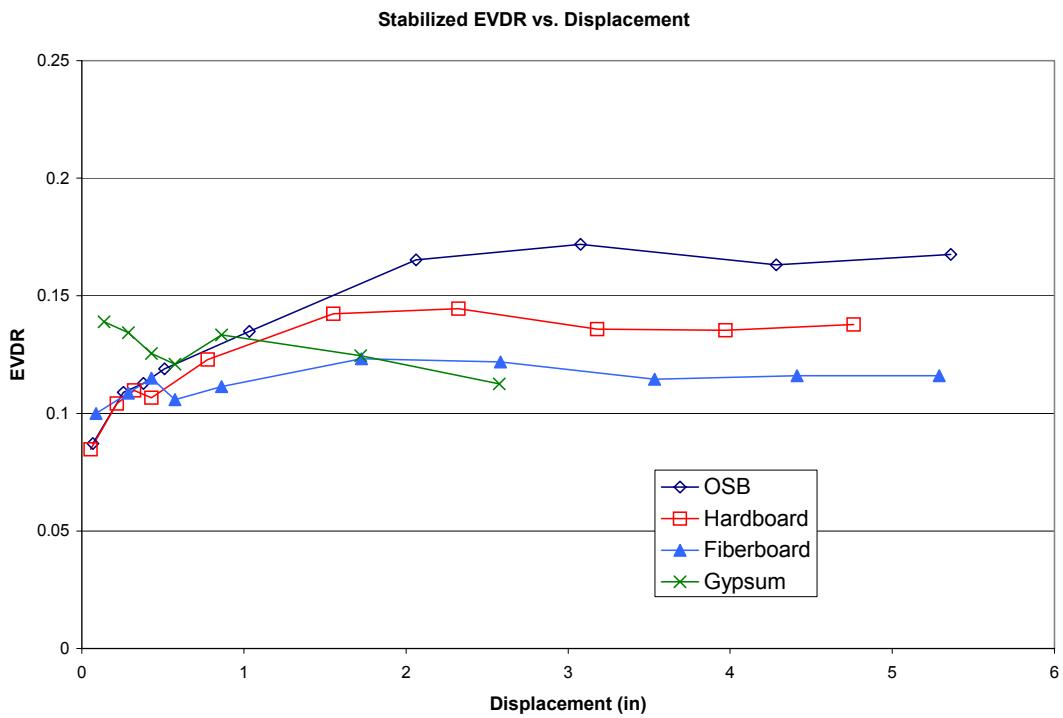

Figure 4.7: Stabilized EVDR of sheathing panels

4.2.6 Cyclic Stiffness

Cyclic stiffness can be calculated during each cycle of a test as described in Chapter Two. 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 tore a larger hole, or pulled out of 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 is shown in Figures 4.8 and 4.9, 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 between 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.

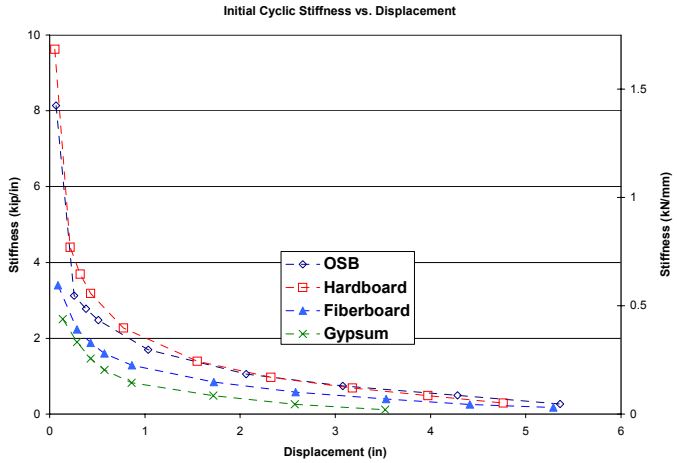

Figure 4.8: Initial cyclic stiffness

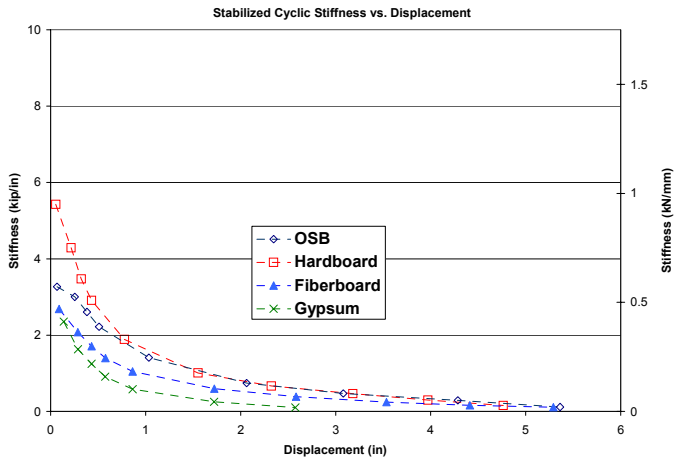

Figure 4.9: Stabilized cyclic stiffness

4.2.7 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. Due to the nature of the cyclic tests, the walls in this study typically failed when the sheathing nails either pulled out of the framing or tore through the sheathing from the continuous fully reversed cycles. When this happened, the sheathing panels separated from the framing and the wall was unable to resist any additional shear forces. The use of overturning restraints in the form of hold-downs enabled the applied load to be transferred around the perimeter of the wall as opposed to strictly the bottom plate as seen in walls without hold-downs. Since

different nails and nail spacing were used for the sheathing panels, the interaction of nails with the sheathing and framing produced different wall behavior and modes of failure.

4.2.7.1 OSB Walls

All OSB-sheathed walls were connected to the framing with ϕ3.33mm x 63.5mm long, 8d common nails that were pneumatically driven. The perimeter spacing was 152mm (6in.), and the field spacing was 305mm (12in.). The nails typically pulled out of the framing or tore through the sheathing on the bottom plate first as seen in Figure 4.10. During Test 10, the nails also began to pull out of the framing along the top plate and left end stud. From Figure 4.11, it can be seen that nails that pulled out of the framing were bent in an S-shape. Nail pullout allowed the end stud to separate from the top plate, which enabled the wall to fail.

Figure 4.10: Sheathing nails unzipping along bottom plate of OSB

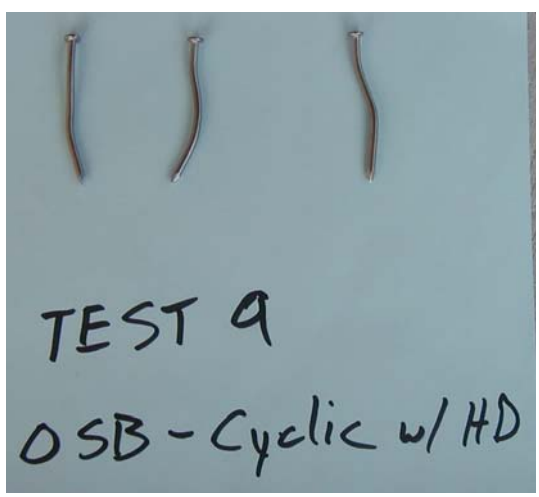

Figure 4.11: OSB nail shape after cyclic test

4.2.7.2 Hardboard Walls

All hardboard-sheathed walls were connected to the framing with ϕ2.5mm x 51mm long x 6.8mmϕ head, 6d box nails. The perimeter spacing was 102mm (4in.), and the field spacing was 203mm (8in.). Whereas the OSB nails either pulled out of the framing or tore through the sheathing, the sheathing nails used in hardboard panels always pulled out of the framing. The nails never damaged the hardboard sheathing as can be seen in Figure 4.12. This could be contributed either to the smaller diameter of the nail in the hardboard panels or to the hardboard sheathing being denser than OSB sheathing.

The nails were first observed to pullout along the bottom plate, then along the end studs. The walls failed because the nails pulled out of the framing, which enabled the wall to deflect, then the sheathing material could not resist the shear load.

Figure 4.12: Nail pullout in hardboard panels

4.2.7.3 Fiberboard Walls

All fiberboard-sheathed walls were connected to the framing with ϕ3mm x 38mm long x ϕ9.5mm head, 11gauge Galvanized roofing nails. The perimeter spacing was 102mm (4in.), while the field spacing was 152mm (6in.). Fiberboard is a much weaker material than OSB and hardboard, so the sheathing nails could easily tear through the sheathing. The fiberboard nails were observed to tear through the sheathing along the entire perimeter of the wall as seen in Figure 4.13. The walls failed because the

sheathing separated from the framing by tear-out. Separation of framing did not occur, and there was no visible damage to the framing. Nails were usually not bent or only slightly bent. Some of the nail bending shown in Figure 4.14 may be due to damage caused while pulling the nail.

Figure 4.13: Nail tear-through in fiberboard panels

Figure 4.14: Fiberboard nails after cyclic test

4.2.7.4 Gypsum Walls

All gypsum-sheathed walls were connected to the framing with ϕ3mm x 38mm long x ϕ9.5mm head, 11gauge Galvanized roofing nails. The perimeter spacing was 178mm (7in.), while the field spacing was 406mm (16in.). As expected, the gypsum was the weakest sheathing material allowing the nails to easily penetrate the outer layer and sink through the panel as pictured in Figure 4.15. When the gypsum panels failed, they actually dropped off of the frame before the cyclic protocol was close to finishing as

shown in Figure 4.16. The average failure displacement of the gypsum wallboard was 64.8mm (2.55 in.).

Figure 4.15: Gypsum nail tear-through along end stud

Figure 4.16: Gypsum panel separation from framing

4.3 Comparison of Sheathing Materials Without Hold-downs

4.3.1 Load-Displacement Relationship

A typical load-displacement curve is shown in Figure 4.17 for a cyclic test without hold-downs. The average load-displacement envelope curve for each material is shown in Figure 4.18. The cyclic tests were fully reversed, so 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. From Figure 4.18, it is evident that the performance of the wall is dependant on the sheathing material and its corresponding nailing schedule. However, the differences observed for walls with hold-downs are not present for the walls without hold-downs. This is due to the load being concentrated in the bottom row of nails, which are required to resist both the lateral shear and overturning forces. Each sheathing material reaches its maximum strength at a different load and fails at a different load and displacement. Therefore, it is important to look at several different parameters when analyzing the various sheathing materials.

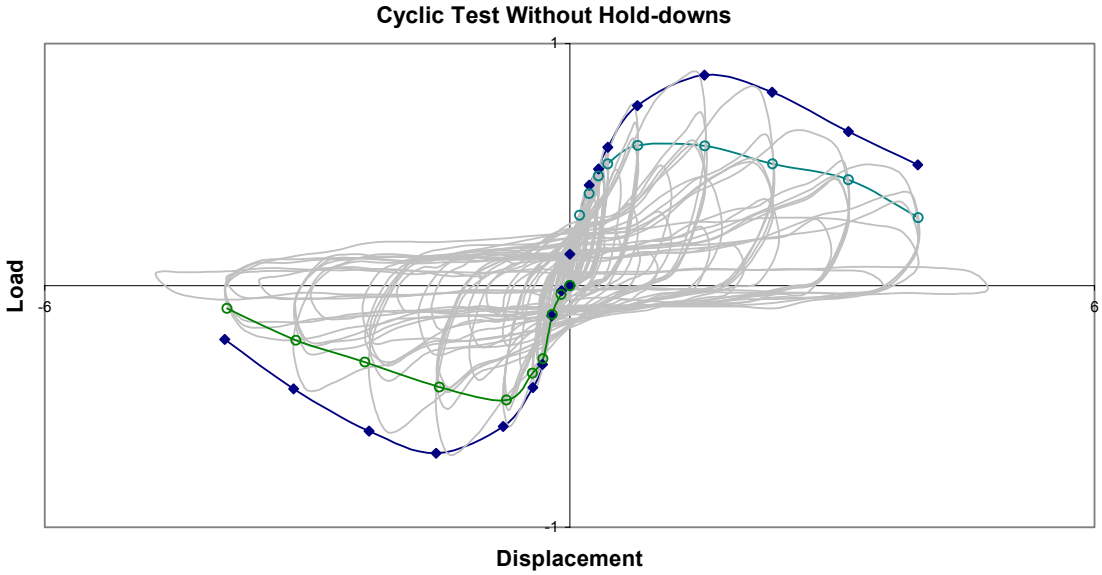

Figure 4.17: Typical load-displacement graph of cyclic tests without hold-downs

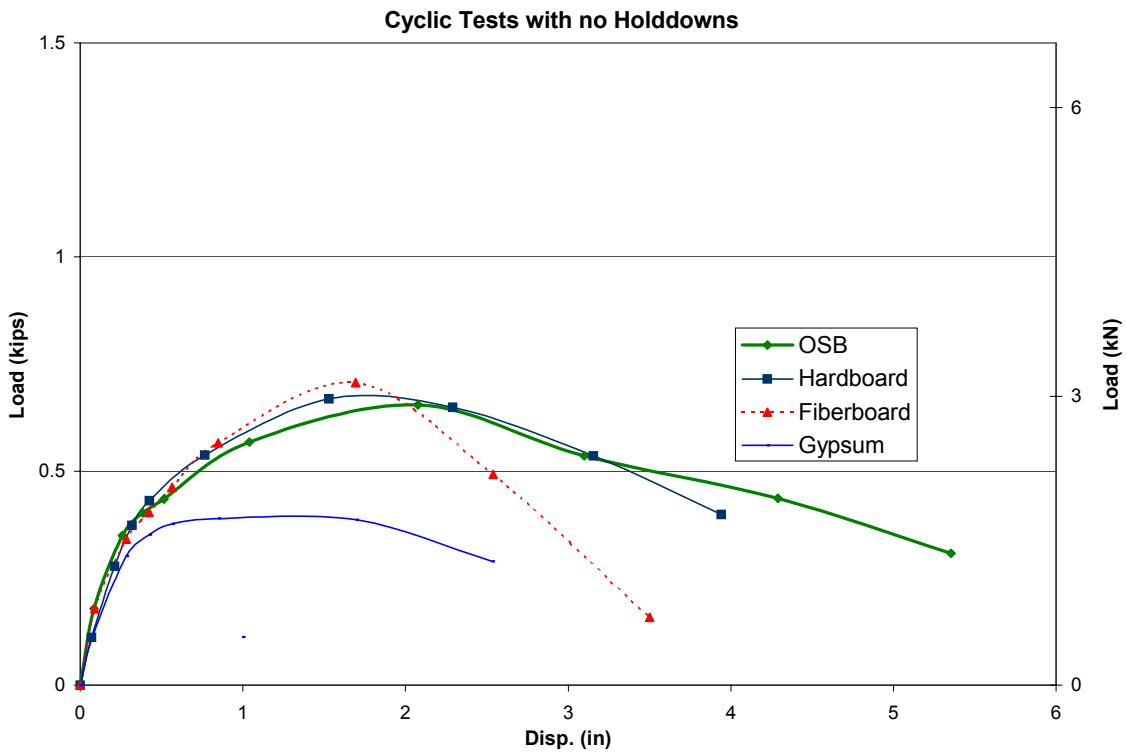

Figure 4.18: Average initial envelope curve of tests without hold-downs

OSB, hardboard, and fiberboard walls reached nearly the same average maximum load of 3.2 kN (0.72 kips). The walls without hold-downs were observed to fail at a lower displacement value resulting from the nails unzipping along the bottom plate. The gypsum was much weaker than the other sheathing materials only resisting 1.96 kN (0.44 kips) at the peak load. The strength of the stabilized cycles typically ranged from 70-75% of the initial cycles as displayed in Table 4.5.

Table 4.5: Initial and stabilized values for walls without hold-downs

	SHEATHING MATERIAL							
	OSB		Hardboard		Fiberboard		Gypsum	
	(kN)	(kips)	(kN)	(kips)	(kN)	(kips)	(kN)	(kips)
Capacity:								
Initial cycle	3.11	0.70	3.29	0.74	3.14	0.71	1.94	0.44
Stabilized cycle	2.25	0.51	2.39	0.54	2.21	0.50	1.47	0.33
Stabilized/Initial	73%		73%		70%		75%	

4.3.2 Failure Capacity and Displacement

As defined in Chapter Two, the failure load was taken to be $0.8F_{peak}$. Since this is obviously based on the maximum load, the walls with the highest strength will also have the largest load capacity at failure. Displacement capacity is an important parameter to investigate and is not a function of the ultimate load. The ability of a structure to dissipate more energy results from it being able to deform without failing.

Although the peak load and failure load for OSB, hardboard, and fiberboard panels were similar, the displacement at failure was different. OSB panels failed at 83mm (3.28 in.) and hardboard panels failed at 76mm (2.98 in.).

Fiberboard panels reached the same maximum load as the other two sheathing panels, but the displacement at failure was much lower. After reaching maximum load, the fiberboard panels experienced a drastic reduction in strength. Displacement at peak load was 38mm (1.5 in.), while the wall failed at a displacement of only 58mm (2.28 in.). The ease with which the nails in the fiberboard could tear through the sheathing could have contributed to the drastic reduction in strength. The nails suddenly tore through the sheathing after peak load, while the sheathing nails in hardboard and OSB typically deformed before total failure.

Table 4.6: Failure displacement of walls without hold-downs

Material	**Cyclic without HD**	
	(mm)	**(in.)**
OSB	83	3.28
Hardboard	76	2.98
Fiberboard	58	2.28
Gypsum	53	2.10

4.3.3 Hysteretic Energy Dissipation

One way to evaluate the energy dissipating capacity of a sheathing panel subjected to cyclic loading is to plot the hysteretic energy during each cycle. When tested using the sequential phased displacement method, the critical cycles are the initial and final stabilized cycles. The plot in Figure 4.19 displays the amount of energy

dissipated by walls without hold-downs during the initial and stabilized cycle of each phase.

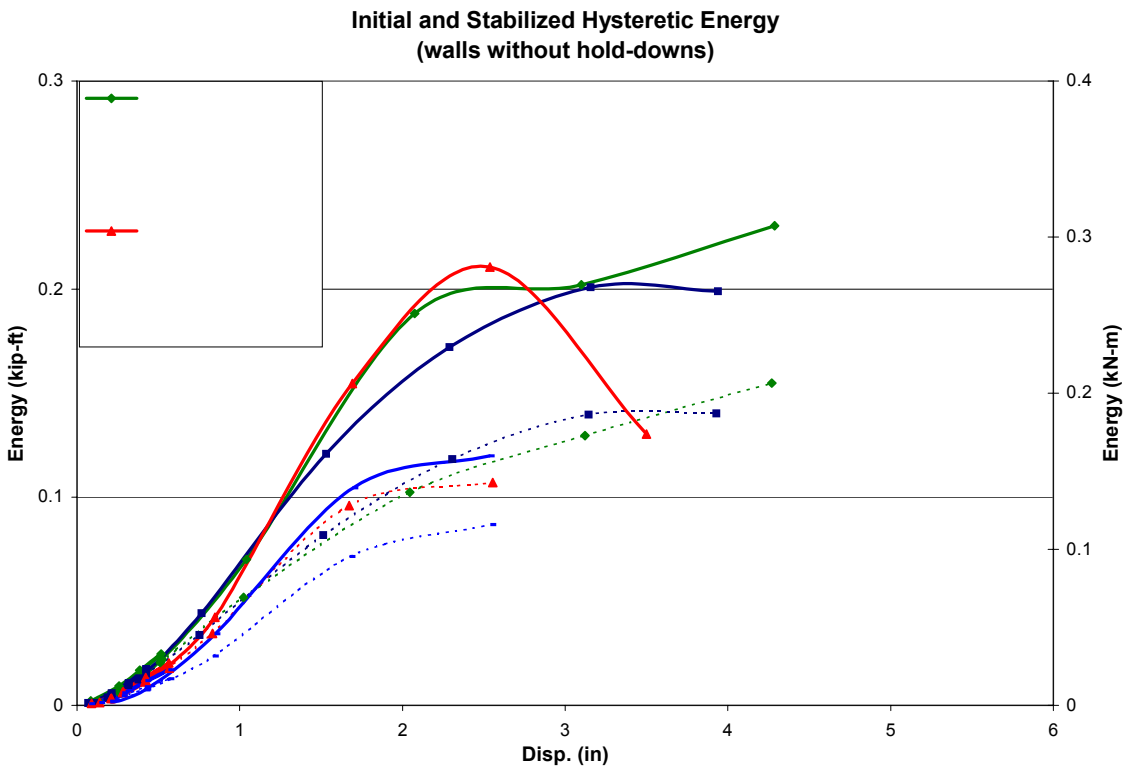

Figure 4.19: Hysteretic energy versus displacement for walls without hold-downs

Plotting the hysteretic energy versus the displacement at a given cycle enables one to compare the energy dissipating capacity of the sheathing materials. OSB, hardboard, and fiberboard all exhibited similar energy dissipating capability until a displacement level of 63.5mm (2.5 in.). At this displacement, the hysteretic energy of fiberboard drastically reduced, while OSB and hardboard leveled off. The drastic reduction observed in fiberboard panels result from the nails tearing through the sheathing along the bottom plate. This hindered the wall from resisting load and dissipating energy.

Although discussed later in this chapter, it should be noted that the hysteretic energy of walls without hold-downs is minimal when compared to walls with hold-downs. Walls without hold-downs exhibit little toughness, which can result in massive damage to a house during an earthquake.

As observed in walls with hold-downs, there was also a large reduction in the hysteretic energy of the initial and stabilized cycles for walls without hold-downs (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.

4.3.4 Equivalent Viscous Damping

The equivalent viscous damping ratio, EVDR, is only an approximation since the walls were 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). Even though it is not appropriate to compare timber structures to structures with other materials when considering the equivalent viscous damping beyond the elastic limit (Lowe and Edwards 1984; Polensek 1988; Foliente 1994), the comparison does provide useful information for modeling the walls as simple single-degree-of-freedom spring-mass systems.

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. As shown in Figure 4.20, the EVDR was the smallest during elastic cycles. The EVDR calculated for walls without hold-downs was random and did not follow a definite pattern. OSB and hardboard panels exhibited similar values with a relatively constant EVDR throughout all displacement levels. The EVDR of fiberboard and gypsum panels was small when the wall was still elastic. As the displacement levels increased, so did the damping ratio. When the hysteresis loops of these materials were examined, it was discovered that the loops did not follow the common shape shown in Figure 2.12. Instead of a gradual increase, the hysteresis loop was much more circular, which increased the damping ratio by increasing hysteretic energy.

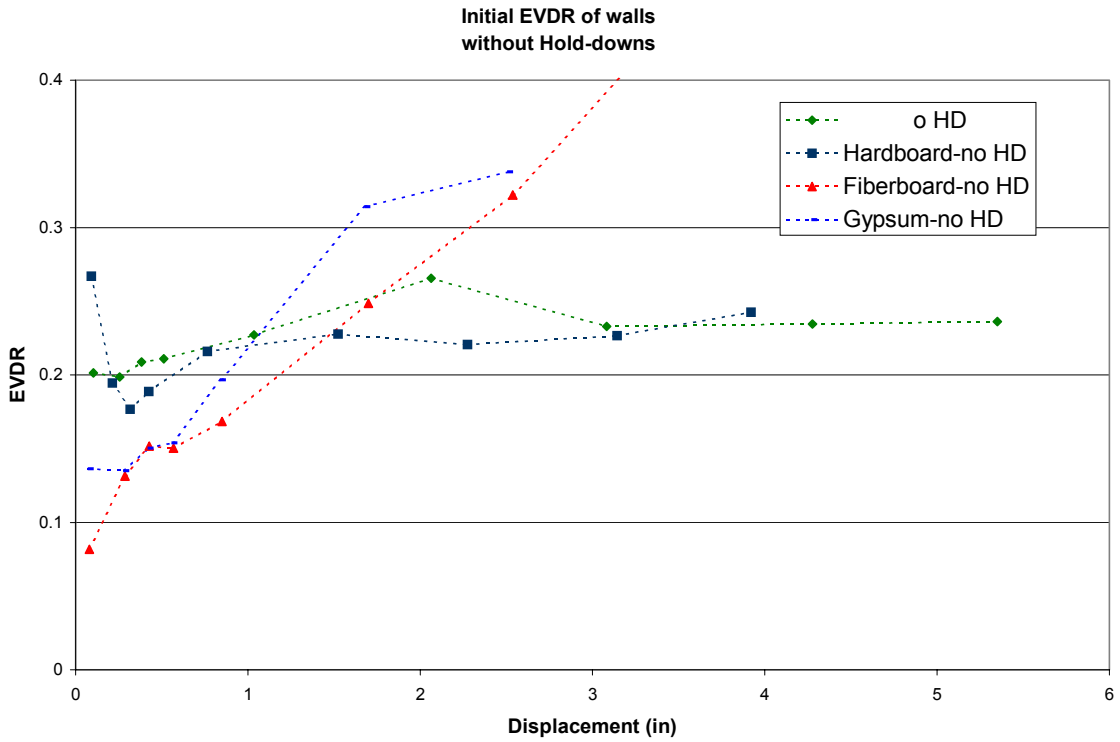

Figure 4.20: EVDR of walls without hold-downs

4.3.5 Cyclic Stiffness

The cyclic stiffness can be calculated during each cycle of a test as described in Chapter 2. 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 pulled out of 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 pull out of 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. 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. As the stiffness decreases, 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 cycles in walls without hold-downs is shown in Figure 4.21. All the sheathing materials behaved similarly. OSB, hardboard, and fiberboard panels degrade at nearly the same value throughout the tests. Since the load displacement graphs were similar for these materials, it confirms that the cyclic stiffness will also be similar. The cyclic stiffness of gypsum was slightly less than the other sheathing materials. 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 discussed in Section 4.2.6.

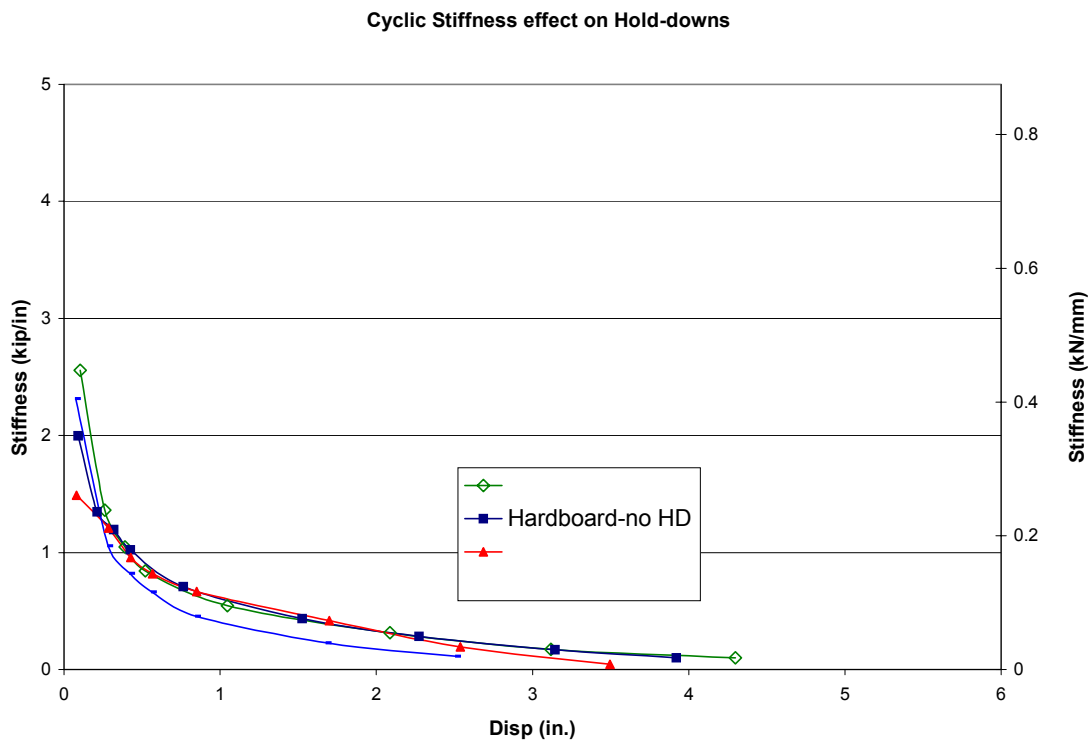

Figure 4.21: Cyclic stiffness of walls without hold-downs

4.3.6 Wall Behavior and Failure Mode

All of the walls without hold-downs failed along the bottom plate. When the walls were not restrained from overturning, all of the force was transferred directly to the bottom plate connection. The nail behavior for walls without hold-downs was similar to its corresponding wall with hold-downs. Refer to section 4.2.7 for the nail behavior of walls with hold-downs. The predominant failure modes of walls without hold-downs can be seen in the pictures below.

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 as shown in Figures 4.22 and 4.23. The end stud-to-bottom plate connection was four end grain nails at the corners, which would easily become separated by withdrawal forces. 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, shown in Figure 4.24, or fracturing perpendicular to the grain at the location of the shear anchor bolt as pictured in Figure 4.25. 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 this type of failure, while the other half experienced the first mode of failure.

Figure 4.22: Wall separated from bottom plate after failure

Figure 4.23: Nail pull through exclusively on bottom plate (fiberboard)

Figure 4.24: Bottom plate splitting parallel to grain

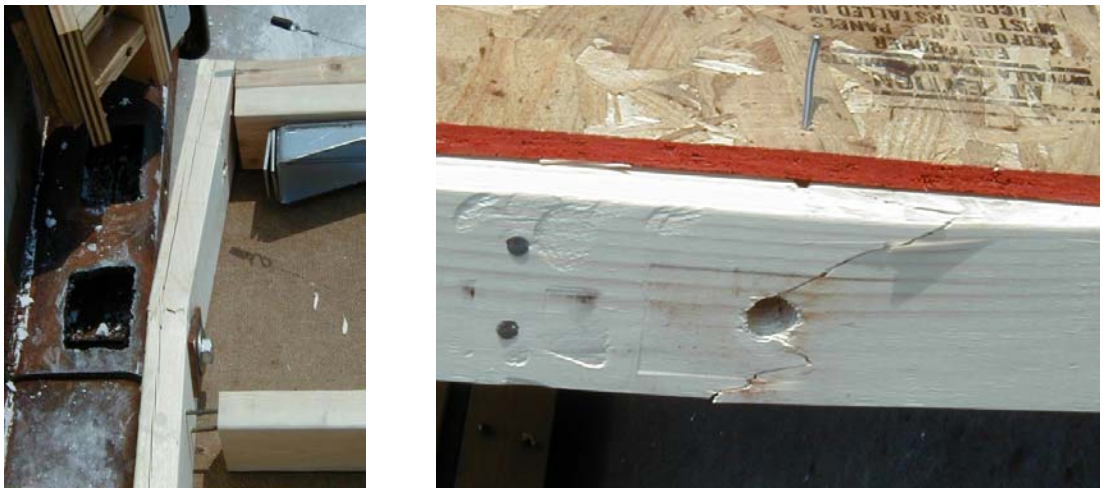

Figure 4.25: Bottom plate fracture at anchor bolt location

4.4 Contribution of Gypsum Wallboard

The contribution of gypsum wallboard subjected to cyclic loading on walls with and without hold-downs is discussed in this section. Two walls were tested with only one panel of gypsum, and two walls each were tested with OSB, hardboard, or fiberboard on the exterior of the wall and gypsum sheathing on the interior of the wall as shown in Table 4.1. The purpose was to determine the performance of the wall when dissimilar sheathing materials was present on opposite sides of the wall.

4.4.1 Load-Displacement Relationship

Performance indicators obtained from the load-displacement curves are ultimate strength, yield strength, displacement at failure, and elastic stiffness. The average initial envelope curves for cyclic tests with hold-downs are displayed in Figure 4.26. The initial envelope curves for cyclic tests without hold-downs are shown in Figure 4.27.

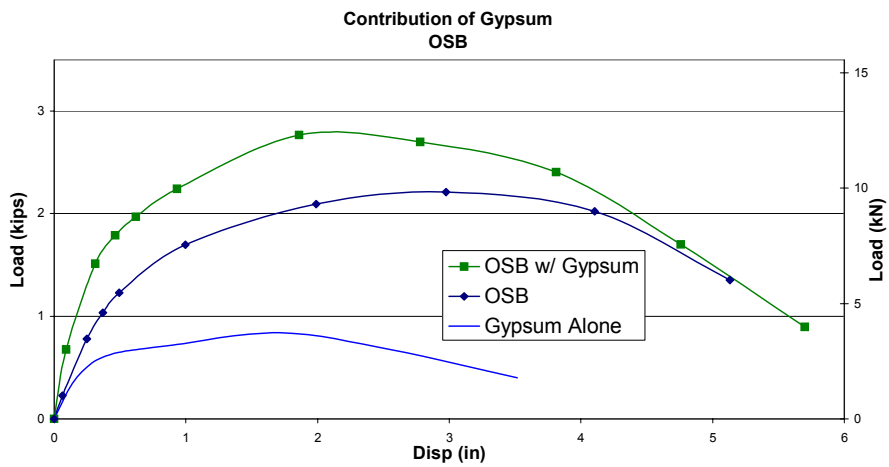

Figure 4.26a: Contribution of gypsum to cyclic tests with hold-downs (OSB)

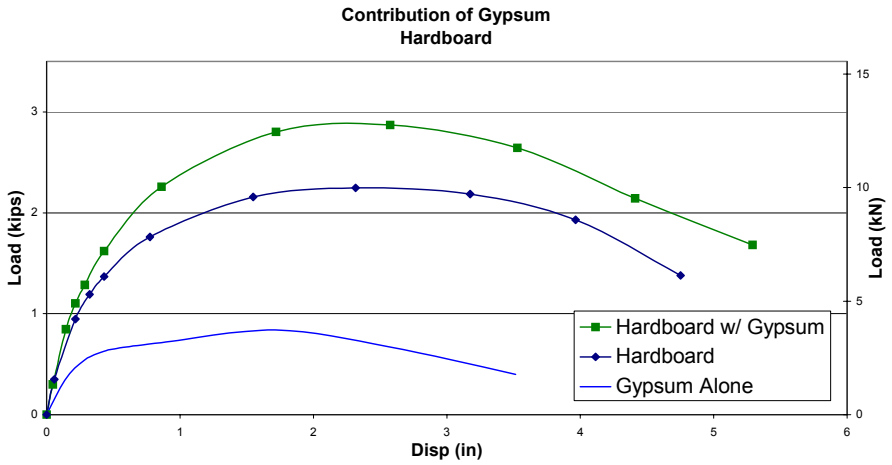

Figure 4.26b: Contribution of gypsum to cyclic tests with hold-downs (hardboard)

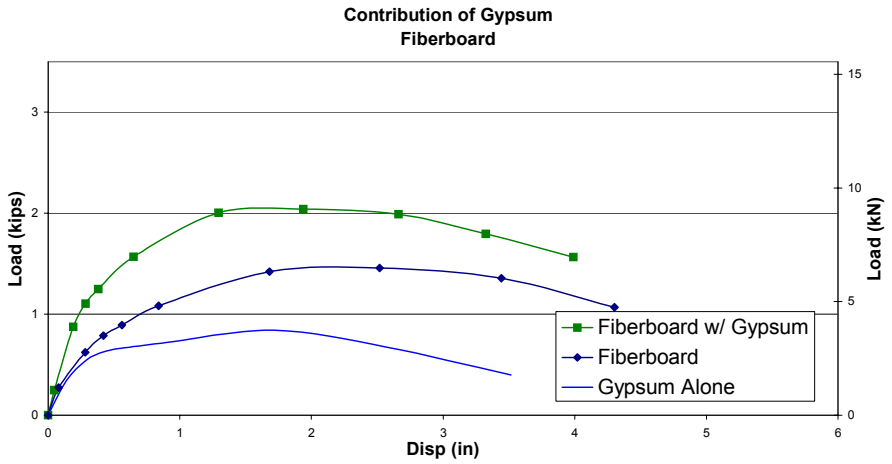

Figure 4.26c: Contribution of gypsum to cyclic tests with hold-downs (fiberboard)

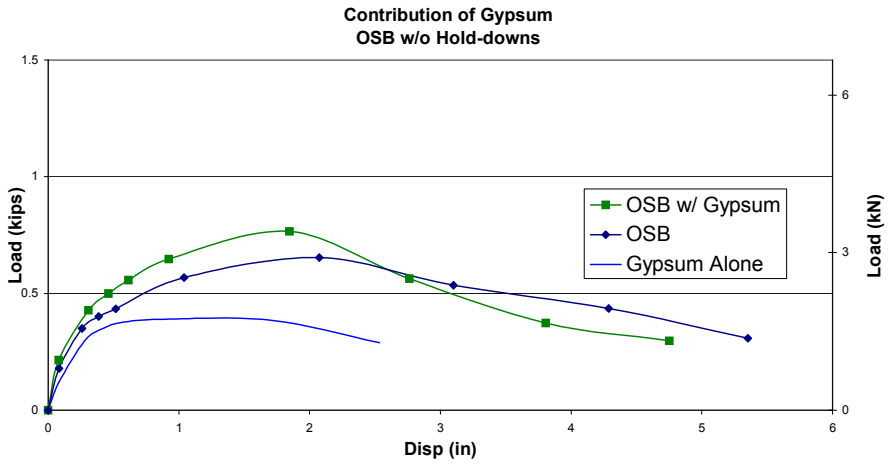

Figure 4.27a: Contribution of gypsum to walls without hold-downs (OSB)

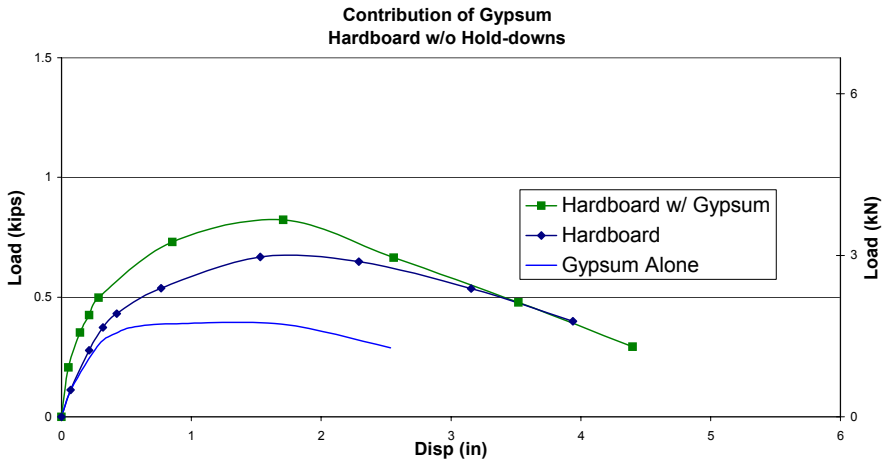

Figure 4.27b: Contribution of gypsum to walls without hold-downs (hardboard)

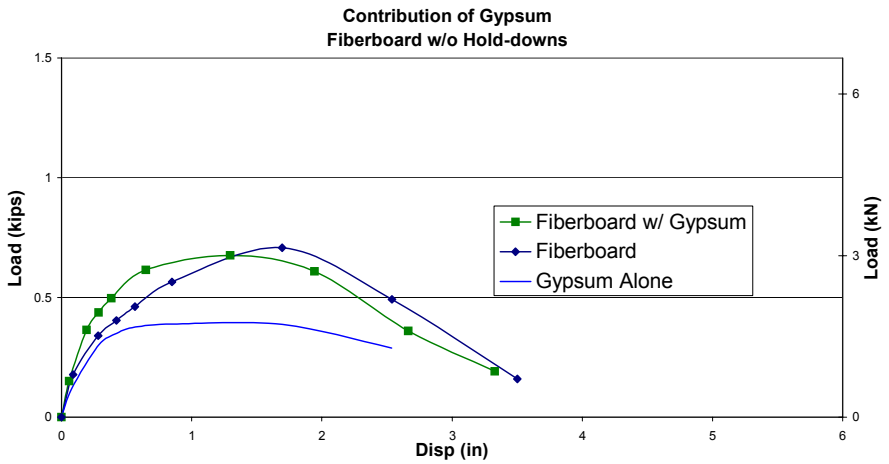

Figure 4.27c: Contribution of gypsum to walls without hold-downs (fiberboard)

The ultimate load of walls sheathed with dissimilar materials on opposite sides of the wall with hold-downs was compared to walls sheathed with one material to determine the contribution of gypsum to the maximum strength of the structure. The peak loads for all walls with hold-downs are displayed in Table 4.7. When sheathed in combination with OSB, the contribution of gypsum was 2.50 kN (0.56 kips). When sheathed in combination with hardboard and fiberboard, the contribution of gypsum was 2.76 kN (0.62 kips) and 2.67 kN (0.60 kips), respectively. When the contribution of gypsum was averaged for the three sheathing materials, the resulting strength was 2.64 kN (0.59 kips). The ultimate load of the walls tested only with gypsum was 3.74 kN (0.84 kips), which indicates that the capacities are not linearly additive, but rather only 70% of the gypsum

capacity was effective in the walls with two types of sheathing. On a per unit length basis, the gypsum contributed 2.2 kN/m (0.15 kip/ft).

Table 4.7: Contribution of gypsum to peak load for walls with hold-downs

Material	Average Peak Load		**Contribution of Gypsum**	
	(kN)	(kip)		
OSB	9.83	2.21	**0.56 (kip)**	**0.14 (kip/ft)**
OSB/GWB	12.3	2.77	**2.49 (kN)**	**2.05 (kN/m)**
Hardboard	10.0	2.25	**0.62 (kip)**	**0.16 (kip/ft)**
Hardboard/GWB	12.8	2.87	**2.76 (kN)**	**2.27 (kN/m)**
Fiberboard	6.48	1.46	**0.60 (kip)**	**0.15 (kip/ft)**
Fiberboard/GWB	9.14	2.06	**2.67 (kN)**	**2.19 (kN/m)**
Only GWB	**0.21**	**0.84**	**0.84 (kip)**	**0.21 (kip/ft)**
			3.74 (kN)	**3.65 (kN/m)**

The ultimate load of the walls sheathed on both sides without hold-downs was compared to the walls sheathed with one material to determine the contribution of gypsum to the maximum strength of the structure. As shown in Table 4.8, the contribution of gypsum to walls without hold-downs is basically zero. The strength of gypsum when tested alone was 1.9 kN (0.44 kips), but when combined with a dissimilar material, the gypsum did not increase the resistance of the wall.

Table 4.8: Contribution of gypsum to peak load for walls without hold-downs

Material	Average Peak Load		**Contribution of Gypsum**	
	(kN)	(kip)		
OSB	3.1	0.70	**0.07 (kip)**	**0.02 (kip/ft)**
OSB/GWB	3.4	0.77	**0.31 (kN)**	**0.26 (kN/m)**
Hardboard	3.3	0.74	**0.09 (kip)**	**0.02 (kip/ft)**
Hardboard/GWB	3.7	0.83	**0.40 (kN)**	**0.33 (kN/m)**
Fiberboard	3.1	0.71	**0.00 (kip)**	**0.00 (kip/ft)**
Fiberboard/GWB	3.0	0.68	**0.00 (kN)**	**0.00 (kN/m)**
Only GWB	**1.9**	**0.44**	**0.44 (kip)** **1.9 (kN)**	**0.11 (kip/ft)** **1.61 (kN/m)**

Elastic stiffness was generally observed to increase when gypsum was included on walls with and without hold-downs. Elastic Stiffness values are shown in Table 4.9. However, the elastic stiffness during hardboard tests was similar with and without gypsum. This can be seen in Figures 4.26 and 4.27. The initial load that occurred in the walls had an influence on the elastic stiffness, but the tendency of gypsum to increase the elastic stiffness was observed. For walls with hold-downs, the increase in the elastic stiffness and the ultimate strength increased the yield load by a proportional amount. For walls without hold-downs, the elastic stiffness increased with the addition of gypsum, but the ultimate load was not affected.

Table 4.9: Effects of gypsum on elastic stiffness

Material	**With Hold-downs**		**Without Hold-downs**	
	(kN/mm)	**(kip/in)**	**(kN/mm)**	**(kip/in)**
OSB	0.59	3.37	0.27	1.57
OSB/GWB	1.10	6.28	0.38	2.17
Hardboard	0.92	5.27	0.23	1.34
Hardboard/GWB	0.83	4.74	0.43	2.48
Fiberboard	0.51	2.91	0.24	1.39
Fiberboard/GWB	0.93	5.34	0.39	2.22
Gypsum	0.45	2.56	0.22	1.25

Displacement at failure when using gypsum on walls with hold-downs either remained constant, or decreased when compared to its corresponding wall without gypsum as seen in Table 4.10. The failure displacement of walls sheathed with OSB and fiberboard decreased by 13mm (0.51 in.) when gypsum was included. The failure displacement of the hardboard walls stayed constant when gypsum was included. Gypsum appears to increase the elastic stiffness and the ultimate load of the walls. However, after reaching the ultimate load, the slope of the envelope curve tends to decrease faster in the walls with gypsum included. The increase in the slope indicates that the wall fails at a quicker rate, and reduces the displacement capacity of the structure. This tends to be a characteristic of the gypsum material when used with a dissimilar material.

Table 4.10: Effects of gypsum on failure displacement for walls with hold-downs

Material	Failure Disp.		Change in Disp.	
	(mm)	(in.)	(mm)	(in.)
OSB	116	4.58	-13.0	-0.51
OSB/GWB	103	4.07		
Hardboard	106	4.17	0.5	0.02
Hardboard/GWB	106.5	4.19		
Fiberboard	109	4.29	-13.0	-0.51
Fiberboard/GWB	96	3.78		

Displacement at failure when including gypsum on walls without hold-downs typically decreased when compared to its corresponding wall without gypsum as displayed in Table 4.11. The average failure displacement of the walls decreased by 10mm (0.4 in.) when gypsum was included. As shown in Figure 4.27, the envelope curve is generally the same until the walls reach peak load. After reaching peak load, the envelope curves tended to decrease at a faster rate when gypsum was included, which led to a reduced failure displacement. This tends to be a characteristic of the gypsum material when used with a dissimilar material.

Table 4.11: Effects of gypsum on failure displacement for walls without hold-downs

Material	Failure Disp. (mm)	Failure Disp. (in.)	Change in Disp. (mm)	Change in Disp. (in.)
OSB	83	3.28	-16	-0.63
OSB/GWB	67	2.65		
Hardboard	76	2.98	-10	-0.4
Hardboard/GWB	66	2.58		
Fiberboard	58	2.28	-3.6	-0.14
Fiberboard/GWB	54	2.14		

As presented in Table 4.12, there does not appear to be a direct correlation between the addition of gypsum panels to the ductility of the specimens. The coefficient of variation was too large to make accurate comparisons. Gypsum generally decreased the failure displacement, but the higher elastic stiffness tended to decrease the yield displacement. Therefore, the change in the ratio of the failure displacement to the yield displacement was erratic and the ductility changed accordingly.

Table 4.12: Effects of gypsum on ductility

	Walls with Hold-downs			Walls without Hold-downs		
Material	**Ductility**	**COV (%)**	**% Increase**	**Ductility**	**COV (%)**	**% Increase**
OSB/GWB	7.66 10.5	30.0 14.2	+37	9.2 8.0	35.4 3.5	-13
Hardboard Hardboard/GWB	10.41 7.5	2.7 5.2	-28	6.7 8.7	29.6 1.6	30
Fiberboard Fiberboard/GWB	9.42 10.5	26.7 9.4	+11	5.4 7.4	36.7 11.5	37
Gypsum	8.94	14.9	0	6.9	23.7	0

4.4.2 Hysteretic Energy

Plotting the hysteretic energy versus the displacement at a given cycle enables one to compare the energy dissipating capacity of the sheathing materials. The graphs displayed in Figures 4.28 and 4.29 show the effects of gypsum on the hysteretic energy with and without hold-downs, respectively. It can be seen from Figure 4.28 that the hysteretic energy increases when gypsum 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 the gypsum panels was larger than when combined with another sheathing material. Therefore, the gypsum is not linearly additive in regards to hysteretic energy, but does contribute.

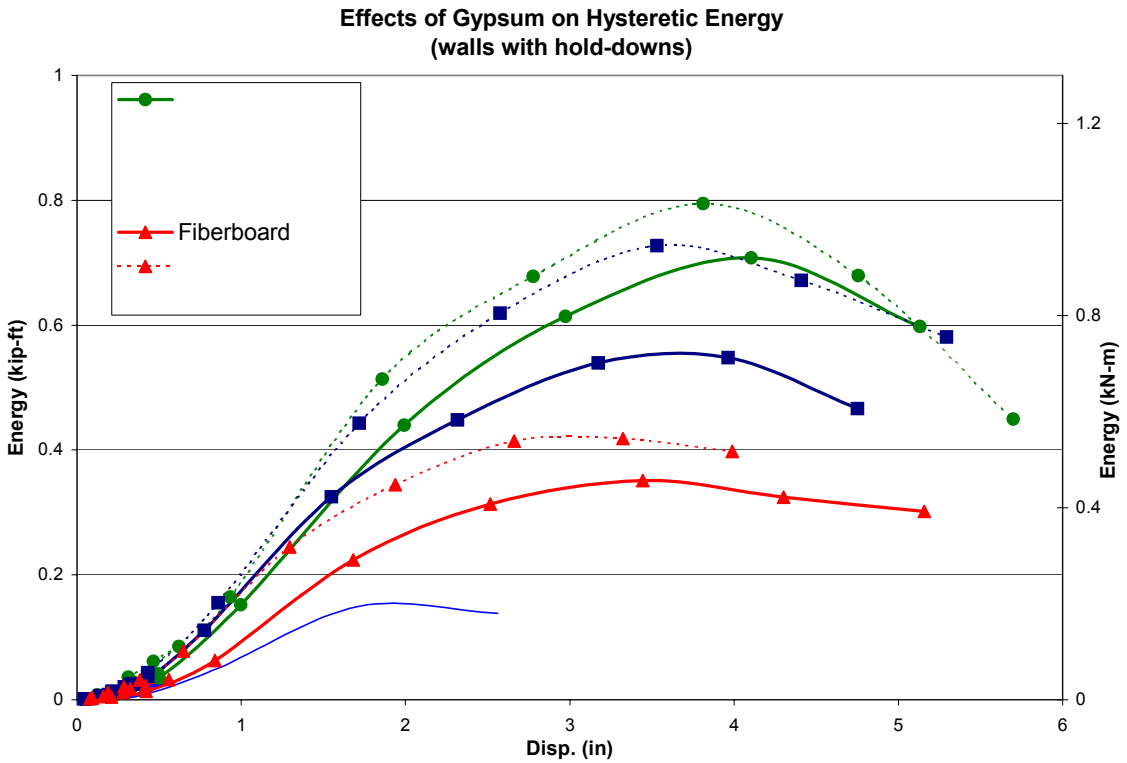

Figure 4.28: Contribution of gypsum to hysteretic energy in walls with hold-downs

For walls without hold-downs, it can be seen from Figure 4.29 that gypsum does not increase the hysteretic energy when combined with a dissimilar sheathing material. When compared to the values in Figure 4.28, the walls without hold-downs do not exhibit much toughness, and the gypsum does not contribute to the hysteretic energy.

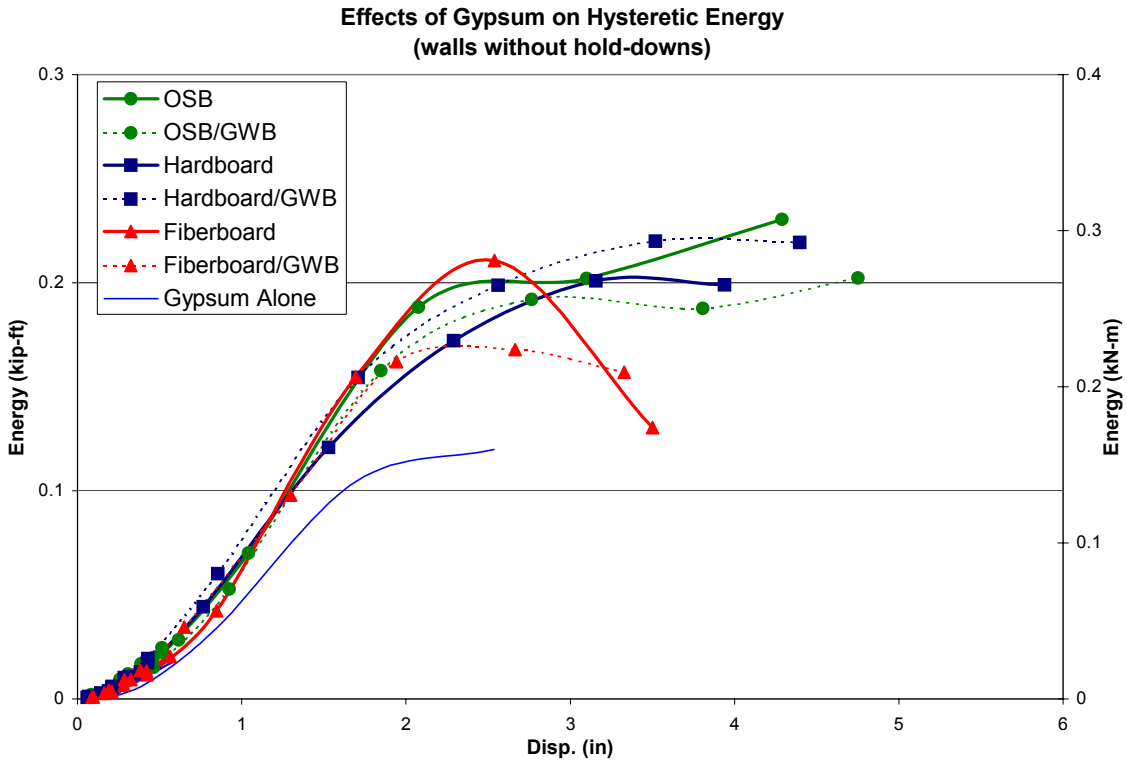

Figure 4.29: Contribution of gypsum to hysteretic energy without hold-downs

4.4.3 Equivalent Viscous Damping

The equivalent viscous damping ratio, EVDR, is only an approximation since the walls were loaded quasi-statically beyond the elastic limit. Theoretically, the viscous damping should be zero at low loading rates where the inertial forces are zero. The EVDR is relatively unaffected by the addition of gypsum on walls with hold-downs, as shown in Figure 4.30. The walls with gypsum follow the same curve as the walls without gypsum so it can be stated that gypsum contributes nothing or very little to the viscous damping of a shear wall. The reason is that when gypsum 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. As shown in Figure 4.31, a direct relationship between the viscous damping ratio and gypsum cannot be made for walls without hold-downs. At low displacement levels, the damping ratio is lower when gypsum is included, but as the displacement levels increase, so does the damping ratio of the gypsum walls.

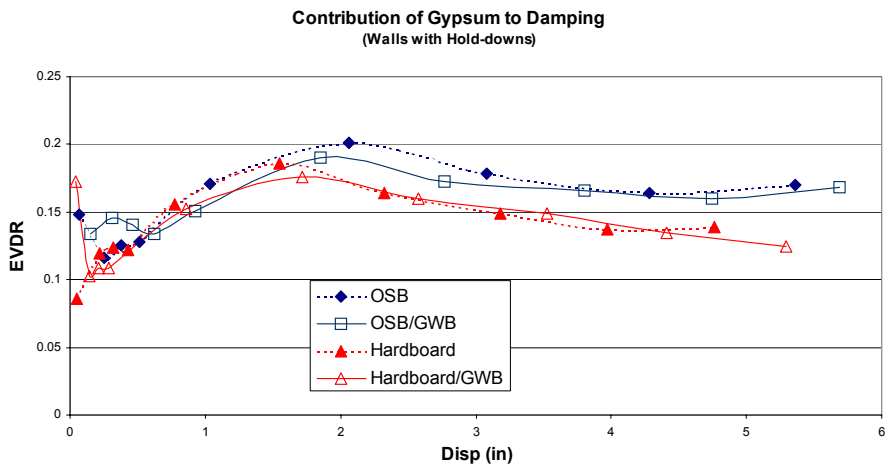

Figure 4.30: Effect of gypsum on damping for walls with hold-downs

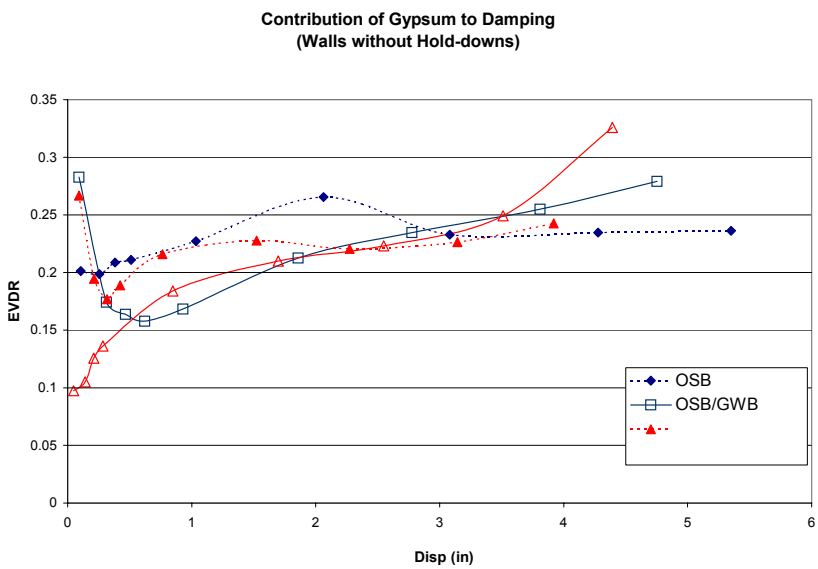

Figure 4.31: Effect of gypsum on damping for walls without hold-downs

4.4.4 Cyclic Stiffness

When plotted against displacement, the cyclic stiffness 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. Every cycle produces more damage to the sheathing-to-framing connection, which produces a reduction in cyclic stiffness. The same was noticed for walls with gypsum included. The graph in Figure 4.32 shows the relationship when gypsum panels are included on OSB-sheathed walls, but the same relationship exists for the other sheathing materials. For walls with hold-downs, the gypsum increased the cyclic stiffness by a substantial amount

when the displacements were small. By the time the walls began to fail, typically 76mm (3 in.), the contribution of gypsum was zero, and the two curves were equal. The gypsum panels would always fail first, so at higher displacements, it could not contribute to the performance of the walls. For walls without hold-downs, gypsum did not contribute to the cyclic stiffness of the wall. The curves are almost identical for every displacement level.

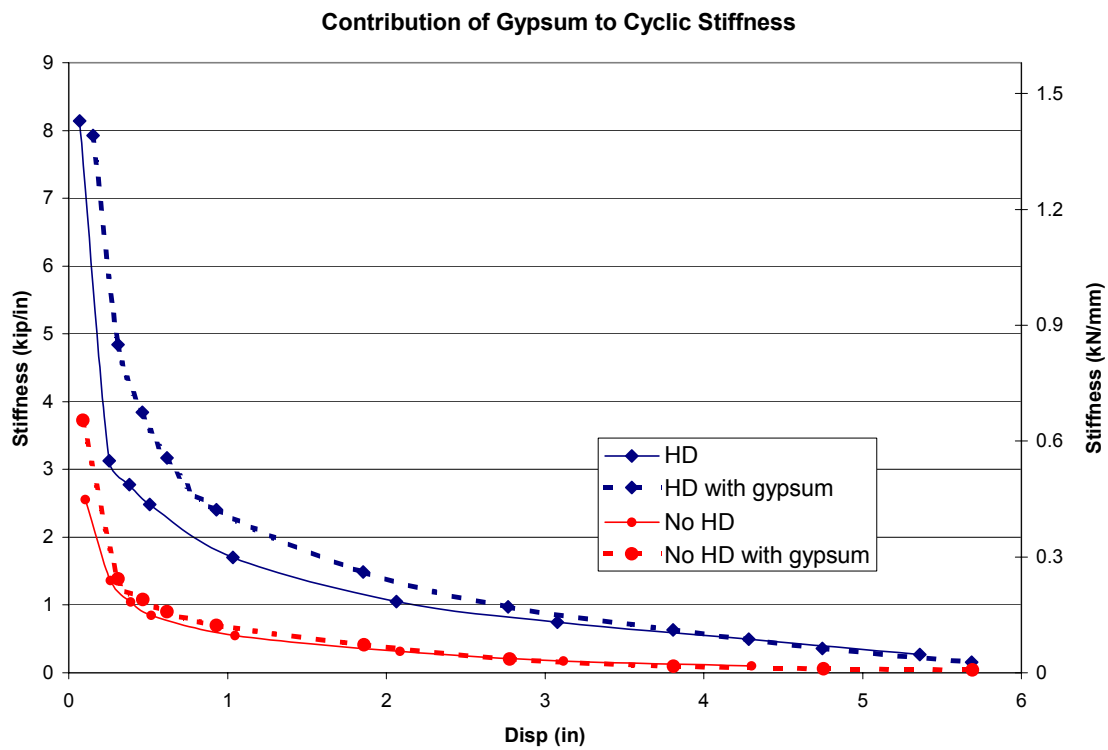

Figure 4.32: Contribution of gypsum to cyclic stiffness

4.5 Effects of Hold-downs

Half of the cyclic tests were performed with hold-downs, and half were performed without hold-down connections. For the walls without hold-downs, the only connection of the foundation to the bottom plate was two shear bolts placed approximately one foot from the ends of the wall. None of the monotonic tests were performed without hold-down connections. Every parameter was kept constant during the construction of the walls, except the use of hold-downs.

The cyclic protocol used to test the walls with hold-downs was also used to test the walls without hold-downs. This was done because no walls subjected to monotonic

loading were tested without hold-downs, and the information obtained from the monotonic tests was used in the cyclic protocol. This was also done for comparison purposes.

When walls are tested cyclically with hold-downs, the performance of walls longer than four feet are similar on a unit length basis. However, when walls are tested with only intermediate shear anchor bolts, the strength, stiffness, and ductility are dependant on the aspect ratio and number of panels in the wall. Salenikovich (2000) showed that four-foot walls were half as strong (on a unit length basis) as eight-foot walls, but the displacement at failure was 50% larger.

The walls in this study were tested with no in-plane gravity loads (dead load) in order to test the most conservative condition. When the walls were fully restrained and overturning was restricted, the addition of gravity loads would not greatly affect the performance of the wall. Considering walls not restrained against overturning moments (i.e. walls without hold-downs), the addition of in-plane gravity loads would make a significant difference to the performance of the wall. If dead loads were applied in-plane at the top of the wall, the amount of uplift in the structure would be reduced and allow the wall to resist more load before the framing-to-sheathing connection was damaged. Average load-displacement envelope curves for the four materials tested are shown in Figure 4.33 to illustrate the effect of using overturning anchors.

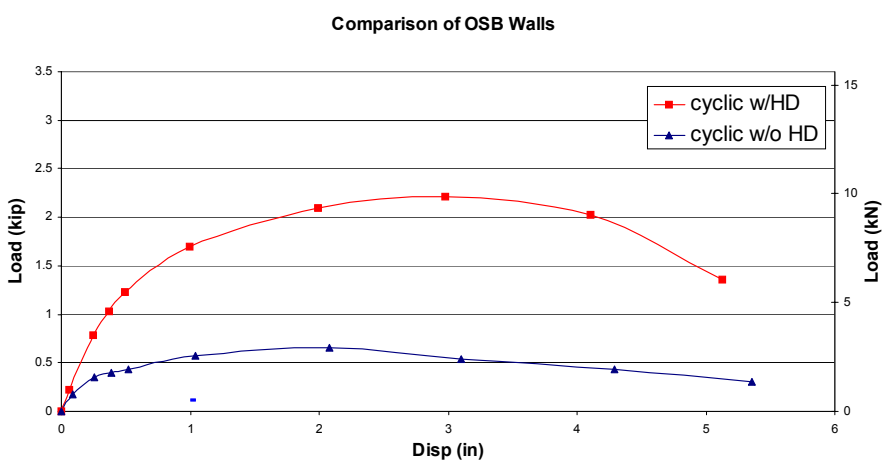

Figure 4.33a: Load-Displacement curves for cyclic tests (OSB)

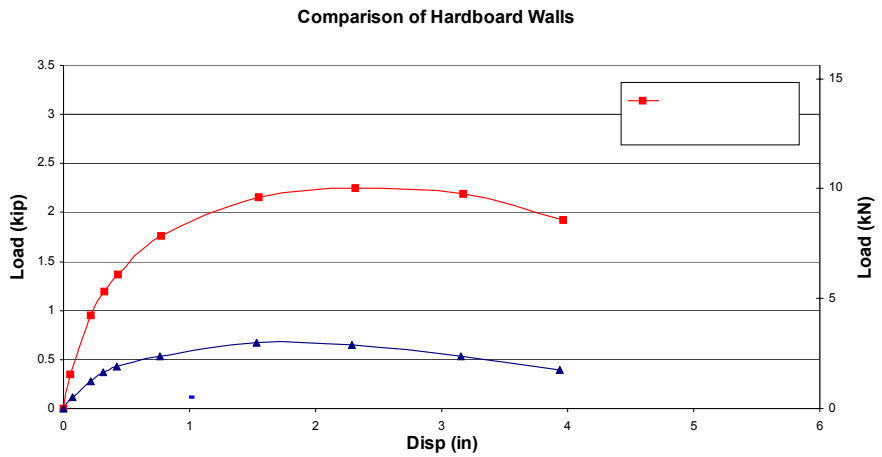

Figure 4.33b: Load-Displacement curves for cyclic tests (hardboard)

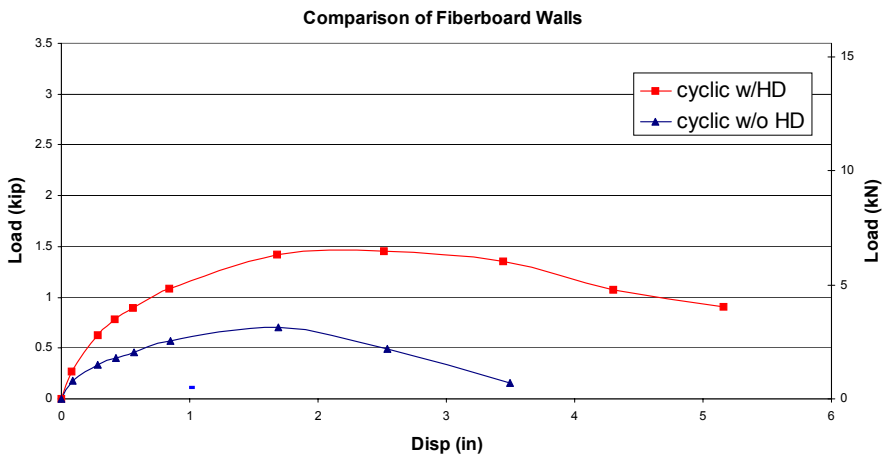

Figure 4.33c: Load-Displacement curves for cyclic tests (fiberboard)

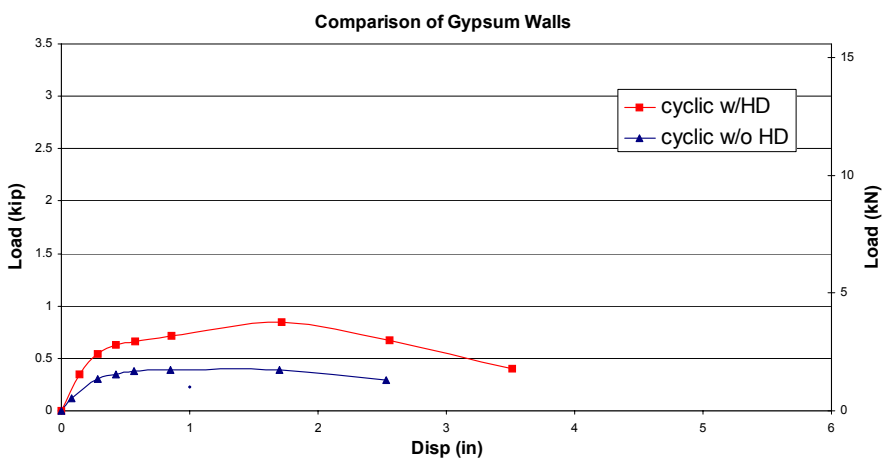

Figure 4.33d: Load-Displacement curves for cyclic tests (gypsum)

4.5.1 Load-Displacement Relationship

The amount of overturning restraint has a direct influence on the performance of a shear wall, especially the ultimate load and elastic stiffness. Heine (1997) tested the effects of overturning restraint on 2.4 x 12.2m (8 x 40ft) walls with various amounts of openings. It was shown that as the amount and size of the openings in the wall increased, the effect of hold-downs had a more drastic effect. Therefore, it is evident that the effects of overturning restraint will greatly influence the performance of the four-foot walls tested during this study.

The average reduction in the ultimate strength when hold-downs were not included was almost two-thirds the value when hold-downs were included as displayed in Table 4.13. The sheathing materials most affected by the absence of hold-downs were OSB and hardboard panels. The sheathing materials least affected by the absence of hold-downs were fiberboard and gypsum panels, with slightly less than 50% reduction.

Table 4.13: Percent reduction in peak load without hold-downs

Material	Cyclic with HD		Cyclic without HD		Reduction
	(kN)	(kips)	(kN)	(kips)	
OSB	9.8	2.21	3.1	0.70	68%
Hardboard	10.0	2.25	3.3	0.74	67%
Fiberboard	6.5	1.46	3.1	0.71	49%
Gypsum	3.7	0.84	1.9	0.44	48%
OSB/GWB	12.3	2.77	3.4	0.77	72%
Hardboard/GWB	12.8	2.87	3.7	0.83	71%
Fiberboard/GWB	9.2	2.06	3.0	0.68	67%

When gypsum was included with another sheathing material, the reduction in strength was the highest. This can be explained by the fact that gypsum contributed very little, or nothing to the strength of the wall when tested with no hold-downs. When tested with hold-downs, gypsum contributed an average of 2.6 kN (0.59 kip), but when hold-downs were not included, the gypsum contributed on average only 0.2 kN (0.05 kips).

It was also observed that OSB, hardboard, and fiberboard all achieved close to the same ultimate strength without hold-downs (3.2 kN, 0.72 kips average). With hold-

downs, fiberboard panels were typically much weaker than OSB and hardboard panels. This indicates that fiberboard and hardboard may be considered effective sheathing materials for prescriptive construction.

As expected, the elastic stiffness of the specimens decreased by a significant amount when hold-downs were not included. The stiffness values for the configurations tested are presented in Table 4.9. When considering all of the tests, the average reduction in the elastic stiffness was 60% when hold-downs were not included. Hardboard-sheathed walls experienced the largest reduction in stiffness (75%), while fiberboard-sheathed walls only experienced a reduction of 52 %. There was no major change in the average elastic stiffness reduction in the walls that included gypsum.

Overturning restraint appears to have an impact on the ductility factor of the specimens. Values determined for ductility are shown in Table 4.12. Due to the large variation that occurred when calculating the ductility factor, a comparison of the exact values cannot be made. However, a slight to moderate decrease in the ductility was observed in nearly all tests when hold-downs were not included. The failure displacement decreased in the absence of overturning restraints, but the elastic stiffness also decreased, which affected the ductility of the structure.

One sheathing material in particular that experienced the largest reduction in ductility was the fiberboard panel. With hold-downs, the fiberboard panels averaged a ductility factor of 10. When hold-downs were not included, the ductility factor dropped to 6.4. The ductility factor of the walls without hold-downs dropped significantly because the failure displacement decreased much more than the elastic stiffness.

For all other tests, fiberboard was much weaker than OSB and hardboard, and had a lower elastic stiffness. However, when tested cyclically with no hold-downs, the fiberboard reached the same peak load and had a similar elastic stiffness, but the failure displacement was much lower and the ductility factor decreased accordingly.

4.5.2 Failure Capacity and Displacement

As defined in Chapter two, the failure load was taken to be $0.8F_{peak}$. Since it was already noted that the peak load of the walls with hold-downs is much larger than the peak load of walls without hold-downs, it is obvious that the failure load will also be

much larger for walls with hold-downs. The reduction in the failure displacement was also significant in walls without hold-downs.

The values for the displacement at failure are presented in Table 4.14. The average reduction in the failure displacements was 35.6mm (1.4 in.) or 35%. The most drastic reduction occurred in the fiberboard-sheathed walls, which experienced a reduction of 51mm (2 in.). Hold-downs enabled the sheathing nails along the entire perimeter of the wall to effectively resist shear, but during tests without hold-downs, the sheathing nails along the bottom plate were responsible for resisting both shear and overturning loads. Given the weakness of the fiberboard material, when the forces were restricted to only the bottom plate, the nails could easily tear through the sheathing and enabled failure to occur at a much lower displacement.

Table 4.14: Displacement at failure comparison of hold-downs

Material	Cyclic with HD		Cyclic without HD		% Decrease
	(mm)	(in.)	(mm)	(in.)	
OSB	116	4.58	83	3.28	28%
Hardboard	106	4.17	76	2.98	29%
Fiberboard	109	4.29	58	2.28	47%
Gypsum	65	2.55	53	2.10	18%
OSB/GWB	103	4.07	67	2.65	35%
Hardboard/GWB	106	4.19	66	2.58	38%
Fiberboard/GWB	96	3.78	54	2.14	43%

The displacement at failure also tended to decrease when gypsum was included. When the walls were sheathed on only one side, the average reduction in failure displacement was 31.5mm (1.24 in.) for walls without hold-downs. When gypsum was included with a dissimilar sheathing material, the average reduction was 40mm (1.56 in.). The use of gypsum in walls without hold-downs contributes no additional resistance to the wall, and actually decreases the wall's displacement capacity.

4.5.3 Hysteretic Energy Dissipation

As shown in Figure 4.34, a drastic reduction in hysteretic energy is observed when walls do not have overturning restraints. The most drastic reduction occurs for OSB panels. The reason is because when hold-downs were not included, the sheathing

materials displayed similar energy dissipating abilities. But when hold-downs were included, OSB panels displayed the largest hysteretic energy. The least reduction occurs for fiberboard panels because it had the lowest hysteretic energy 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 the shear forces. It is also observed from the graph that the peak hysteretic energy is achieved at a much lower displacement for walls without hold-downs.

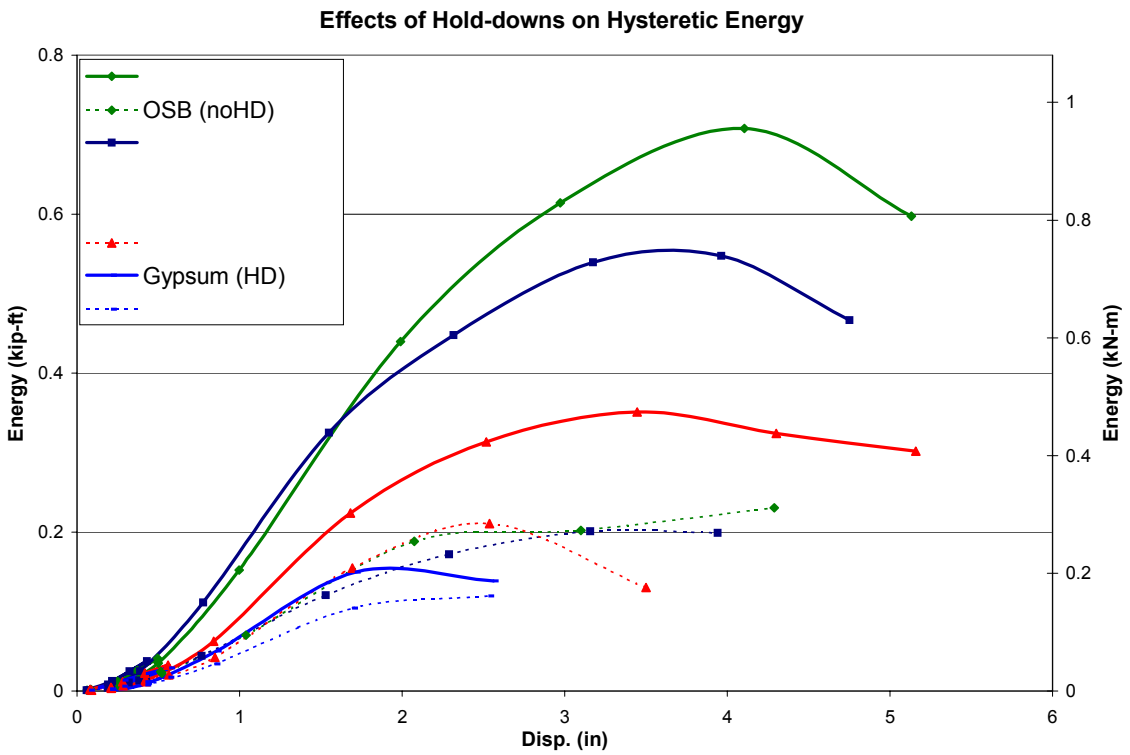

Figure 4.34: Effects of hold-downs on hysteretic energy

4.5.4 Equivalent Viscous Damping

The equivalent viscous damping ratios for walls without hold-downs 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-downs, the bottom of the wall could easily separate. This additional interstory drift 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 the hold-down and non hold-down walls should be similar. As shown in Figure 4.35, the curves are closer together during the low displacements. After the walls reach peak load, the EVDR of the walls with no hold-downs begin to increase compared to the walls with hold-downs. This is most evident in the walls sheathed with hardboard, fiberboard and gypsum. The large increase in EVDR of the fiberboard occurs after peak load is reached.

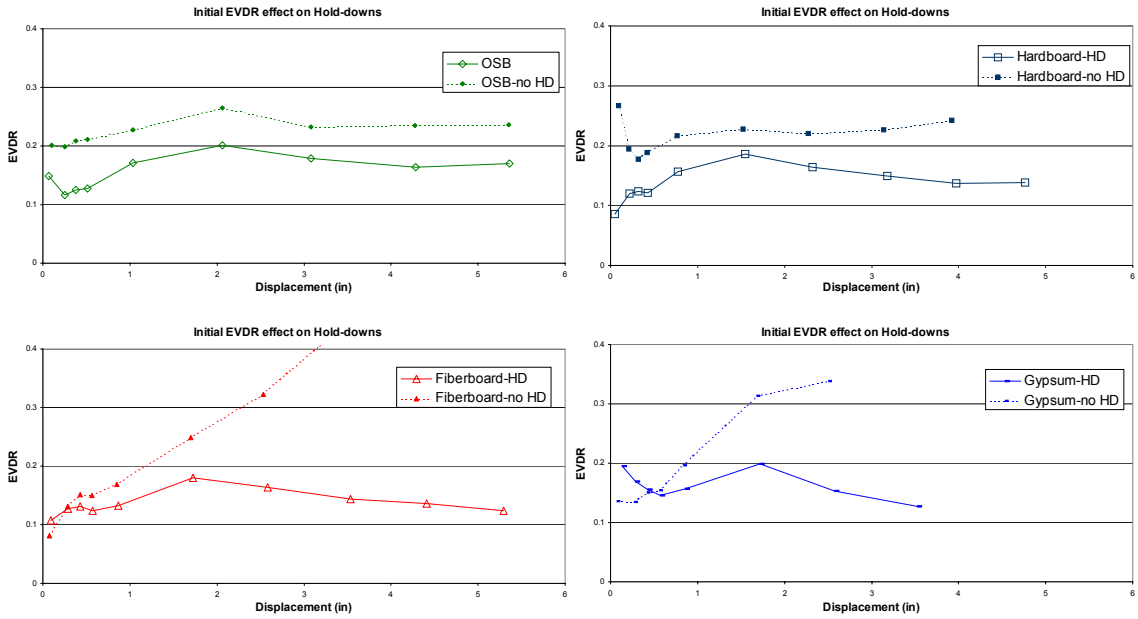

Figure 4.35: Effect of hold-downs on damping

4.5.5 Cyclic Stiffness

Cyclic stiffness can be calculated during each cycle of a test as described in Chapter 2. When plotted against displacement, it can serve as an indication of the stiffness degradation of the structure subjected to cyclic loading. The effects of using hold-downs compared to no hold-downs are demonstrated in Figure 4.36. OSB is the sheathing material plotted in Figure 4.36, but the cyclic stiffness of all other sheathing materials performed in a similar manner. There is a definite increase in the cyclic stiffness when hold-downs are used. The increase gets 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 basically zero, while there is still some stiffness in the walls with hold-downs after failure.

The cyclic stiffness when gypsum is included follows the same trend as the peak load. In walls with hold-downs, the gypsum contributes to the cyclic stiffness until the wall reaches the peak load. After maximum load, the gypsum has failed and cannot contribute to the cyclic stiffness of the wall. In walls without hold-downs, the gypsum did not contribute to the cyclic stiffness at any displacement level. This reiterates the point that gypsum does not contribute to the performance of four-foot shear walls when hold-down restraints are not present.

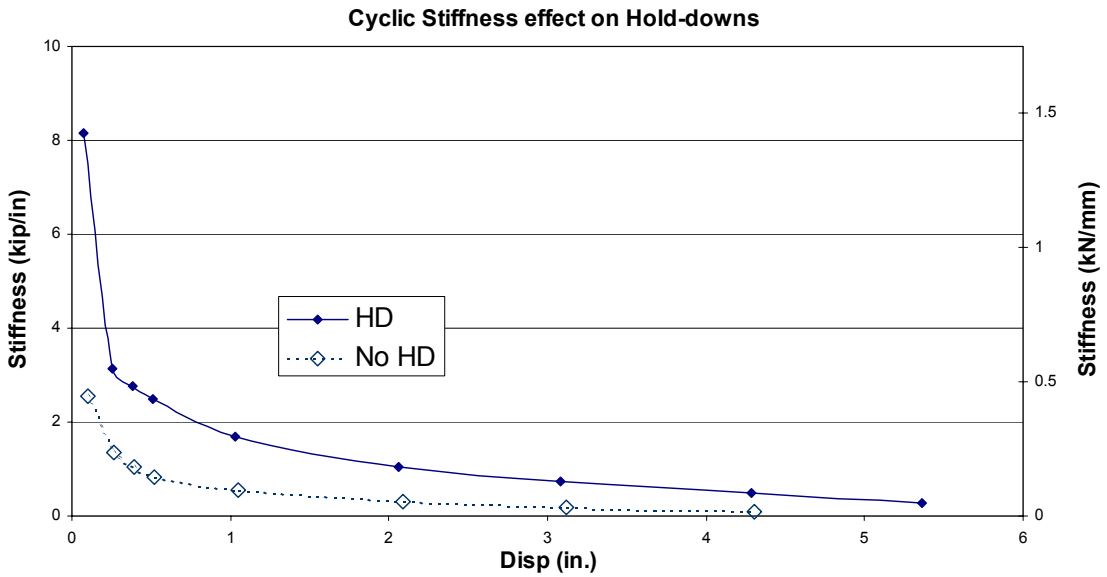

Figure 4.36a: Effect of hold-downs on cyclic stiffness (OSB)

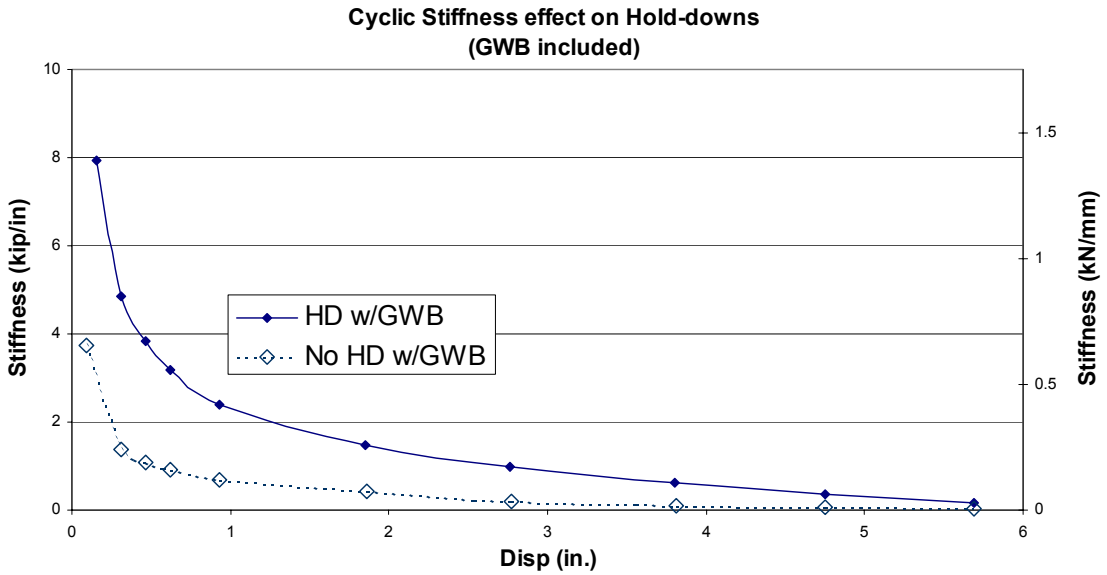

Figure 4.36b: Effect of hold-downs on cyclic stiffness with gypsum (OSB)

4.5.6 End Stud Uplift

Vertical displacement of the end studs was measured for each test, from which the amount of rigid body rotation can be calculated. The end studs experience uplift by either tension, which tries to pull the end stud away from the bottom plate, or compression, which forces the end stud to crush the bottom plate. When hold-downs are included, the end studs cannot move away from the bottom plate and the uplift is small. When hold-downs are not included, only the sheathing nails and the stud to plate connection are holding the wall together. Since the stud to plate connection is an end grain nail connection, it has virtually no resistance to withdrawal. Therefore, the sheathing nails along the bottom plate are responsible for holding the wall together. This illustrates why the walls without hold-downs always failed by the sheathing nails unzipping along the bottom plate, allowing the wall to separate from the bottom plate. On the other hand, walls with hold-downs enabled the load to distribute throughout the perimeter of sheathing nails exhibiting a higher degree of load sharing.

The difference in the amount of uplift at the peak load when hold-downs were not present was typically two to three times the amount when hold-downs were included. The amount of uplift for walls with and without hold-downs is shown in Table 4.15. 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-downs 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-downs began to increase very rapidly and by failure, the uplift was an order of magnitude larger than walls with hold-downs as seen in Figure 4.37.

Table 4.15: 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
Gypsum:				
Left end stud	3.6	0.14	8.9	0.35
Right end stud	3.3	0.13	7.1	0.28

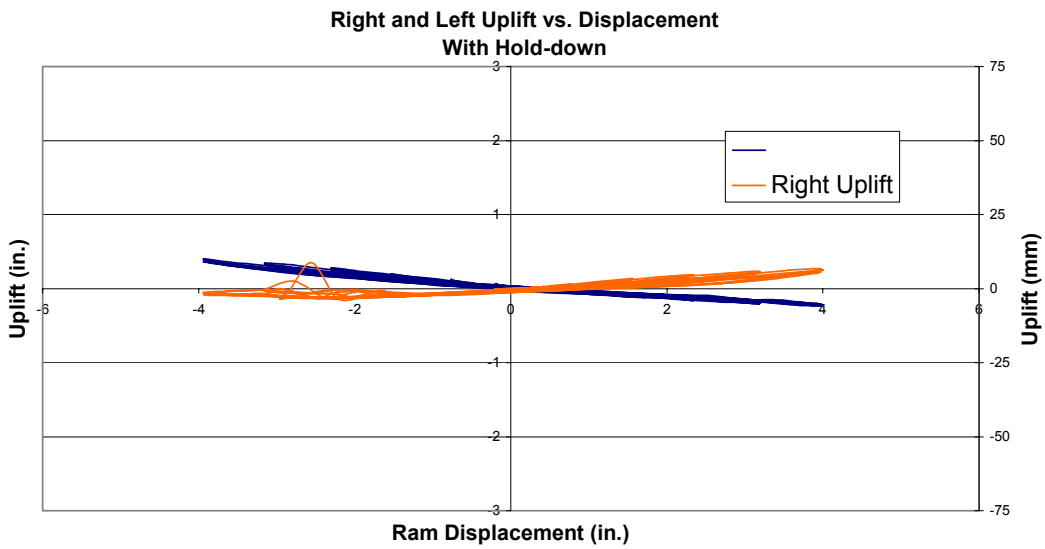

Figure 4.37a: Vertical uplift in walls with hold-downs

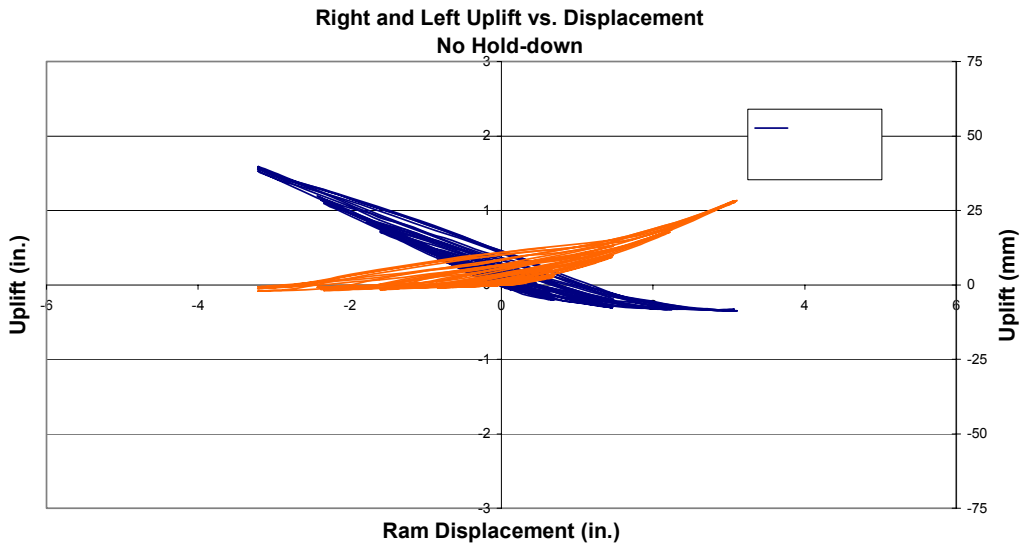

Figure 4.37b: Vertical uplift in walls without hold-downs

4.6 Summary

A total of 28 walls were tested 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 in-plane gravity loads applied to the walls. This chapter provided information regarding the performance of OSB, hardboard, fiberboard, and gypsum-sheathed walls. Also of interest is the contribution of gypsum when included with a dissimilar material on the opposite side of

the wall, and the effects of using hold-down connectors. The conclusions drawn from the cyclic tests are:

(1) For walls with hold-downs, hardboard-sheathed walls resisted the largest load of all the sheathing materials, 10.0 kN (2.25 kips). The OSB walls could resist an average maximum load of 9.83 kN (2.21 kips). Fiberboard walls were much weaker and could only resist 6.48 kN (1.46 kips). Gypsum was by far the weakest material, only resisting 3.7 kN (0.84 kips).

(2) Displacement at failure of hardboard-sheathed walls with hold-downs was smaller than OSB and fiberboard walls (4.17 in., 4.58 in., and 4.29 in., respectively).

(3) Between the initial and stabilized cycles, the strength of the specimens decreased by an average of 30-40%.

(4) The equivalent viscous damping ratio, EVDR, reached a maximum near the peak load and gradually got smaller. In the elastic range, the EVDR ranged between 10-13%.

(5) The contribution of gypsum to the walls with hold-downs was 2.64 kN (0.59 kips). On a unit length basis, the contribution was 2.2 kN/m (0.15 kip/ft).

(6) Displacement at failure of the specimens decreased when gypsum was included. In the OSB and fiberboard walls, the failure displacement decreased by as much as 12.7mm (0.5 in.).

(7) The addition of gypsum did not change the damping of the system.

(8) There was an increase in the cyclic stiffness due to the addition of gypsum until the displacement exceeded the peak load. Between the peak load and failure, the gypsum no longer contributed to the cyclic stiffness of the walls.

(9) There was a large reduction in the strength of the walls when hold-downs were not included. The walls without hold-downs averaged only one-third of the strength as the walls with hold-downs.

(10) OSB, hardboard, and fiberboard achieved nearly the same peak load when hold-downs were not included (3.18 kN, 0.71 kips). Gypsum was by far the weakest material, only resisting 1.9 kN (0.43 kips).

(11) Fiberboard panels achieved the same maximum load as the OSB and hardboard panels when hold-downs were not included. However, the failure displacement of the fiberboard panels was significantly smaller.

(12) Displacement at failure was also drastically reduced when the walls did not have overturning restraints. The average reduction was 35%, and the hardboard walls failed 50mm (2 in.) earlier than its corresponding test with hold-downs.

(13) The hysteretic energy of the walls without hold-downs was small (0.73 kN-m, 0.2 kip-ft). The hysteretic energy was significantly larger for the walls with hold-downs.

(14) The use of gypsum wallboard on the walls without hold-downs did not contribute any strength or stiffness to the structure and actually decreased the failure displacement of the wall.

(15) The ductility of the walls without hold-downs appeared to decrease due to the reduction of the failure displacement.

Chapter 5: Monotonic and Cyclic Comparisons

5.1 General

The results of shear walls subjected to monotonic and cyclic loading on an individual basis were discussed in Chapters Three and Four. This chapter compares the results of the two loading procedures so that inferences can be made of the cyclic performance compared to the monotonic performance. All of the monotonic tests were conducted according to ASTM E564, and the cyclic tests were conducted following the provisions of ASTM E2126. The walls tested monotonically had hold-downs, while the cyclic tests were performed with and without hold-downs. For comparison purposes, only the cyclic tests with hold-downs can be compared to the monotonic tests.

Due to the number of different sheathing materials tested, and the number of configurations, only two tests were conducted for each configuration. As a result, no statistical information can be provided. Although the materials were of the same grade, variability of material properties will cause no two walls of the same configuration to perform in the exact same manner. Because the performance of light-frame shear walls is primarily influenced by the sheathing-to-framing nail load-slip characteristics (Soltis and Mtenga 1985; Stewart 1987; Dolan 1989), the large number of nails in each wall will allow the results to be similar. For every two-wall set, the individual values from each test are averaged to get the general performance of that particular configuration.

5.2 Load-Displacement Relationship

When comparing cyclic performance to monotonic performance, the critical parameters obtained from the load-displacement relationship are the peak load, elastic stiffness, energy dissipation, ductility, and the displacement at failure. The contribution of gypsum to monotonic and cyclic loading is also investigated. All of the monotonic tests were performed with hold-downs, so no comparisons can be made with the cyclic tests that did not contain hold-downs.

The load-displacement relationship of the monotonic tests and the average initial envelope curve of cyclic tests with hold-downs are shown in Figures 5.1 to 5.7. Although

not included in most comparisons, the initial envelope curve of the cyclic tests without hold-downs is also displayed in Figures 5.1 to 5.7. From the graphs, it is obvious that the performance of the walls is drastically reduced when hold-downs are not included. It is assumed that if monotonic tests were performed without hold-downs, the response would be similar to the effect that was observed during cyclic tests with and without hold-downs.

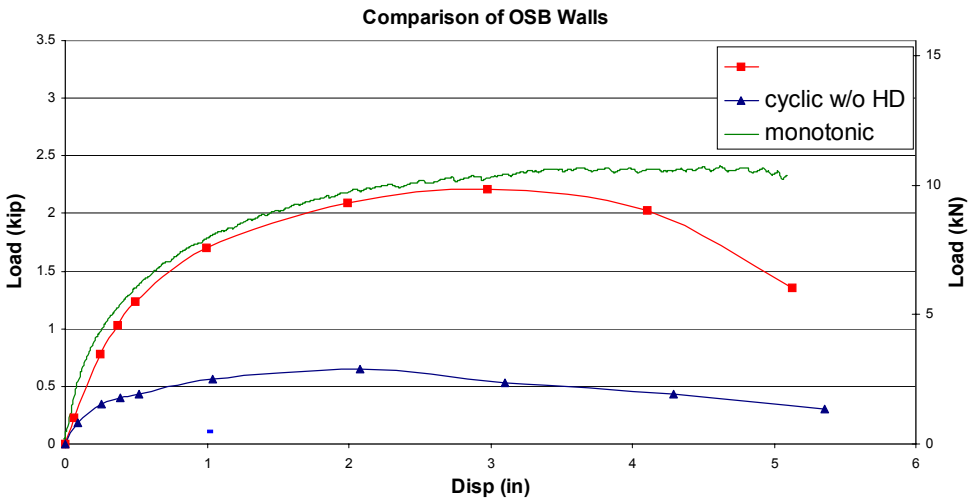

Figure 5.1: All curves from OSB testing

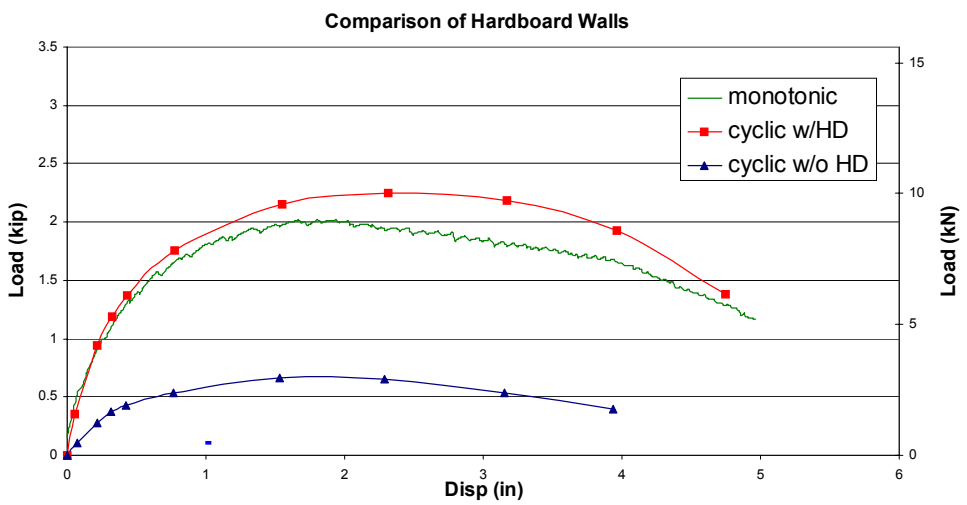

Figure 5.2: All curves from hardboard testing

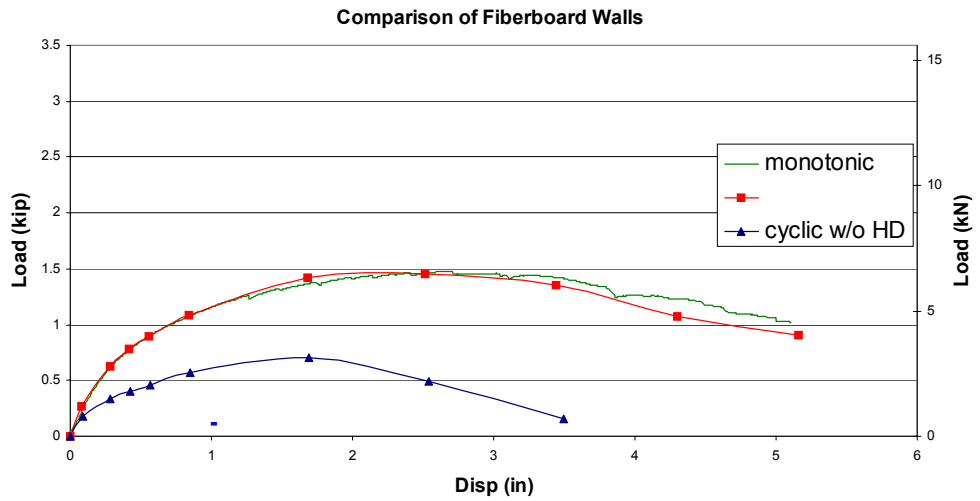

Figure 5.3: All curves from fiberboard testing

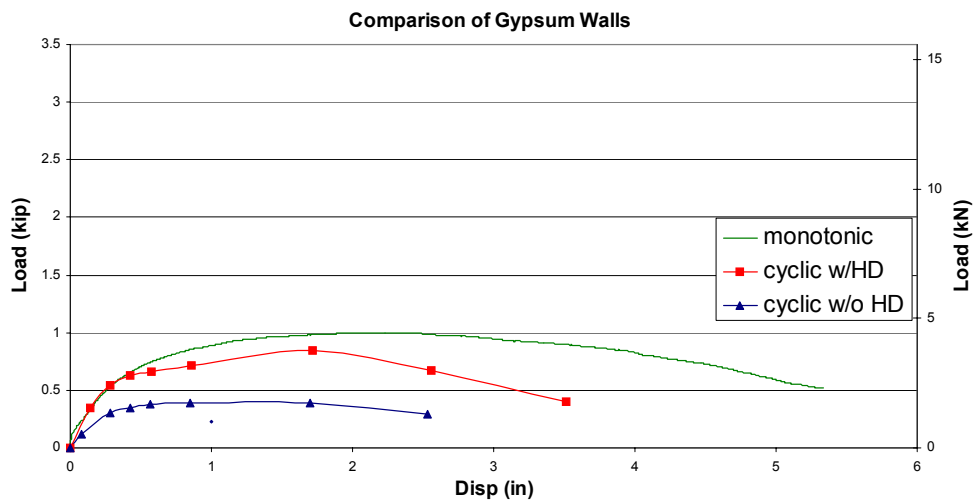

Figure 5.4: All curves from gypsum wallboard testing

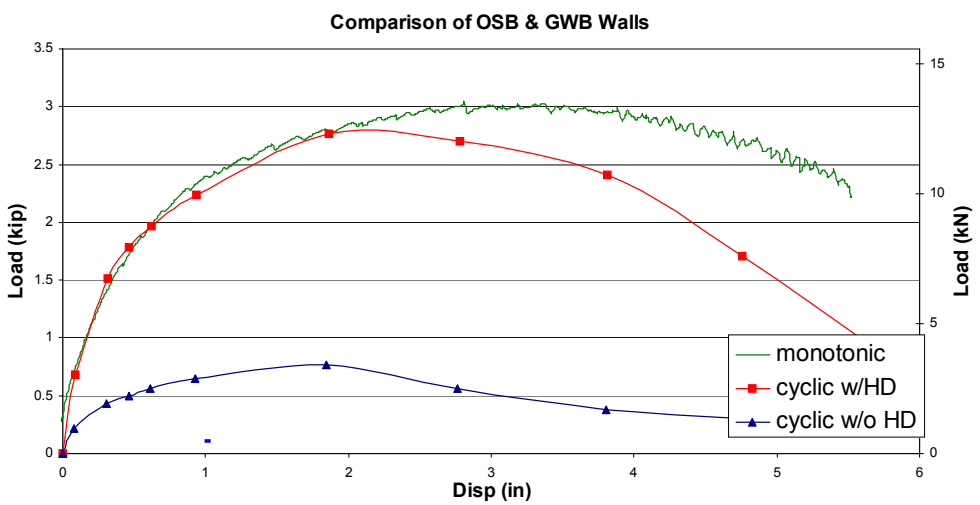

Figure 5.5: All curves from OSB with gypsum testing

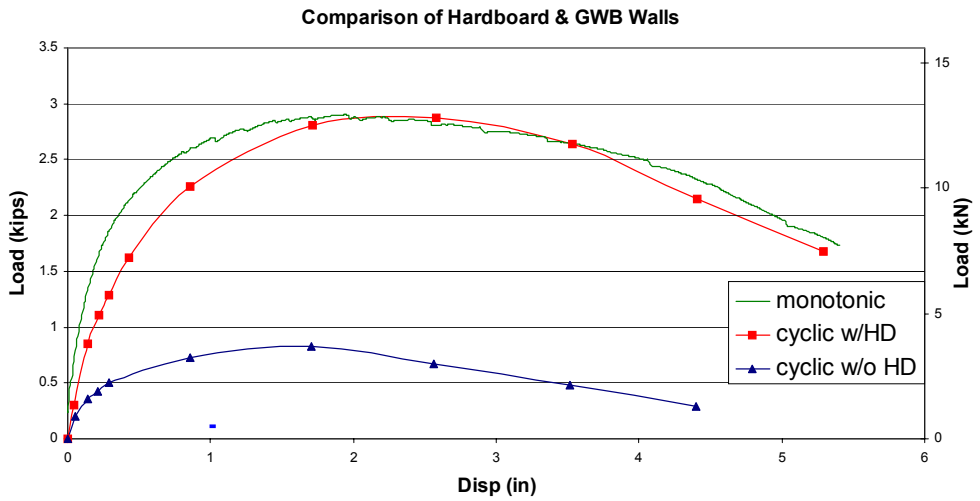

Figure 5.6: All curves from hardboard with gypsum testing

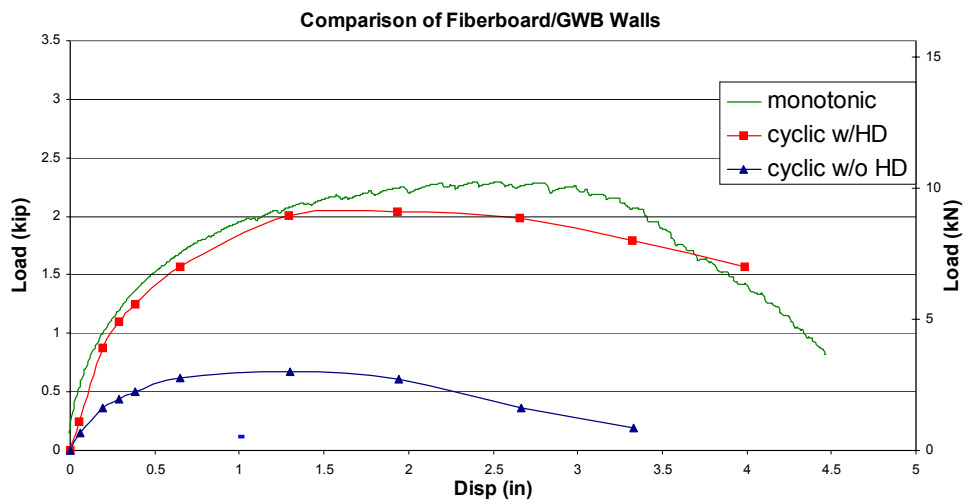

Figure 5.7: All curves from fiberboard with gypsum testing

5.2.1 Peak Load and Displacement

Peak load values for the monotonic and cyclic tests with hold-downs are displayed in Table 5.1, along with the reduction that occurred from testing the wall under cyclic loading. The sheathing material that experienced the largest reduction was gypsum wallboard when tested alone. The gypsum's strength dropped from 4.4 kN (1.0 kips) to 3.7 kN (0.84 kips). That equates to a 16% decrease in strength. OSB-sheathed walls experienced a decrease of 12%, while the fiberboard panels experienced a decrease of only 3.9% in strength. All of the sheathing materials reduced in strength when tested cyclically except for hardboard-sheathed walls, which actually increased in strength. The hardboard wall's strength increased by 0.76 kN (0.17 kips) or 8.2%.

Table 5.1: Peak load comparison of monotonic and cyclic tests

Material	Monotonic		Cyclic with HD		Reduction		
	(kN)	(kips)	(kN)	(kips)	(kN)	(kips)	%
OSB	11.2	2.51	9.8	2.21	1.33	0.3	12.0
Hardboard	9.3	2.08	10.0	2.25	-0.76	-0.17	-8.2
Fiberboard	6.8	1.52	6.5	1.46	0.27	0.06	3.9
Gypsum	4.4	1.00	3.7	0.84	0.71	0.16	16.0
OSB/GWB	13.5	3.04	12.3	2.77	1.20	0.27	8.9
Hardboard/GWB	13.0	2.92	12.8	2.87	0.22	0.05	1.7
Fiberboard/GWB	10.3	2.31	9.2	2.06	1.11	0.25	10.8

One surprising observation was that hardboard reached a higher maximum load than OSB during cyclic loading, but OSB reached a higher maximum load during monotonic testing. One reason may result from the amount of overturning restraint, which allowed the force to be resisted by all the nails along the entire perimeter of the wall. Hardboard panels had a dense nailing schedule of 102mm (4 in.) around the perimeter. The nails never tore through the sheathing, but instead, pulled out of the framing. OSB had a perimeter nailing schedule of 152mm (6 in.), and the nails were observed to tear through the sheathing in some areas. Once a nail tore through the sheathing, that nail was incapable of transferring shear to the sheathing. When a nail pulled out of the framing, it could still transfer shear, but the nail bent and moved through a larger displacement before transferring shear. Although the nails in OSB were larger than the nails in hardboard, the amount of nails and failure mode may have contributed to the increased resistance to cyclic loading.

As shown in Figure 5.1 and 5.5, OSB-sheathed walls typically reached maximum load at a larger displacement when tested under monotonic loading. The continuous fully reversed cycles caused the nails to fatigue and fail at a much faster rate than when subjected to monotonic loading.

When gypsum panels were included on the walls, the additional strength was typically higher during the monotonic tests than the cyclic tests. The average contribution of gypsum to monotonic loading was 3.2 kN (0.72 kips), while the average contribution during cyclic loading was 2.64 kN (0.59 kips). The addition of gypsum also increased the reduction in strength of the walls when compared to the single-sheathed

walls. The reduction was most notable during the fiberboard tests. With gypsum, the fiberboard-sheathed walls reduced by 1.11 kN (0.25 kips) when subjected to cyclic loading. Without gypsum, the reduction was only 0.27 kN (0.06 kips). The ease of which the sheathing nails could tear through the fiberboard and gypsum may result in reduced strength.

Displacements at the peak load for monotonic and cyclic tests are displayed in Table 5.2. 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 walls sheathed with dissimilar materials on opposite sides of the wall. Displacement at peak load during the hardboard tests actually increased during cyclic loading. The reason can be explained by the fact that the peak load was also higher when tested under cyclic loading. The nail behavior and overturning restraints also contributed to the ability to undergo larger deflections under cyclic loading.

Table 5.2: Displacements at peak loads of monotonic and cyclic tests

Material	Monotonic		Cyclic with HD		Reduction		
	(mm)	(in.)	(mm)	(in.)	(mm)	(in.)	(%)
OSB	97.5	3.84	75.7	2.98	21.8	0.86	22.4
Hardboard	57.4	2.26	58.9	2.32	-1.5	-0.06	-2.7
Fiberboard	86.4	3.4	64.0	2.52	22.4	0.88	25.9
Gypsum	56.9	2.24	43.7	1.72	13.2	0.52	23.2
OSB/GWB	82.0	3.23	53.1	2.09	29.0	1.14	35.3
Hardboard/GWB	50.3	1.98	60.2	2.37	-9.9	-0.39	-19.7
Fiberboard/GWB	66.3	2.61	49.8	1.96	16.5	0.65	24.9

5.2.2 Displacement at Failure

Displacement capacity at failure is an important parameter to investigate, and is not a function of the maximum load. The ability to deform without failing increases the energy dissipating capability and toughness of a structure.

Displacements at failure of all tests are compared in Table 5.3. Every cyclic test experienced a reduction in the failure displacement when compared to its monotonic test. The average reduction in the displacement at failure for all of the tests was 14%. The largest reduction occurred during gypsum tests. The failure displacement decreased by 39mm (1.55 in.), or 37%. During monotonic tests, nails in the gypsum would easily tear through the sheathing, but since the loading was in one direction, the wall could still resist load while undergoing large deformations. During fully reversed cycles, the nails would tear through the sheathing in both directions at a faster rate, which enabled the wall to fail at a lower displacement.

Table 5.3: Displacement at failure comparison of monotonic and cyclic tests

Material	**Monotonic**		**Cyclic with HD**		**REDUCTION**		
	(mm)	**(in.)**	**(mm)**	**(in.)**	**(mm)**	**(in.)**	**(%)**
OSB	142	5.6	116	4.6	25	1.0	18%
Hardboard	117	4.6	106	4.2	10	0.4	9%
Fiberboard	117	4.6	109	4.3	8	0.3	7%
Gypsum	104	4.1	65	2.6	38	1.5	37%
OSB/GWB	128	5.1	103	4.1	25	1.0	20%
Hardboard/GWB	114	4.5	106	4.2	8	0.3	7%
Fiberboard/GWB	93	3.7	96	3.8	-3	-0.1	-3%

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 average reduction was only 9.7 mm (0.38 in.), or 8%. The decrease in failure displacement can be attributed to the nail behavior. During monotonic testing, the nails are pulled in only one direction at a constant rate. During cyclic loading, the nails are subjected to many fully reversed cycles at an increased rate, which produces fatigue.

When gypsum was included on the walls, the reduction in the failure displacement followed the same trend as when the walls were single sheathed. OSB-with-gypsum walls experienced an average reduction of 25mm (1 in.), while the reduction in hardboard-with-gypsum walls was 8mm (0.3 in.). The displacement at

failure of the fiberboard actually increased by a small amount when tested under cyclic loading.

5.2.3 Elastic Stiffness

As defined in chapter two, the elastic stiffness is the slope of the secant line that passes through the origin and the load that is equal to $0.4F_{peak}$. From monotonic tests, elastic stiffness is taken directly from the response curve. Elastic stiffness is calculated using the initial envelope curve when considering cyclic tests. As discussed in Section 3.2.3, there was a wide variation in the elastic stiffness within the individual pair of tests. This was due in part to the initial load that was put on the wall. An initial load was sometimes applied to the wall when it was attached to the test frame and the bolt holes did not match up perfectly. This was actual load on the wall, so it could not be neglected. The initial load caused the elastic stiffness to increase because the wall reached a higher load than a wall without an initial load at a given displacement.

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 5.4, the elastic stiffness increases in some cases and decreases in other cases when subjected to cyclic loading. The addition of gypsum is also unclear as the stiffness sometimes increases and sometimes decreases.

Table 5.4: Elastic stiffness comparison of monotonic and cyclic tests

Material	Monotonic		Cyclic with HD		CHANGE		
	(kN/mm)	(kip/in)	(kN/mm)	(kip	(kN/mm)	(k	(%)
OSB	0.76	4.31	0.57	3.27	0.18	1.05	24
Hardboard	0.73	4.18	0.93	5.27	-0.19	-1.09	-26
Fiberboard	0.38	2.18	0.51	2.91	-0.13	-0.73	-33
Gypsum	0.39	2.22	0.45	2.56	-0.06	-0.34	-15
OSB/GWB	1.01	5.72	1.10	6.28	-0.10	-0.56	-10
Hardboard/GWB	1.96	11.14	0.83	4.74	1.12	6.40	57
Fiberboard/GWB	1.11	6.34	0.94	5.34	0.17	1.00	16

Elastic stiffness of the tests should be similar since the value is calculated at a point equal to forty percent of peak load. At this point, the wall has not yielded and should not matter how many cycles the wall has been subjected to. The values displayed

in Table 5.4 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 with gypsum. 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.

5.2.4 Ductility

Ductility values alone do not provide much insight into the performance of the walls. Ductility is a function of the elastic stiffness, yield displacement, and failure displacement. As discussed before, the elastic stiffness can vary with the amount of initial load, which affects the yield point and in turn the ductility. The ductility ratio used for this study is the failure displacement divided by the yield displacement. Ductility values of the monotonic and cyclic tests are listed in Table 5.5.

Table 5.5: Ductility comparison of monotonic and cyclic tests

Material	**Monotonic**	**Cyclic with HD**	**CHANGE**	
OSB	7.2	7.7	0.5	6%
Hardboard	8.7	10.4	1.7	16%
Fiberboard	7.3	9.4	1.9	20%
Gypsum	10.0	8.9	-1.1	-12%
OSB/GWB	10.3	10.5	0.2	2%
Hardboard/GWB	18.5	7.5	-11	-147%
Fiberboard/GWB	11.0	10.5	-0.5	-5%

Ductility values of the cyclic tests varied by a large amount when compared to monotonic tests. 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. In general, the addition of gypsum did not change the ductility when comparing monotonic to cyclic results. Hardboard-with-gypsum walls were the only walls that experienced a large difference in

ductility. As explained in the previous section, the large initial loads induced during the monotonic tests yielded an unrealistic yield displacement, which increased the ductility.

5.2.5 Energy dissipation

Energy dissipation is important when discussing the performance of a shear wall. The lateral force that is exerted on a structure and transferred to the shear wall produces large amounts of energy that must be absorbed in order to avoid failure. 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 calculated in Table 5.6 was determined from the area under the initial envelope curve. 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 5.6: Energy dissipation comparison of monotonic and cyclic tests

Material	Monotonic		Cyclic with HD		CHANGE		
	(kN-m)	(kip-ft)	(kN-m)	(kip-ft)	(kN-m)	(kip-ft)	(%)
OSB	1.31	0.97	0.96	0.71	0.35	0.26	26
Hardboard	0.95	0.70	0.91	0.67	0.04	0.03	4
Fiberboard	0.66	0.49	0.63	0.46	0.04	0.03	5
Gypsum	0.41	0.30	0.23	0.17	0.17	0.13	42
OSB/GWB	1.49	1.10	1.08	0.80	0.40	0.30	27
Hardboard/GWB	1.24	0.92	1.17	0.86	0.08	0.06	6
Fiberboard/GWB	0.81	0.60	0.75	0.56	0.06	0.04	7

As shown in Table 5.6, 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 nail fatigue and tear out. The response during the two loading conditions were nearly identical until reaching peak load, but quite different after peak load as shown in Figure 5.1.

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 5.2 and 5.3, 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 weak panels during monotonic and cyclic loading. Similar nail behavior allowed the walls to fail in the same manner and at the same displacement.

Gypsum wallboard experienced the largest reduction of energy when tested cyclically (43%). This was not surprising given the ease of which the nails in the gypsum could tear through the sheathing. The actuator displacement moved at a much more rapid speed during the cyclic tests. The increased rate damaged the gypsum sheathing more drastically than the slower, more constant rate experienced during monotonic tests.

When gypsum wallboard was included on the walls, the reduction in the amount of energy dissipation was nearly the same as when the walls were single sheathed. The energy reduction in the OSB with gypsum walls was 27%, while the reduction was 6.5% and 7% during the hardboard-with-gypsum, and fiberboard-with-gypsum tests, respectively. When comparing the loading procedures, the gypsum seems to be the least effective when combined with OSB panels, and the most productive when combined with hardboard panels.

5.3 Summary

A total of 45 walls were tested during this study. All of the walls were 1.2 x 2.4m (4 x 8ft), and to be conservative there were no in-plane gravity loads applied to the walls. This chapter compared the results of the cyclic tests to the monotonic tests. All of the monotonic tests were performed with hold-downs, so the comparisons only include the cyclic tests that were performed with hold-downs. The conclusions drawn from the cyclic tests are:

(1) In general, the performance parameters decreased when the cyclic tests were compared to the monotonic tests. The most drastic reduction was observed during the gypsum-sheathed tests, followed by the OSB-sheathed tests.

(2) Hardboard panels tested under cyclic loading performed similar to the monotonic tests. The peak load during cyclic tests was actually larger than its corresponding monotonic test.

(3) Fiberboard panels also performed in a similar manner when tested under cyclic and monotonic loading.

(4) OSB panels tested cyclically did not perform as well as when tested monotonically. The peak load was reduced by 12%, and the energy dissipation was 27% lower when tested under cyclic loading.

(5) Gypsum panels performed poorly under cyclic loading. The peak load reduction was 16%, the failure displacement decreased by 38mm (1.5 in.), and the amount of energy dissipation was nearly reduced by one-half when subjected to cyclic loading.

(6) The contribution of gypsum was more effective when tested under monotonic loading than cyclic loading.

(7) When comparing the two loading procedures, gypsum appears to be least effective when combined with OSB, and most affective when combined with hardboard panels.

Chapter 6: Summary, Conclusions, and Recommendations

6.1 Summary

A total of forty-five 1.2 x 2.4m (4 x 8ft) shear walls were tested for this study. The purpose of the tests was to determine key performance characteristics of various sheathing materials. Sheathing materials investigated were OSB, hardboard, fiberboard, and gypsum wallboard. All walls were tested using monotonic (one-directional) and cyclic loading procedures. Also of interest was the use of overturning restraints. Half of the cyclic tests were performed with hold-down connectors, and the other half were tested without hold-down connectors. The contribution of gypsum when included on a wall with another sheathing material was studied. Data analysis involved calculation of performance parameters such as capacity, yield strength, failure displacement, elastic and cyclic stiffness, ductility, and energy dissipation.

6.2 Conclusions

6.2.1 Characteristics of Sheathing Materials

In summary, the behavior of the sheathing materials when singly sheathed and tested under monotonic and cyclic loading are listed below:

(1) Based on monotonic testing, OSB panels were the strongest material based on ultimate strength, able to resist 11.16 kN (2.51 kips). Hardboard had an average strength of 9.26 kN (2.08 kips), while fiberboard's strength was 6.75 kN (1.52 kips). As expected, gypsum was the weakest material with an average maximum strength of 4.45 kN (1.00 kips).

(2) During monotonic testing, some of the walls had an obvious failure pattern, while some walls reached their peak load and then maintained a load above $0.8F_{peak}$ through a substantial displacement. It could not be determined from two tests why this occurred.

(3) When subjected to cyclic loading, hardboard-sheathed walls resisted the maximum load of all the sheathing materials, 10.0 kN (2.25 kips). OSB walls

could resist an average maximum load of 9.83 kN (2.21 kips). Fiberboard walls were much weaker and could only resist 6.48 kN (1.46 kips). Gypsum was by far the weakest material, only resisting 3.7 kN (0.84 kips).

(4) Although stronger under cyclic loading, the displacement at failure of hardboard-sheathed walls with hold-downs was smaller than OSB and fiberboard walls (106mm, 116mm, and 109mm, 4.17 in., 4.58 in., and 4.29 in., respectively).

6.2.2 Testing Procedures

In summary, the comparison of tests subjected to monotonic and cyclic loading are listed below:

(1) In general, the performance indicators decreased when the cyclic tests were compared to the monotonic tests. The most drastic reduction was observed during the gypsum-sheathed tests, followed by the OSB-sheathed tests.

(2) Of all sheathing materials, the cyclic performance of the hardboard panels was the most similar to the monotonic tests. Peak load during cyclic tests was actually larger than its corresponding monotonic test.

(3) Fiberboard panels also performed in a similar manner when tested under cyclic and monotonic loading.

(4) OSB panels tested cyclically did not perform as well as when subjected to monotonic loading. The peak load was reduced by 12%, and the energy dissipation was 27% lower when tested under cyclic loading.

(5) Gypsum panels performed poorly under cyclic loading when compared to monotonic loading. The peak load reduction was 16%, the failure displacement decreased by 38mm (1.5 in.), and the amount of energy dissipation was reduced by nearly one-half when subjected to cyclic loading.

6.2.3 Contribution of Gypsum Wallboard

In summary, the contribution of gypsum when sheathed with a dissimilar material on the opposite side of the wall subjected to monotonic and cyclic loading are listed below:

(1) The contribution of gypsum wallboard to monotonic tests was shown to increase the overall strength, elastic stiffness, and energy dissipation of the structure. The average additional strength provided when gypsum was included was 3.2 kN (0.72 kips). The strength of a wall sheathed only with gypsum was 4.45 kN (1.00 kips). Therefore, gypsum should be considered to supply a substantial amount of shear resistance when subjected to monotonic loading, but is not linearly additive.

(2) The contribution of gypsum to the walls tested cyclically with hold-downs was 2.64 kN (0.59 kips). On a unit length basis, the contribution was 2.2 kN/m (0.15 kip/ft). However, the failure displacement of the specimens decreased when gypsum was included. In the OSB and fiberboard walls, the failure displacement decreased by as much as 12.7mm (0.5 in.).

(3) There was an increase in the cyclic stiffness until the displacement exceeded the peak load. Between the peak load and failure, gypsum no longer contributed to the cyclic stiffness of the walls. In general, after reaching the maximum load, gypsum did not contribute to the performance of the walls.

(4) The contribution of gypsum to walls tested cyclically without hold-downs was for all practical purposes zero. There was no significant addition to strength or stiffness. The displacement at failure decreased when gypsum was included on the walls.

6.2.4 Effects of Using Hold-downs

In summary, the effects of using overturning restraints in the form of hold-downs connectors are listed below:

(1) There was a large reduction in the strength of the walls when hold-downs were not included. The walls without hold-downs averaged only one-third of the strength as the walls with hold-downs. The failure displacement was also drastically reduced with an average reduction of 35%.

(2) As opposed to walls with hold-downs, the use of gypsum wallboard on the walls without hold-downs did not contribute any strength or stiffness to the structure and actually decreased the failure displacement of the wall.

(3) As opposed to when hold-downs were used, fiberboard panels achieved the same maximum load as the OSB and hardboard panels when hold-downs were not included. However, the displacement at failure of the fiberboard panels was significantly smaller.

6.3 Limitations

Throughout this study, there were several parameters held constant that would vary within actual construction practice. Variations within sheathing materials exist. Although there are minimum standards for the production of OSB, hardboard, fiberboard, and gypsum, all manufacturers produce a slightly different product in terms of the quality. Only one type, from one manufacturer was tested for each sheathing material in this study. Therefore, the same sheathing panel developed by a different manufacturer may perform slightly different.

For each sheathing material, only a single industry nail schedule was used. Slight variations in nailing schedule can exist between manufacturers. For this study, the minimum nail schedule recommended by the supplier of the sheathing material was used. The gypsum material was tested using galvanized roofing nails, however, screws are also commonly used to attach gypsum to the framing, which would alter the performance of the gypsum-sheathed walls.

The application of in-plane gravity loads (dead load) would also have an effect on the performance of shear walls. To give conservative results, no gravity loads were applied at the top of the walls to simulate the dead loads that would occur in a typical shear wall. The application of dead loads would reduce the amount of overturning moment caused by the racking force, whereby reducing the uplift and increasing the strength of the wall. This has the most drastic effect on the walls without hold-downs, which only utilized the sheathing nails along the bottom plate to keep the bottom plate from separating from the rest of the wall.

Time effects were not considered for this study. The walls built and stored for three to fourteen days before being tested. Due to the short amount of storage time, the relaxation effects were reduced, and corrosion and other time dependant effects could not accounted for.

6.4 Recommendations

A modification needs to be made to the cyclic testing procedure, ASTM E2126. According to the sequential phased displacement loading procedure (Method A), it is possible to have phases where the initial amplitude does not increase. If the ductility factor is less than 20, then the 5th phase in the loading protocol will be less than the previous phase. Incorporating decreasing displacement levels does not appear to provide any beneficial information into the performance of the wall.

Although the loads at the critical points were in close agreement, their corresponding displacements exhibited large variations. This may be due to the nature of the materials, but one way to eliminate the discrepancy is to ensure that no initial load is applied to the walls. In addition, if accurate displacement values need to be determined, then numerous tests need to be performed. The two tests that were performed for each scenario during this study were not enough to provide statistical information.

One objective of this study was to provide baseline information of hardboard and fiberboard sheathing panels. Therefore, no design values, R-factors, etc., were provided. More tests should be conducted using these sheathing materials to determine such values.

It is also recommended that further tests be conducted to study the sheathing-to-framing connection of hardboard panels. The nails used in this study always pulled out of the framing and never damaged the hardboard panels. Perhaps using a nail that would not pull out as easily, along with a different nail spacing, would allow better utilization of the hardboard material. Also, the use of shorter nails in all materials (i.e., a maximum penetration) would minimize the effects of cyclic loading and maintain structural toughness. The nail size and nail schedule is compromise of strength and ductility. A shear wall needs to resist a substantial amount of strength, but also be ductile in order to resist the ground motions caused by earthquakes.

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