AN ABSTRACT OF THE THESIS OF

Sarah M. Bultena for the degree of Master of Science in Wood Science and Civil Engineering presented on June 16, 2006.

Title: Hybridized Framing to Modify Load Paths and Enhance Wood Shearwall Performance

Abstract approved:

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Heavy timber framing relies primarily on bracing to withstand lateral loads due to earthquakes and wind events. Bracing configurations in heavy timber framed buildings vary widely and include cross bracing, knee bracing, and other geometries. Many heavy timber frames constructed during colonial American times are still standing, exceeding the expected life of many structures being built today. Limited research has been conducted on the lateral resistance of heavy timber frames and their connections and design aids and procedures are not readily available for engineers to assist in the design of these structures. This method of wood construction has been largely replaced with the development of light-framed wood buildings, which utilize sheathing (typically plywood or OSB) attached to the frame to resist lateral loads.

Today, the primary form of wood construction is light-frame. These structures rely on shearwalls to resist lateral loads. The shearwall consists of 2x4 or 2x6 studs regularly spaced with wood structural panel sheathing attached to the wall frame. This assembly is lightweight and ductile. Extensive research has been conducted on light-frame shearwalls since the 1950's. The effects of construction variables (i.e., fastener schedule,

sheathing thickness and grade, anchorage, and openings) on shearwall performance have been cataloged through numerous studies. Studies have found the sheathing-frame connection, particularly the perimeter connection, is critical to the performance of a shearwall. This connection is typically nailed, although sometimes staples or adhesives are used.

The lateral load path in light-frame shearwalls relies on the sheathing-framing connection. If the load path can be modified then shearwall design can more fully utilize compressive and tensile properties of the wood materials and be less sensitive to the sheathing-framing connection properties. The idea of combining bracing typical of heavy timber framing with techniques used in light-frame construction has not been widely explored by research or analysis. This study investigates the use of bracing in conjunction with light-frame construction (a hybrid framing) to relieve the sheathing nails as the critical load path and enhance the shearwall performance under lateral loading.

A 4 by 8-ft. shearwall was designed consisting of an internal cross brace without intermediate framing studs and a lapped connection at the cross intersection. A 4x4 top-plate was used to improve vertical capacity of the braced shearwall because no intermediate stud was included. Four different types of shearwalls were tested under cyclic loading following the CUREE protocol; a conventional light-framed shearwall, a cross-braced shearwall with no mechanical connection at the corners of the walls, a cross-braced shearwall with plywood gusset plates at the corners of the walls, and a cross-braced shearwall with metal truss plates at the corners of the walls.

The conventional shearwall and the braced shearwall without mechanical connections at the corner of the wall performed similarly - the sheathing-frame connections controlled their performance. Withdrawal of the sheathing nails was the dominate failure mode. The braced shearwalls with the plywood gusset plate and the metal truss plates at the corners exhibited greater ultimate loads, greater initial stiffness and dissipated more energy compared to the conventional shearwall. The modes of failure for these walls were shear failures in the plywood gusset plates and buckling in the metal truss plates. Some failure was observed in the sheathing nails, however, to a lesser degree than observed in the conventional shearwall.

The load path of vertical forces must be addressed in areas where intermediate studs are excluded due to the bracing configuration. Four additional walls were tested under vertical loading; two conventional shearwalls and two cross-braced shearwalls with metal truss plates at the corners. The braced shearwalls proved to adequately resist service level vertical loads similar to those resisted by the conventional shearwall.

Overall, using a hybridized shearwall as a part of light-frame construction appears to be viable option to enhance the lateral performance.

Hybridized Framing to Modify Load Paths and Enhance Wood Shearwall Performance

by
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A THESIS

submitted to

Oregon State University

in partial fulfillment of
the requirements for the
degree of

Master of Science

Presented June 16, 2006
Commencement June 2007

Master of Science thesis of Sarah M. Bultena presented on June 16, 2006.

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ACKNOWLEDGEMENTS

The author expresses sincere appreciation to the following individuals and organizations:

Robert J. Leichti
Milo Clauson
Devco Engineering
Stanley Fastening Systems, L.P.

CONTRIBUTION OF AUTHORS

Robert Leichti assisted with writing the chapters.

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1 Introduction

1.1 Background

Heavy post-beam framing, sometimes referred to as timber framing, is a method of wood construction that has been used since settlement in the New World. Today, light-frame residential wood buildings are the most common type of structure in North America (Folz and Filiatrault 2004a), however, post-beam framing has increased in popularity in recent decades although little is known about the timber-frame industry or its customers (O'Connell and Smith 1999). Timber framing typically transfers lateral loads due to wind and earthquakes through bracing members and traditional wood joints. Bracing configurations in post-beam framing include knee bracing, cross bracing, and bracing in other geometries. Common wood joint connections include the pegged mortise and tendon joint, birdsmouth joint, and dovetail joint. In recent years, structural insulated panels have been used as part of the lateral system for timber frames with structural purpose similar to oriented strand board (OSB) or plywood in light-framed building construction (Carradine et al. 2004). Research on the lateral strength and stiffness of timber frames, with and without structural insulated panels, is limited. Research on the behavior of traditional timber joints (other than bearing only joints) is also relatively limited.

Light-frame wood buildings have become the dominate type of wood construction in North America. The lateral force resisting system in a light-framed wood building consists of horizontal diaphragms, shearwalls, and connections. These elements work together to effectively transfer vertical and lateral loads from the building frame into the foundation. Shearwalls are typically framed with 2x4 or 2x6 lumber and sheathed with plywood or OSB. This creates a shearwall that is lightweight and ductile. Extensive research has been conducted on shearwalls since the 1950's. The strength and stiffness of a shearwall is dependent on the perimeter nail spacing, the sheathing thickness and the anchorage to the foundation. The effect of construction variables (i.e., nail spacing, sheathing thickness, anchorage, and openings) on shearwall performance has been cataloged through numerous studies.

Studies have found the sheathing-frame connection, particularly the perimeter connection, is critical to the performance of a shearwall (APA 1993, APA 2004). This connection is typically nailed, although sometimes staples (ICC-ES 2005) or adhesives are used. Forces in a shearwall are transferred from the top-plate to the sheathing, from the sheathing to the nails, from the nails to the frame, from the frame to the shearwall anchorage and ultimately to the foundation. A common failure of nailed shearwalls under lateral loading is withdrawal, tear-out, pull-though, or fatigue of the sheathing nails (Lattin 2002, Anderson 2005).

Design values assigned to conventional shearwalls by the National Design Specification (NDS) are based on only monotonic tests even though loads induced by earthquakes are not monotonic. Much of the research devoted to wood shearwalls in recent years has to determine the behavior of the shearwalls and connections under cyclic loading. In particular, the Consortium of Universities for Earthquake Engineering (previously the California Universities for Research in Earthquake Engineering Caltech Woodframe, commonly referred to as CUREE) project developed a cyclic loading protocol for woodframe buildings and their connections during an earthquake (Krawinkler et al. 2000). This protocol is intended to be more representative of earthquake loading than other previously developed cyclic loading protocols. The CUREE protocol is characterized by a displacement controlled, fully-reversed cyclic loads with leading and trailing cycles starting at relatively low amplitudes with successive cycles of increasing amplitude. The amplitude of the cycles is scaled to a reference displacement, which is determined from monotonic tests.

Shearwall performance could be enhanced by changing the load path so that the forces are transmitted through the frame rather than only the sheathing nails. This could be accomplished by combining techniques of conventional shearwall construction with heavy timber framing construction, which has not been explored. Limited research has been conducted on using a braced shearwall system that combines bracing techniques from timber framing with the sheathing and framing techniques of conventional shearwalls.

Commercial strongwall products are available (for example, the Simpson Strong Tie products). In addition, narrow wall strongwalls have been the subject of code evaluation reports (APA 2005, ICC-ES 2001). However, these are based on a different load path strategy than considered in this research.

1.2 Objectives

The objective of this project is:

1. To enhance lateral load performance in light-frame wood construction by changing the load path for lateral forces from the sheathing connections to the frame of the wall, relieving the sheathing nails to dissipate energy.
2. To assess the feasibility of using a braced shearwall when subjected to vertical loading.

1.3 Scope

Within this project, the lateral and vertical performance of a sheathed braced wall was examined. Since sheathing-framing connections govern the behavior of conventional shearwalls, providing internal bracing to relieve the sheathing nails could modify the behavior of the shearwall. The first objective was addressed with a study where the lateral performance of a sheathed and braced shearwall was evaluated. Given that connections are vital in shearwall performance several different types of brace connections were studied. The investigation consisted of lateral tests of eight shearwalls, including two conventional shearwalls and six braced shearwalls of various connection details.

The second objective was to assess vertical performance of the braced shearwall in comparison with conventional shearwalls. The braced shearwall lacks the intermediate framing stud, which typically provides the vertical load path in conventional light-frame construction. This objective was to address the vertical performance of a sheathed and braced shearwall. The investigation included development of a vertical testing protocol and tests of four shearwalls, including two conventional shearwalls and two braced shearwalls.

1.4 Thesis Structure and Units

The thesis is written in manuscript format. This style has a comprehensive literature review (Chapter 2), a manuscript that is ready for journal submission (Chapter 3), a conclusion (Chapter 4), a bibliography (Chapter 5), and set of appendices. The appendices provide details of testing, analysis, and results that cannot be used in the journal manuscript. The inevitable result of this format styles is some redundancy across the thesis.

The research project was conducted and reported in inch-pound (in-lb) units. SI units are widely used in academic publications, but the wood construction industry in the U.S. has not widely adopted SI units. It was concluded that the thesis will be more useful in the short term if written in in-lb units.

2 Literature Review

2.1 Heavy Timber Framing

Heavy timber framing, sometimes referred to as post-beam construction, is a method of constructing wood structures using substantially sized timber pieces that function as a frame. Timbers as large as 20 in. by 27 in. were used in heavy timber framing. It was also common to construct with vertical 4x4 studs at spacing of up to 5 ft. on center. This was the most common type of building construction during the first 300 years of settlement in the New World (Lewandoski 1992). Heavy timber framing during this period is referred to as traditional timber framing. During colonial times, iron and steel were too expensive to use in building construction, therefore, traditional timber framing used no metallic fasteners to make connections of the wood pieces (Schmidt et al. 1996). Common methods of providing connections without the use of metallic fasteners included pegged mortise and tendon joints, dovetail joints and birdsmouth joints.

In the 19th century, as sawmilling developed and nail manufacturing improved, the use of traditional timber framing for residential construction faded away. Traditional timber framing for barn construction was common until roughly the middle of the 20th century (Lewandoski 1992). However, in the past few decades, restoration and new construction with post-beam construction has revitalized timber-frame construction. The Timber Framers Guild of North America, which was formed in 1984, has aided the increased popularity by providing membership, technical support, journal dedicated solely to traditional timber framing, and most recently an engineering list serve. Although solid sawn lumber has been the most commonly used wood product for post-beam framing, alternatives such as recycled timbers and glue-laminated timbers are also being used (O’Connell and Smith 1999).

With the increasing popularity of timber framing, there comes a need for standard methods of design and analysis along with the need for further research. Currently there is little information on performance and design methods for timber frames (O'Connell and Smith 1999), especially for traditional connections and lateral resistance. Very little research has been conducted on the behavior of various traditional timber framing joints and even less research has been conducted on the lateral resistance performance of traditional timber frames.

No practical, realistic models have been developed for most of the traditional timber joints and there are no current building code provisions unique to timber framing. The design of the wood members in a timber frame, including beams, columns and braces, are straightforward and follows the design procedures currently in use for wood design, as given in the National Design Specification for Wood Construction (NDS). This is not the case for traditional timber framing connections. The NDS does not address connections using wood fasteners, such as wood pegs, and notched and shaped ends and housings. Forces transferred through bearing are the only wood-wood connections covered in the NDS. For this reason many of today's timber frame buildings are designed using metallic fasteners, creating a hybrid of traditional timber framing and current connection design and materials. A growing number of specialty metal products are available to aid the design engineer in safe, serviceable building design with heavy timbers.

Research has been conducted to increase the understanding of traditional timber frames and their connections. Schmidt et al. (1996) found that the lateral performance of timber frames is sensitive to the joint stiffness, meaning that understanding the joint and bracing behavior is critical in analysis of timber frames. Bulliet et al. (1999) found that timber frame members should be modeled as beam-columns and include the effects of shear. Their research of a knee-braced connection also showed that the beam-column joint should be assumed to carry no moment, the eccentricity of force needs to be included (i.e., wood pegs not at centerline of post) and the effect of contact should be examined where the contact of one member with another may have significant effects. Seo et al. (1999) investigated the response of mortise-and-tenon beam-column joints under static and cyclic loading by conducting tests on traditional timber frames. The ultimate load capacity of the frame was found to be dependent on the joint details (e.g., degree of fixity of the joint, skills of the carpenter). Failure modes of the timber frame were found to be either shear or bending failures of the column mortise branch of a tenon at the joints.

Sensitivity of the behavior of a timber frame structure to the stiffness of its joints has been shown experimentally through several studies. To fully understand the effect of joint stiffness on timber frames, much more work has yet to be done. Modeling of timber joints is a relatively economical and time efficient method of performing sensitivity studies. However, few researches have attempted to capture the behavior of traditional timber frame joints. Brungraber (1985) performed a two-dimensional finite-element analysis on some joint details and proposed a three-dimensional joint model for frame analysis. The proposed model would have required testing many different joint configurations and was later shown to be insensitive to joint behavior by Weaver (1993). Bulleit et al. (1999) modeled three joint configurations (mortise-and-tenon without a shoulder, mortise-and-tenon with a shoulder, and a fork and tongue). The models dealt with the relatively flexible wood peg by modifying the cross sectional area of certain members in the frame model. Bending moment and deflection predictions closely matched those observed through experimental tests. Burnett et al. (2003) studied the effect of end distance in traditional wood-peg mortise-tenon joints.

Understanding the joint behavior is essential to predicting the lateral strength and stiffness of traditional timber frames. Erikson and Schmidt (2001, 2002a, 2002b, 2002c, 2002d) reported the lateral behavior of timber frames having knee braces. Initial lateral tests performed on a one-story, one-bay timber frame found that the initial joint stiffness is low at small displacements, but increases as displacement increases. Tests were also performed on frames that were unsheathed and frames that were sheathed using structural insulated panels (SIPs), which is a common method of enclosing timber frames in North America. Erikson and Schmidt found that sheathing the frames greatly increased the stiffness of the knee braced frames. Like conventional light-framed shearwalls, the lateral stiffness of sheathed timber frame structures is dependent on the characteristics of the panel-to-frame connection. Placing a shim between the panel and framing will significantly reduce the strength and stiffness of the joint, whereas friction between the panel and the timber increases both the strength and stiffness (Erikson and Schmidt 2002c). In 2004, Carradine et al. showed that timber framing structures can benefit from the diaphragm action of structural insulated panels (SIP).

2.2 Light-framed Buildings

Light-framed wood buildings, particularly residential structures, are the most common structures in North America. Recent statistics show that 90 percent of residential building construction is wood and most of that is light-frame (CUREe-Caltech 1998). These structures can range from small-single story structures to large multi-story buildings for multi-family or commercial use. As with any structure that houses people, safety and performance are of the utmost importance for design and construction.

In a light-framed wood building, diaphragms and shearwalls are the main elements designed to resist lateral forces, which are generally caused by earthquakes or wind events. Lateral forces are resisted by the horizontal diaphragm (either at the roof or floor level) then transferred to vertical shearwalls. While the design of the diaphragm and the connection of the diaphragm to the shearwalls is crucial, this report focuses on the performance of the shearwall.

A conventional shearwall is made of four engineered components, sheathing (typically plywood or oriented strand board, OSB), a wood stud frame, nails attaching the sheathing to the wood frame, and anchorage devices (typically anchors bolts and/or hold-downs that attach the wall to the foundation). A shearwall must be designed to have the capability to resist predetermined lateral loads; otherwise, the wall could fail by separating from the foundation, collapsing, or causing excessive architectural damage (White and Dolan 1995).

2.3 Shearwall Design

Shearwall design, like other structural component design, is providing shearwall capacity that exceeds the demand on the shearwall. Seismic provisions for wood structures are built on traditional force-based design procedures. This procedure is chiefly concerned with providing an adequate lateral strength to the structure under a single seismic hazard level associated with life safety (Filiatrault and Folz 2002). The lateral strength required of a shearwall, using the force-based procedure, is based on containing the shearwall response to elastic behavior under a seismic event. The force-based design procedure, which has been used extensively in the past and continues to be used widely today, is simple and economical.

The capacity of a conventional shearwall is determined from tables set forth in the International Building Code, IBC (International Code Council 2003). These tables provide an allowable unit load based on the framing lumber and framing details, thickness and grade of plywood or OSB, and the fastener size and spacing. The Engineered Wood Association (APA, formerly the American Plywood Association) derived the code values from shearwall tests (Gatto and Uang 2003). The tests that are the basis of the IBC shearwall tables consisted of monotonic loading following either ASTM E 72 (ASTM 2005a) or ASTM E 564 (ASTM 2005b). Cyclic loading tests, such as ASTM E 2126, have not been incorporated into the allowable values for wood shearwalls in the current building codes, even though there is concern that the current design values may be unconservative under earthquake loading (Gatto and Uang 2003).

The demands on shearwalls are determined using building code design equations. Earthquake spectral acceleration values are read from maps provided by the USGS (U.S. Geological Survey). These acceleration values along with the soil type, natural period of the building, importance factor and the response modification coefficient, R, are used to determine the base shear of a building. The R-coefficient is representative of the inherent over-strength and global ductility of the lateral-force resisting system. Values for R are given in tables in the IBC; for wood shearwalls with plywood or OSB sheathing, R = 6.5 (IBC 2003, Table 1617.6.2). The base shear is determined from ASCE 7, which is a referenced standard for the IBC. Base shear is (ASCE 7-02 2002, eqn. 9.5.5.2-1 and 9.5.5.2.1-1):

$$V = C_s W$$

where $C_S = \frac{S_{DS}}{R/I}$ and W = effective seismic weight, S_{DS} = design spectral response acceleration, R = response modification factor, and I = importance factor.

For multi-story buildings the base shear is distributed to the different floor levels based on the weight and height of each floor and an exponent, k, which relates to the period of the building (ASCE 7-02 2002, eqn 9.5.5.4-1 and 9.5.5.4-2):

$$F_x = C_{vx} V$$

$$\text{where } C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

and F_x = seismic force at floor x, C_{vx} = vertical distribution factor, V = base shear, w_i and w_x= portion of the total gravity load of the structure assigned to level *i* or *x*, h_i and h_x = height from the base to level *i* or *x*, and n = number of floor levels.

2.4 Nailed-Sheathing Connections

The behavior of conventional shearwalls is strongly influenced by the connections, particularly the sheathing-to-framing connection (Dolan 1989). Nails or staples are used to connect the sheathing to the framing; therefore the characteristics of the fastener connection are very important to the performance of shearwalls. The response of sheathing-nailed connections is generally nonlinear and exhibits strength and stiffness degradation under cyclic loading (Folz and Filiatrault 2004a).

One method of determining the capacity of nailed connections is by using the yield mode equations set forth in Chapter 11 of the NDS (AF&PA 2001). Six yield modes are identified in the NDS (I_m, I_s, II, III_m, III_s, IV). These yield modes are based on fibers crushing in the main and side members (I_m and I_s respectively), fibers crushing in both members (II), the nail yielding forming a plastic hinge at the shear plane along with wood fiber crushing primarily in the main or side member (III_m, III_s respectively), and fastener yielding with multiple plastic hinges (IV) (Tucker et al. 2000). The reference capacity of the single nail connection is taken as the lowest computed value of the six of the yield mode equations. The yield equations are based on the material properties and connection geometry. However, duration of load has also been found to affect the strength of nail connections, that is, capacity of the connections increases with shorter duration loads (Tucker et al. 2000). An increase of 60 percent or 33 percent is allowed by the building codes depending on the controlling yield mode when short term loading is expected due to wind or earthquake. This adjustment is applied to the reference connection capacity.

Another method of determining nail connection capacities is through the use of hysteretic tests and modeling. Fully reversed tests produce hysteretic load-displacement curves. The area contained in the loop represents the energy dissipation of the system (Foilente 1995). To accurately predict the overall system response of a wood shearwall under cyclic loading, the energy dissipation mechanisms of the wood joints must be known. Researchers have found that the hysteresis trace of a wood subsystem (e.g., a wood shearwall) is governed by the hysteretic characteristics of its primary connection (e.g., nailed-sheathing behavior) (Foilente 1995). Only the hysteretic behavior of the nailed-sheathing connection is needed to determine the characteristics of a wood shearwall (Dowrick 1986).

In 2005, Anderson reported the effect of sheathing nail bending-yield strength on shearwall performance as determined by test and computations. Her tests showed that shearwall capacity has some sensitivity to the sheathing nail bending-yield strength. However, considering the performance parameters, such as displacement at maximum load, initial stiffness and energy dissipated, the overall effect of sheathing nail bending-yield strength is not a major factor in wall performance.

2.5 Cyclic Testing

Earthquakes cause cyclic loads on shearwalls. In recent years, research performed on cyclic behavior of shearwalls has increased, but even less research has been conducted on the cyclic response of timber frame lateral-resistance systems. The lack of cyclic testing data for wood shearwalls has kept the design procedures set forth by current building codes based on static tests of shearwalls and the related performance of other building materials with wood when subjected to dynamic loading (White and Dolan 1995). Earthquake testing of shearwalls has been performed by Stewart (1987), Dolan (1989), and Foschi and Filiatrault (1990). Cyclic shearwall tests have been performed by many researchers including Stewart (1987), Gray and Zacher (1988), Dolan (1989), Hanson (1990), Seaders (2004), White (2005), Carroll (2006), Anderson (2005), Lattin (2002), Salenikovich and Dolan (2003a, 2003b), the CUREE project (Krawinkler et al. 2000), and many others. Current full-structure tests are being conducted by NEESWood (2006). A host of refereed and proceeding publications have resulted from these and other projects.

2.5.1 Dynamic and Cyclic Modeling

The high cost and time intensive labor associated with performing dynamic tests on shearwalls dictate the amount of data collected and analysis from these tests. For this reason, it is important to find other, more economical and time saving methods, to research the dynamic response of shearwalls and light-frame buildings that are subjected to wind and seismic forces. Finite-element modeling of conventional shearwalls has been used to reduce the costs of investigating the response of shearwalls to earthquake loading. Finite-element modeling has been reported by Foschi (1977), Falk and Itani (1989), and Kasal and Leichti (1992) to model monotonic shearwall response. Stewart (1987), Filiatrault (1990), and Dolan (1989) developed finite-element models to determine the response of shearwalls to dynamic loads. Many of these early attempts lacked the ability to determine the forces and stresses throughout the different components of the shearwall (White and Dolan 1995). White and Dolan (1995) presented a finite-element model for dynamic shearwall analysis. The wall is composed of beam elements to model the wood frame, plate elements to model the sheathing, nonlinear springs to model to framing-to-sheathing nail connections, and a bilinear spring to model bearing between neighboring sheathing panels. This finite-element program, known as WALSEIZ, is capable of determining displacements at nodes and forces and stresses in each element or plate.

CASHEW, Cyclic Analysis of SHEar Walls, (Elkins and Kim 2003a) is another more recent numerical model developed to predict the load-displacement response and energy dissipation of a shearwall under cyclic loading. Three structural components are defined in this model, rigid framing members, linear-elastic sheathing panels and nonlinear sheathing-framing connectors. The hysteretic model for the sheathing-framing connectors takes into account the pinching behavior and strength and stiffness degradation under cyclic loading (Folz and Filiatrault 2001). Results from CASHEW analyses have been compared to full-scale cyclic shearwall tests and found to accurately predict the nonlinear response of shearwalls under cyclic loading. CASHEW was used by Anderson (2005) for research regarding the response of shearwalls to cyclic loading as affected by sheathing nail bending-yield strength and biodegradation of sheathing. CASHEW was created for an 8 x 8-ft. shearwall with two sheets of sheathing, and may not be appropriate for shearwalls with framing configurations.

Research has also been conducted to model complete light-framed wood buildings. Kasal et al. (2004) modeled L-shaped light-frame structures to explore the effect of building asymmetry on structural response. Recent research includes a two-dimensional planar model for light-framed wood buildings developed by Folz and Filiatrault (2004a, 2004b). Two components were used in the model, a rigid horizontal diaphragm and nonlinear lateral load resisting shearwall elements. The shearwalls were entered into the model as zero-height nonlinear spring elements connected between the diaphragm and foundation. A numerical model of the load-displacement response of the shearwall under cyclic loading, which represented the degrading strength and stiffness behavior of the shearwall, characterized the nonlinear spring element. This model was incorporated into a computer program called SAWS (Seismic Analysis of Woodframe Structures). Results from SAWS were compared to tests results obtained from shake table tests performed on a full-scale two-story woodframe house under the CUREE-Caltech Woodframe Project. SAWS was shown to reasonably predict the seismic response of the shearwall when compared to the full-scale test.

Most recently, Judd and Fonseca (2005) have reported analytical models for sheathing-framing connections characteristic of wood shearwalls and diaphragms. Their model uses a spring pair orientated along initial displacement trajectory. Their intent is to use it in finite element analysis.

2.5.2 Testing Protocol

With the introduction of cyclic shearwall tests came several loading protocols. To better represent the effects of dynamic loading on wood shearwalls during earthquakes, protocols have been developed that incorporate fully-reversed cyclic loading cycles. Four of the more common cyclic loading protocols being used are the sequential phased displacement (SPD) protocol (Porter 1987; Dinehart and Shenton 1998; Ficcadenti et al. 1998), the International Standards Organization (ISO) protocol (ISO 1998), the CUREE-Caltech standard (Krawinkler et al. 2000), and the CUREE-Caltech near-fault protocol (Krawinkler et al. 2000). Each of these loading protocols is defined differently using a reference displacement. The reference displacement can be determined from monotonic shearwall tests and is typically defined as a percentage of the displacement at the ultimate load for the given wall. Some cyclic test protocols have been standardized in ASTM E 2126.

A study by Gatto and Uang (2003) compared these four testing protocols along with monotonic testing protocol with the objective of making a recommendation of the testing protocol that should be standardized for further cyclic testing. Full size shearwalls were tested with the criteria that the failure modes observed in the walls should match the failure observed in actual shearwalls during earthquake events. Gatto and Uang (2003) found that the shearwall behavior induced using the CUREE protocol most closely resembled the observed behavior of shearwalls during earthquakes. The CUREE protocol had strengths similar to those established using monotonic tests, however, there is a reduction in strength due to the reverse cycles that is only detected using cyclic tests.

The CUREE protocol is based on the hysteretic response of woodframe structures and developed exclusively for woodframe testing. This protocol models ordinary ground motions representative of most far-field locations. It is characterized by a pattern of primary cycles followed by trailing cycles that are equal to 75 percent of the previous primary cycle. A primary cycle is defined as a cycle in which a given displacement level is reached for the first time (Gatto and Uang 2003). The reference displacements for the CUREE near-fault protocol and the CUREE protocol are Δ_m and Δ respectively, where Δ_m is the displacement at 80 percent of the maximum load on the degradation portion of the monotonic curve, and Δ is defined as 60 percent of Δ_m.

2.6 Let-in Bracing

Let-in bracing refers to bracing applied to a wall frame such that the brace is set into the edges of studs, flush with the surface. The studs are cut to "let-in" the braces. Commonly 1x4 lumber is used as the bracing member, however, metal straps have also been used.

Research conducted at the Forest Products Laboratory (Anderson 1965) showed that the addition of let-in bracing to horizontally sheathed walls increased the relative rigidity and strength when compared to a wall without let-in bracing. Similarly, let-in bracing increased the relative rigidity and strength of unsheathed walls. The let-in bracing for these wall tests were 1x4 lumber pieces spanning from the top-plate to the bottom-plate at a 45-degree angle, attached with two 8d nails at each crossing stud. Similar findings were reported by Trayer (1947).

Previous building codes required the use of bracing in light-frame construction (this was prior to the common use of structural panels). Iizuka (1975) researched the use of structural panels in lieu of bracing. He found that walls with various types of structural panels had greater shear resistance than walls with bracing. Research was also conducted by Hirashima et al. (1981a, 1981b) had finding similar to Anderson (1965) and Iizuka (1975). This research tested and analyzed seventeen different types of frames with let-in bracing.

2.7 Metal Connector Plates

Metal connector plates have commonly been used in wood construction to provide a mechanical connection for members of wood trusses since the 1950's. The plates are normally made from galvanized or stainless sheet steel. The sheet steel is punched so that teeth are formed perpendicular to the sheet. The teeth of the metal connector plates are pressed into wood to create a connection between two or more pieces of wood. Metal connector plates are commonly used in wood trusses due to the effectiveness to load resistance, ease of construction, adaptability to manufactured production (Zhang et al. 2005) and relatively low cost. One drawback to metal connector plates from a design standpoint is that they are propriety products and design values are not usually published (Hoyle and Woeste 1989). Hoyle and Woeste (1989) have determined typical normal duration load design values are approximately 80-100 psi over the surface area of the connector. ASTM A446-67 grade A is the most common type of steel used for metal connector plates.

Concern has arisen regarding the possibility of reduction in the strength of the wood fiber in tension and bending when the teeth of the metal connector plate is pressed into the wood. McAlister and Faust (1992) showed that metal connector plates performed equivalently in several hardwoods and southern pine. Gupta and Wagner (2002) tested the bending strength of solid sawn lumber and laminated veneer lumber (LVL) in specimens with metal connector plates and specimens without metal connector plates. The study found no statistical difference in the bending strength of the specimens with metal connector plates compared to those without metal connector plates. Clarke et al. (1993) used metal truss plates to repair pallet stringer and showed they could repair broken notched stringers to full bending capacity.

Triche and Suddarth (1988) noted that wood trusses with metal connector plates are indeterminate structures with semi-rigid joints. Although, for typical truss applications, metal connector plates are assumed to only provide tensile resistance and are not considered to provide bending resistance (Wolfe 1990). Wolfe (1990) found that the axial capacity of metal connectors is significantly affected by bending. Therefore the interaction of axial and bending forces must be considered when designing with metal connector plates. Zhang et al. (2001) found that the bending capacity of metal truss plate was significantly affected by gusset-plate thickness, width, and length. Joints subjected to tension and bending typically fail in steel net-section of the truss plate (O'Regan et al.). Various design procedure have been presented to determine the safe capacity of the steel net-section of tension splice joints of metal truss plate joints when subjected to combined tension and bending. Poutanen (1988) provides the background in design of semi-rigid joints with truss plates.

Truss plate assemblies have been evaluated under static and dynamic loading (Emerson and Fridley 1996, DeMelo et al. 1995, and Gupta et al. 2004). The general conclusion by these researchers was that metal truss plates are suitable for use in assemblies subjected to high wind and seismic loadings.

The effect of gaps between the truss plate and the wood members of the connection were shown to detract from truss plate performance (Via et al. 2001, and Kirk et al. 1989). Kirk et al. (1989) reported a plate buckling yield based on gap between members of compression chord splices. Leichti et al. (in press) also reported plate buckling yield that affected compressive capacity in truss webs. Stahl et al. (1996) suggested that plate buckling should be treated as a truss failure load for design purposes.

Basta (2005) showed that compression perpendicular to grain could be large in some connections, which could affect truss plate performance. This is supported by Poutanen (1988) who suggested only 20 to 40 percent of truss plate capacity is available in compression before plate buckling occurs.

Truss-framed assemblies consist of truss members, which are constructed using metal truss plates, for both the roof and floor framing of a structure. Metal truss plates are also used to attach the roof and floor trusses to the exterior wall framing; this connection is unique to truss-framed systems and is not used in conventional light framed construction. Luttrell and Tuomi (1984) conducted tests on truss framed assemblies under vertical loading and compared the measured deflection to predicated deflection. Purdue Plane Structures Analyzer (PPSA) was used to determine the predicted deflection of the system. They found measure deflections to be within 10 percent or less than the levels predicted by the computer program.

2.8 Vertical Forces

Vertical forces are loads that act with gravity. Gravity design of structures usually consists of transferring dead, live and snow loads from the building structure to the foundation. In a conventional light-frame building, the gravity loads are transferred from the roof or floor by joists or beams to the bearing walls of the structure. The bearing walls transfer the gravity loads from the roof or floor to the foundation axially through the studs because the studs act a column members. These bearing walls are sometimes the same walls that are being used as shearwalls to transfer lateral loads.

Many light-frame wood structures have been built using “rule-of-thumb” methods, which have evolved over the years (Gromala and Polensek 1984; AF&PA 2001). The majority of light-frame wood structures have performed satisfactorily with very few failures in light-frame construction that have been a result of gravity loads. For this reason, the research is limited on the vertical transfer of loads in light-frame wood walls. Experimental tests and finite-element modeling have been conducted on the combination of vertical compression and bending of typical sheathed stud walls (Polensek and Atherton 1976, Gromala and Polensek 1984), where the bending in the studs was due to out-of-plane loads, not the in-plane loads typical of shearwalls.

Design of studs for compression and bending assumes that a set of identical studs acts individually as simply supported beam-columns (Polensek and Atherton 1976), however, the actual wall is much more complex. To accurately predict the strength of a wall in combined compression-bending the sheathing must be considered, along with load-sharing of the studs. These studies found that the simplifications used in design seriously underestimate the wall stiffness and strength.

Kasal and Leichti (1992) included this combined bending in finite-element models. Srikanth (1992) also studied the combined bending and compression as he modeled light-frame reliability of structures subjected to wind loads with a focus on load sharing and composite action.

Vertical forces are transferred much differently in heavy timber framing compared to light-frame construction. Large openings, which are one of reasons heavy timber framing might be used over light-frame construction, do not allow for studs to be closely spaced. Beams are used to transfer the vertical forces to column members. The beams and columns can easily be designed for vertical forces using standard design procedures set forth by the NDS (AF&PA 2001) and current building codes. Design of traditional timber framing joints for gravity loads is not specifically addressed by design standards or building codes. Similar to lateral loading, very little research has been conducted on vertical loading of timber frames and their connections. Erikson and Schmidt studied the combination of vertical and lateral loads in a one story, one-bay knee-braced timber frame (Erikson and Schmidt 2001). The addition of gravity loaded to the timber frame under lateral loading decreased the free displacement of the wall and increased the stiffness.

2.9 Literature Integration

The literature shows that shearwall performance could probably be enhanced with a modification of load path. Shearwall ductility can likely be maintained by keeping the sheathing nailing but lateral capacity can be improved if the load path is moved through the framing that must be well connected. Design must accommodate vertical loading in the shearwall.

Lateral and Vertical Performance of Internally Braced Shearwalls

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3 Lateral and Vertical Performance of Internally Braced Shearwalls

3.1 Abstract

In cyclic test of light-frame wood shearwalls, the sheathing-frame nail connection is critical to the lateral load path and often is the weak link in shearwall performance. This study investigates the use of 4x4 cross-bracing in a shearwall to relieve the sheathing-framing connections under lateral and vertical loading. Kiln-dried Douglas-fir lumber and OSB sheathing were used to frame test shearwalls. Four types of walls were tested under cyclic lateral loading; a conventional stud shearwall, a cross-braced shearwall with no mechanical connection at the corners of the walls, a cross-braced shearwall with plywood gusset plates at the corners of the walls, and a cross-braced shearwall with metal truss plates at the corners of the walls. The cross-braced shearwall with plywood gusset plates at the corner had the greatest lateral strength, while the braced shearwalls with the metal truss plates at the corners exhibited the greatest initial stiffness. Two different types of walls were tested under vertical loading; a conventional shearwall and a cross-braced shearwall with metal truss plates at the corners of the walls. The cross-braced shearwall had sufficient strength and stiffness to resist vertical loads at typical service levels.

3.2 Introduction

In a conventional light-frame building, shearwalls comprise the main lateral-force resisting system. Shearwalls consist of sheathing (typically plywood or oriented strand board, OSB), a wood stud frame, nails attaching the sheathing to the wood frame, and anchorage devices (typically anchor bolts and hold-downs that attach the wall to the foundation). A vast body of literature has developed since the 1950’s describing the role construction variables such as nail spacing, exterior sheathing, framing, openings, and hold-downs, etc, on the static and cyclic performance of shearwalls.

Site construction of shearwalls can be hit or miss though. Recently, prefabricated shearwalls have become a popular alternative to site constructed shearwalls (Dunkley 1999). Simplicity in construction, crew motivation, friendlier inspections, and training make these products attractive. The “manufactured shear-resistant products” integrate additional steel components for load path security or use steel to reinforce the load path with wood structural panels.

Wood shearwalls have a reputation for being highly resistant to earthquakes due to the high strength-to-weight ratio of wood and the ductility of connections (Filiatrault 1990). Damage to wood buildings in the Northridge and Loma Prieta earthquakes, however, prompted further investigation of cyclic load effects in shearwalls and connections. The California Universities for Research in Earthquake Engineering Caltech Woodframe (CUREE) project was established to examine the performance of wood-frame buildings and their connections in earthquake prone regions. The CUREE loading protocol was developed by the investigation team (Krawinkler et al. 2000).

Let-in bracing was used in light-frame shearwalls when they were sheathed with planks or fiberboard. However, testing showed that let-in bracing was marginally effective (Anderson 1965). Carroll (2006) provided an extensive review of let-in bracing. The principal short coming of let-in bracing was that it failed to provide an effective load path for lateral forces because it was under fastened and could buckle when subjected to compression loads.

Shearwalls and wall construction have been modeled by many researchers. However, Takino (1977) examined the effect of framing configurations and Polensek (1982) specifically addressed effect of nail modulus. Takino modeled conventional stud walls adding extra studs and horizontal members as well as diagonal and cross-bracing. His results showed additional studs and horizontal bridging did little to improve lateral wall performance, but cross-bracing, in particular, produced a more rigid wall assembly. Polensek’s work showed that walls are strongly influenced by sheathing nail connection modulus. Sutt et al. (2002) provided additional empirical evidence of sheathing nail effect on shearwall capacity and stiffness, while Judd and Fonseca (2005) established models of the sheathing-framing connection to be used in a finite-element analysis of walls.

The weakest link in a structure is predominately the connections (Kalkert and Dolan 1997), particularly the sheathing-framing connection in a wood shearwall, which is most commonly a nailed connection. Generally, to increase the performance of a shearwall the sheathing thickness is increased or the nail spacing is decreased; however, both have limitations; panels can only be manufactured to a certain thickness and nail spacing can only be decreased by a small amount. Changing the load path by providing internal bracing would potentially relieve the nailed sheathing-framing connections, creating a stronger and stiffer shearwall.

3.3 Objectives

In conventional light-frame shearwalls, the sheathing-framing connections have substantive influence on the behavior of the wall (Polensek 1982, Sutt et al. 2002). This project investigates the concept of modifying the load path to relieve the sheathing-framing connections by providing internal cross-bracing in a shearwall. Specific objectives included:

- Evaluate the lateral strength and stiffness of braced shearwalls with several alternates for bracing connectivity.
- Evaluate the feasibility of using an internally braced shearwall in light-frame construction considering the transfer of vertical loads.

3.4 Materials and Methods

3.4.1 Shearwall Configurations and Materials

Eight 4 by 8-ft. shearwalls were constructed for the lateral shearwall tests (Fig. 3.1):

- Two conventional shearwalls with one intermediate stud (Type 1).

- Two cross-braced shearwalls where the brace was snug fit and connected only with sheathing nails (Type 2).
- Two cross-braced shearwalls with plywood gusset plates at each corner forming a three-member joint, brace-stud-plate (Type 3).
- Two cross-braced shearwalls with metal truss plates at each corner forming a three-member joint, brace-stud-plate (Type 4).

Four 4 by 8-ft. shearwalls were constructed for the vertical tests:

- Two conventional Type 1 shearwalls.
- Two braced Type 4 shearwalls.

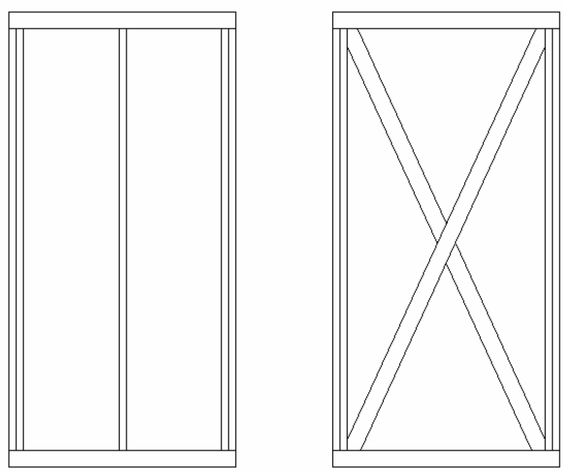


Fig. 3.1. Basic framing layout for conventional Type 1 shearwalls (left) and braced Types 2, 3, and 4 shearwalls (right).

All 2x4 stud members were kiln-dried Douglas-fir Standard & Btr grade. Two 2x4 studs were face nailed and served as end studs at each end of all walls. The Type 1 shearwalls also used one intermediate 2x4 stud at the center of the wall. Top and bottom-plates for all walls were kiln-dried 4x4 Douglas-fir No.2 lumber. Braces for the Type 2, 3, and 4 braced walls were constructed of kiln-dried 4x4 Douglas-fir No. 2 lumber. The braces were cut to form a lap joint at the intersection. No fasteners were applied at the intersection of the braces. The 4 x 8-ft. sheathing panel (7/16-in. Exposure 1 APA Sheathing Rated OSB) was oriented vertically on each shearwall.

For wall construction, 16d (0.162-in. diameter by 3 1/2-in. length) and 8d (0.131-in. diameter by 2 1/2-in. length) common nails were used. Studs were toe-nailed to the top and sill plates. Sheathing was attached to the framing of the Type 1 shearwall with 8d nails at 3-in. on center at the perimeter. Sheathing was attached to the framing of the Types 2, 3, and 4 braced shearwalls with 8d nails at 3-in. on center at the perimeter and along the braces for a vertical length of 18-in. from the top and bottom of the wall. Field nailing was at 12-in. on center for all walls. The hold-downs were installed at each end of all the walls.

The cross-brace of the Type 2 shearwall had no mechanical connection between the brace and the framing. The brace was assumed to be effective only in compression and was connected to the wall frame by only the sheathing nails (Fig. 3.2). The capacity of the Type 2 braced shearwall was assumed to be the same as the conventional Type 1 shearwall. An allowable unit shear of 330 plf was determined in accordance with APA Report 154 (APA 1993) for Type 1 and Type 2 shearwalls.

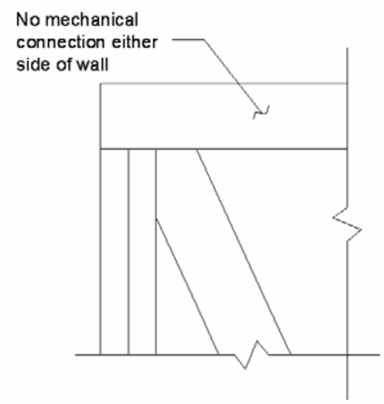
The Type 3 braced shearwall used 9 by 12-in. plywood (1/2-in. CD Exposure 1) gusset plates each side of the wall at the corners, making the braces effective in tension and compression. The braces, top and bottom-plates, and end studs were trimmed 1/2-in. on each side to accommodate the plywood. Thirty-four deformed shank nails (Stanley Bostitch Sheather Plus™, 0.131-in. diameter by 2-1/2 in. length) attached each plywood gusset plate to the framing; 14 nails to the top or bottom-plate, 8 nails to the double end stud, and 12 nails to the brace (Fig. 3.2). The distribution of forces to members of the braced wall under lateral loading was determined by modeling the braced wall in RISA 3D (RISA Technologies 2003). The model assumed pinned boundary conditions at each end of the wall. The member joints were also assumed pinned, however, the distribution of forces in the model was insensitive to the rigidity of the member joints (i.e., pinned or fixed). The distribution of the forces and the allowable shear load of the Sheather Plus™ nails (Leichti 2006) were used to determine expected allowable design capacity of the corner connection. The maximum allowable unit shear was determined to be 584 plf.

The Type 4 braced shearwall used 12 by 12-in. metal truss plates on each side of the wall at the corners, making the braces effective in tension and compression (Fig. 3.2). The metal truss plates were applied to both sides of the wall at each corner with a hydraulic press. The 18-gage metal truss plates had holes 1/8-in. wide by 1/2-in. long spaced at 5/8-in. on center horizontally and 7/8-in. on center vertically. Hoyle and Woeste (1989) suggest the allowable tensile capacity of the metal truss plate is 80 psi. The expected capacity of the Type 4 shearwall was based on the expected buckling capacity of the metal truss plate, the forces and moments in the joint, and the contact area. Metal truss plate buckling was observed by Kirk et al. (1989) who suggested a 20 to 40 percent reduction for buckling. The maximum allowable unit shear was determined to be 595 plf (based on a 40 percent reduction for buckling and bending moment-axial force interaction).

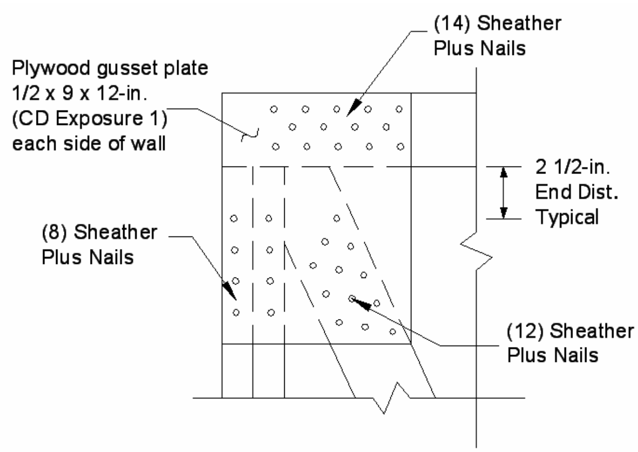
The top-plate in the Type 2, 3, and 4 walls was a nominal 4x4, 48-in. long. Deflection is the controlling design property for this member. If it is designed as a simply-supported beam with two point loads, located at third points and maximum deflection of L/480 (to protect the interior gypsum wall covering from damage, such as cracking), then the deflection limit is 0.10-in. and the corresponding point load is P = 541 lb (each point load).

$$P = \frac{648\delta EI}{23L^3}$$

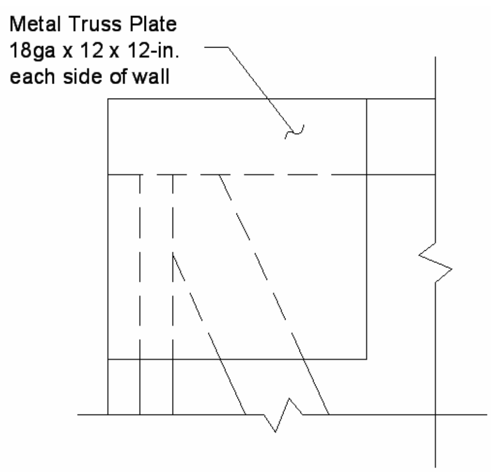
where L = 48-in., E = 1,700,000 psi, I = 12.5-in^4, and δ = 0.1-in.



(a)



(b)



(c)

Fig. 3.2. Corner connections between cross-brace, end stud, and sill plate for (a) Type 2, (b) Type 3, and (c) Type 4 shearwalls.

3.4.2 Lateral Testing

The walls were tested in accordance with E 2126 (ASTM 2002) following the CUREE loading protocol. This loading protocol (Fig. 3.3) is scaled to a reference displacement, which is based on results from monotonic testing. The reference displacement is $0.6\Delta_m$, where Δ_m is the displacement at which the applied load drops for the first time below 80 percent of the maximum load ($0.8F_u$). Rather than test each type of wall monotonically to determine the reference displacement, it was defined as 3-in. based on previous studies conducted by Langlois et al. (2004), Anderson (2005), and Lattin (2002) with the intent to create and observe post peak performance. The tests were run at 0.10 Hz.

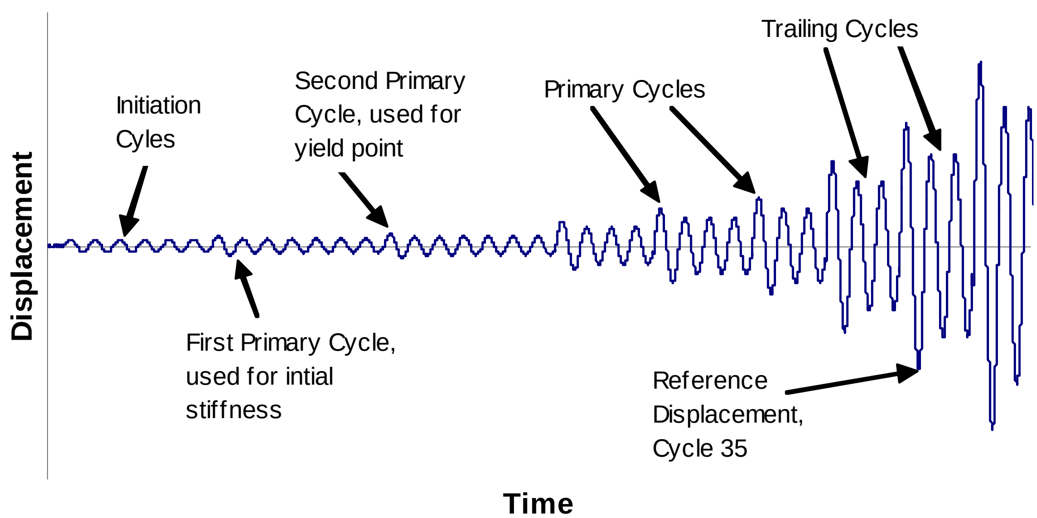


Fig. 3.3. CUREE loading protocol for shearwall tests (Kent 2004).

Hysteretic load-displacement data was produced for each wall tested using the CUREE loading protocol. The backbone curve, which encompasses all of the hysteretic loops, is defined by maximum load in each primary cycle. The hysteretic parameters are extracted directly from the load-displacement data by a computer program, SASHFIT (Elkins and Kim 2003). The four hysteretic parameters that are most useful when depicting shearwall performance are maximum capacity (F_u), displacement at the maximum capacity (Δ_u), initial stiffness (K_0), and energy dissipation (Energy). A fifth parameter is ductility.

K_0 is calculated as the slope of the ascending branch of the hysteresis curve corresponding to the first primary cycle of the CUREE protocol between two and forty percent of the maximum load. The Energy is the sum of the area enclosed by all hysteretic loops. Ductility is Δ_u divided by the displacement at the yield load (Fig. 3.3). The yield load is the maximum load of the second primary cycle of the CUREE protocol.

3.4.3 Vertical Load Testing

No standard testing protocol exists for vertical assessment of light-framed wood walls so an apparatus and protocol were developed to perform vertical tests. A test apparatus consisted of steel wide-flange beams and columns to support the wall and restrict movement (e.g., prevent side-sway) so that the wall would behave as though in a building. Deflection of the steel supports was assumed to be negligible. The apparatus was bolted to the structural floor so the walls were placed horizontally on the floor and supported by wood stickers to minimize frictional resistance.

A hydraulic actuator was used to apply loads to the top of the wall through a steel beam that applied two points of contact to the top of the wall. The points of contact were spaced 16-in. and centered about the centerline of each wall. Each contact point had an adjustable steel bearing plate to apply the loads to the top of the wall.

Three linear variable differential transformers (LVDT) were attached to the wall to assess the movement during vertical loading. The steel beam near the top of the wall limited accessibility to apply an LVDT directly to the top-plate of the walls, therefore, a steel angle was screwed to the top-plate. One LVDT was fixed to the laboratory floor and measured the deflection of the angle. Two additional LVDT's were attached to the sheathing to measure the relative displacement slip between the sheathing and the top and bottom-plates, respectively. The displacements were recorded at 1-second intervals. Fig. 3.4 shows the general set-up of the vertical load test apparatus and the location of the LVDT's for the Type 4 braced wall. The set-up was identical for the conventional Type 1 shearwall. Metal truss plates in the test wall are not shown for clarity.

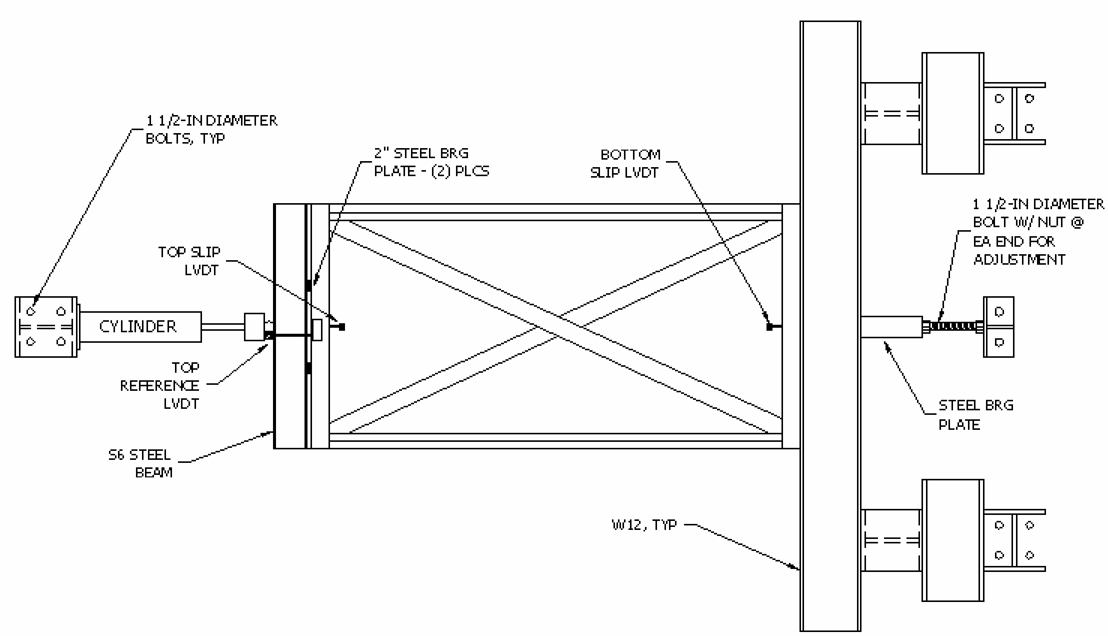


Fig. 3.4. Shearwall vertical load test set-up.

The vertical loading protocol was developed to assess the top-plate deflection because it was determined to be the controlling factor in the assessment of the shearwalls under vertical loading. The objective of the vertical testing was to determine the in-plane bending stiffness of the wall, which was expected to be directly proportional to top-plate deflection. Wood is most commonly designed using allowable stress design, e.g., NDS (AF&PA 2001), which forces wood members to stay within their elastic limits for service level loads. Therefore, the applied loads were kept relatively small. Two different loading positions were used to simulate the different possible configurations of roof and floor joists on a wall. At the roof level of a light-framed wood building the rafter commonly bears on the top-plate and the sheathing, while at the floor level the joist commonly bears only on the top-plate.

A load of approximately 700-lb (350-lb at each point of contact) was initially applied to both the sheathing and top-plate through the steel bearing plate contacting both the top-plate and sheathing (Cycle 1). The load was then released and another load of approximately the same magnitude was applied to the top-plate only by adjusting the steel bearing plate to only contact the top-plate (Cycle 2). Once more the load was released and a third load was applied, again of the same magnitude, to the top-plate and sheathing (Cycle 3). Thus, the loading sequence for the vertical load tests was three cycles of the same magnitude (Fig. 3.5).

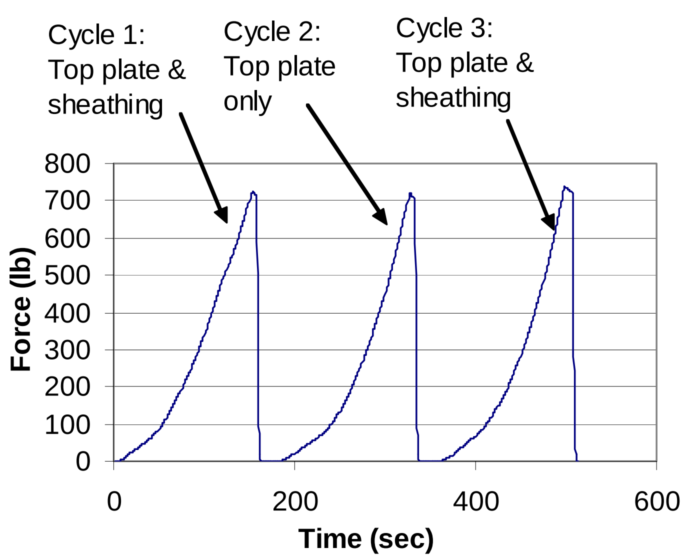


Fig. 3.5. Typical vertical test loading sequence.

3.5 Results and Discussion

3.5.1 Lateral Performance

Load-displacement data were recorded for each wall tested under cyclic loading. Tables 3.1 and 3.2 summarize the performance parameters and unit shears, respectively. Unit shear calculations were based on a factor of safety of 3.5 for the tested shearwalls. Average backbone curves were constructed for each wall type (Fig. 3.6).

The calculated allowable unit shears for the Type 1 and 2 walls were very similarly to the tested unit shears based on a factor of safety of 3.5. The Type 3 shearwall exhibited approximately 16 percent more allowable shear capacity than calculated. Conservatism in the allowable nail shear capacity is a likely source for the difference between the tested and calculated unit shear values. The Type 4 shearwall exhibited approximately 5 percent more allowable shear capacity than predicted by calculations. The Type 3 and 4 shearwalls had higher efficiency values compared to the Type 1 shearwall. The Type 2 shearwall was less efficient than the Type 1, 3 and 4 shearwalls.

Table 3.1. Performance parameters for shearwalls from cyclic lateral tests.

Wall		F_u (lb)	Δ_u (in)	K_0 (lb/in)	Energy (lb·in)	Ductility	Framing Weight (lb)	Efficiency[a]
Type 1	1	4884	3.00	9167	72011	14.4	65.0	75.1
	2	4584	2.03	7883	67628	9.8	65.0	70.5
	Mean	4734	2.52	8525	69819	12.1	65.0	72.8
Type 2	3	4634	2.50	8593	58035	11.8	102.6	45.2
	4	4408	1.85	7078	56332	9.0	102.6	43.0
	Mean	4521	2.17	7836	57184	10.4	102.6	44.1
Type 3	5	9610	2.78	10720	94566	13.5	102.6	93.7
	6	9484	2.62	8807	78872	12.6	102.6	92.4
	Mean	9547	2.70	9764	86719	13.1	102.6	93.1
Type 4	7	8774	1.80	14411	72640	8.5	102.6	85.5
	8	8645	1.92	11586	73196	9.1	102.6	84.3
	Mean	8710	1.86	12999	72918	8.8	102.6	84.9

[a] Efficiency = F_u / Framing Weight

Table 3.2. Unit shear comparisons for shearwalls under lateral loads.

Wall	Tested F_u (lb)	Tested v_u (plf)	Tested v_a (plf)	Estimated v_a (plf)	Percent Difference
Type 1	4734	1183	338	330	2.5%
Type 2	4521	1130	323	330	-2.1%
Type 3	9502	2375	679	584	16.2%
Type 4	8710	2177	622	595	4.6%

where: $v_u = F_u / 4$
$v_a = v_u / 3.5$
estimated v_a = calculated unit allowable shear
percent difference = (v_a - estimated v_a) / estimate v_a x 100

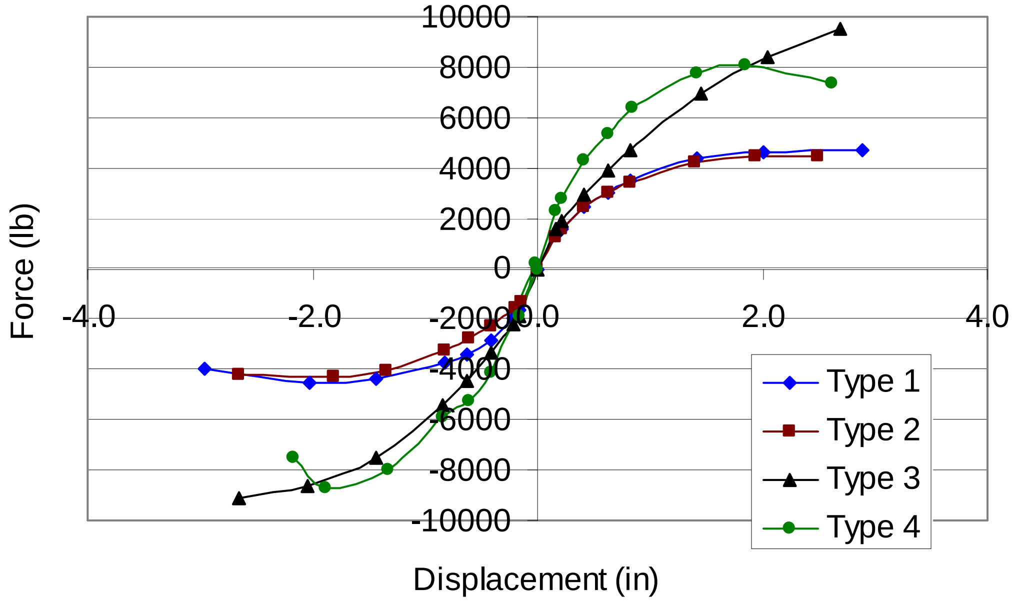


Fig. 3.6. Average backbone curves for shearwalls from cyclic lateral tests.

Statistical analyses were performed using SPSS Version 14.0 (SPSS, Inc. 2005). Analysis of variance showed that there was a significant difference in mean for F_u and Energy (significance level, $\alpha = 0.05$). While the difference is not significant (at significance level, $\alpha = 0.05$), there is strong evidence of a difference among means for initial stiffness, K_0. Ductility and Δ_u did not exhibit differences in means at significance level $\alpha = 0.05$. Table 3.3 summarizes the results of the Tukey multiple comparison tests for the performance parameters, F_u, K_0, and Energy.

The Type 1 and 2 shearwalls performed similarly for all performance parameters; showing no significant differences in means from each other. The Type 3 and 4 shearwalls exhibited greater ultimate loads, F_u, compared to the Type 1 and 2 shearwalls. The Type 3 shearwall also exhibited greater maximum capacity, F_u, compared to the Type 4 shearwall. The Type 3 shearwall dissipated more energy than the Type 2 shearwall, while the Type 4 wall had strong evidence of a greater initial stiffness, K_0, compared to the Type 1 and 2 shearwalls.

Table 3.3. P-values from Tukey multiple comparison tests.

		Type 1	Type 2	Type 3	Type 4
F_u	Type 1	-	0.538	**0.000**	**0.000**
	Type 2	0.538	-	**0.000**	**0.000**
	Type 3	**0.000**	**0.000**	-	**0.016**
	Type 4	**0.000**	**0.000**	**0.016**	-
K_0	Type 1	-	1.000	0.648	**0.064**
	Type 2	1.000	-	0.605	**0.060**
	Type 3	0.648	0.605	-	0.190
	Type 4	**0.064**	**0.060**	0.190	-
Energy	Type 1	-	0.271	0.134	0.946
	Type 2	0.271	-	**0.024**	0.162
	Type 3	0.134	**0.024**	-	0.223
	Type 4	0.946	0.162	0.223	-

The mode of failure for the shearwalls varied, however, Type 1 and Type 2 walls performed similarly during the cyclic testing with respect to the performance parameters. The dominant failure in the conventional Type 1 shearwalls and the Type 2 braced shearwalls was the sheathing pulling away from the frame. Withdrawal of the sheathing nails at the perimeter was the most common failure mode; however, there appeared to be less failure of the sheathing nails at the top-plate in the Type 2 braced shearwalls than in the Type 1 walls. The majority of the nail failures occurred at the largest cycles of the test. In addition to failure of the sheathing nails, the sheathing rotated relative to the wall frame and the top-plate pulled up from the end studs withdrawing the toenails connecting the top-plate to the end studs. The braces in the Type 2 walls, which were only attached to the sheathing, also pulled away from the end studs (Fig. 3.7).

The corner plywood gusset plates on Type 3 walls exhibited failures under cyclic loading (Fig. 3.8) at the bottom of the wall. The plywood gusset plate at the top of the wall also failed along the nails at the end studs. At large cycles, after the plywood gusset plate failed, a small gap occurred between the brace and top-plate and the end studs and the top-plate. Failure of the sheathing-framing nail connections was also present although to a much lesser degree (fewer failed nails and less withdrawal from the frame) than witnessed in Type 1 and Type 2 walls. This trend implies that the load path for the lateral forces was changed from the sheathing-frame connections to the wall frame.

Fig. 3.7. Type 1 shearwall top-plate separation (left) and Type 2 braced shearwall top-plate separation (right).

Fig. 3.8. Type 3 shearwall gusset plate failure.

The metal truss plates on Type 4 walls buckled in compression when the applied lateral force reached approximately 5000 lb. Compression and bending failures of the bottom-plates also occurred in both shearwalls (Fig. 3.9). This failure mode was observed by Leichti et al. (in press) when testing truss webs with metal truss plates at the boundaries and was proposed as a design limit state by Stahl et al. (1996). Poutanen (1988) suggested a 20 to 40 percent reduction in plate capacity for consideration of plate buckling. Plate washers for the anchor bolts embedded into the wood bottom-plate, crushing the wood member and permanently deforming the washers due to bending. Additionally, one Type 4 wall experienced a hold-down failure (Fig. 3.9). There was no indication of separation of the top-plate from the end studs or the brace from the top-plate or end studs. Similar to the Type 3 shearwalls, there was failure of the sheathing-framing nail connections, although to a much lesser degree than witnessed in Type 1 and Type 2 walls, implying that the load path for the lateral forces was changed from the sheathing-frame connections to the wall frame.

A lateral deflection equation can be derived for cross-braced shearwalls like the Type 3 and Type 4 shearwalls. The deflection equation would have the same components as described by Breyer et al. (2003), e.g. deflection contributions from bending, shear, nail slip, and anchorage slip and rotation. However, the shear part of the deflection is reduced by the semi-rigid corner connections that create a frame-like behavior.

Fig. 3.9. Failure of the hold-down and buckling of the metal truss plate (left) and bottom-plate failure (right).

3.5.2 Vertical Performance

Load-displacement data were recorded for each of the four walls tested under the vertical loading sequence. The definitions of wall movements under vertical loading are:

- Bottom slip displacement – the movement between the OSB sheathing and the bottom-plate.
- Top slip displacement – the movement between the OSB sheathing and the top-plate.
- Top reference displacement – the top-plate displacement relative to a stationary point.

Table 3.4 shows the maximum displacements recorded at each of the displacement measurement points. The maximum displacement recorded at the bottom of the wall was approximately 0.002-in. and was neglected in the analysis.

Table 3.4. Maximum displacements for vertical load sequence.

		Maximum Displacement (in.)		
Wall	n	Bottom Slip	Top Slip	Top Reference
Type 1	2	0.0016	0.1000	0.3569
Type 4	1[a]	0.0004	0.0906	0.2062

[a] data for one wall was censored due to deviant test conditions

Some problems were experienced with the test apparatus when testing one of the Type 4 walls. As a result, the data for that wall was removed from the data set. The mean displacements measured in the Type 1 wall were greater than the measurements in the Type 4 wall. The implications of this trend is that the top-plate in the Type 4 wall is stiffer than the top-plate in the Type 1 wall.

Load displacement plots for the Type 1 walls showed two distinct regions of slope (stiffness). This bi-linear load-displacement behavior was characterized by an initial stiffness and a structural stiffness (Table 3.5). The structural stiffness was differentiated by a higher secondary slope. Nuisances of the testing set-up and construction of the walls likely caused the lower initial stiffness. The Type 4 walls appeared to have a constant slope throughout vertical load testing (Table 3.5). The stiffness was determined by calculating the slope from the load deflection curves for each load cycle using the top reference displacement data.

Table 3.5. Wall stiffness (lb/in) from the vertical tests: the cycle number refers to the cycle numbers in Fig. 3.5.

		Cycle 1		Cycle 2		Cycle 3		Average
Wall	n	Initial Stiffness	Structural Stiffness	Initial Stiffness	Structural Stiffness	Initial Stiffness	Structural Stiffness	Structural Stiffness
Type 1	2	1029	3427	972	4382	959	4363	4057
Type 4	1	N/A	4824	N/A	3879	N/A	5256	4653

Little difference existed in the slope (stiffness) of the curves for the three load cycles applied to the Type 1 walls. Loading the top-plate versus loading the top-plate and sheathing appeared to have little effect on the stiffness of the wall. This suggests that the center stud is the controlling factor in the deflection of the top-plate for the conventional Type 1 wall. Type 4 walls experience a lower stiffness when subjected to Cycle 2 loading, which loaded only the top-plate. This suggests that sheathing-framing nails influence the deflection of the top-plate for the Type 4 wall.

The slip measured between the top-plate and the sheathing was much larger than the slip measured between the bottom-plate and the sheathing for all the walls (Table 3.4). This suggests that some of the vertical force is being transferred into the sheathing through the nails on the top perimeter. Since less slip occurs between the sheathing and the wall frame at the bottom of the wall some of the force must be transferred into the end studs as bearing reactions. The forces that are transmitted into the sheathing are partly transmitted to the edge perimeter nails and part of the deformation and reaction is lost to elastic compression in the sheathing at every nail in the perimeter.

The Type 1 and Type 4 walls experienced vertical deflections in excess of L/480 (0.1-in.). All walls experienced permanent displacement after the release of Cycle 1. This was represented by Cycle 2 and 3 not originating at zero displacement in the load-displacement curves. The Type 1 walls had permanent displacements of 0.02 to 0.04-in. after the release of Cycle 1. The Type 4 wall had a permanent displacement of 0.02-in. after the release of Cycle 1. This permanent displacement could have been a product of the test apparatus and wall construction. If the walls were not completely bearing on the continuous steel beam at the bottom of the wall or a gap existed between members of the wall frame, a permanent displacement would occur after Cycle 1. This could partially explain the deflections in excess of 0.1-in. Further research and testing is required to fully understand the vertical transfer of forces through the braced shearwall.

3.6 Conclusions

A major concern with the use of a braced shearwall is constructability and cost. The braced frame with metal truss plates is a more economical wall compared to the braced shearwall with plywood gusset plates in terms of time for construction. Applying the truss plate took significantly less time than trimming the framing members (both sides) and nailing for the application of the plywood gusset plates.

The braced shearwall was able to relieve the sheathing-frame nailed connections by transferring forces through the cross-brace, as long as mechanical connections were applied to attach the brace to the wall frame. Therefore, the use of braced shearwalls as part of a light-framed building is feasible. Further research is needed to evaluate the expected strength and stiffness for use in actual construction. The braced wall can use the same design approach as used in the derivation of allowable shear values for evaluation reports for wood shearwalls. However, for the braced walls, the allowable shear is based only on the corner connection capacity rather than the sheathing-frame fasteners. A lateral deflection equation has not been addressed.

The vertical load path appears to incorporate the top-plate and sheathing through the sheathing-framing fasteners. Vertical deflection of the top-plate in the cross-braced wall is reduced by the semi-rigid end connections of Type 4 walls and does not exceed that of conventional shearwall design.

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4 Conclusions

This study investigated the feasibility of using a braced shearwall in light-frame wood construction to relieve the sheathing-to-frame nailed connections, which typically dominate the behavior of conventional shearwalls. Lateral and vertical performance of the braced shearwalls was considered in the assessment. It was found that a braced shearwall could relieve the sheathing-to-frame nail connections by transferring lateral forces through the brace. The braced shearwall also has sufficient vertical capacity and stiffness to effectively transfer vertical loads from the top of the wall into the foundation.

The concept was conceived for new construction, but it would also be feasible to implement a hybrid framing strategy in older structures undergoing seismic rehabilitation and upgrading. The method will likely be effective with plank sheathing as well as structural panel sheathing.

4.1 Lateral Testing

The Type 2 braced shearwalls performed similarly to the conventional Type 1 shearwalls considering initial stiffness and ultimate load. A cross-braced shearwall without mechanical connections at the corners to integrate the brace with the frame would not be beneficial over using a conventional shearwall.

The Type 3 braced shearwalls withstood the highest ultimate load and dissipated the most energy, while the Type 4 braced shearwalls had the highest initial stiffness. A major concern with the use of a braced shearwall is constructability and cost. The Type 4 braced shearwall (metal truss plates) is a more economical wall compared to the Type 3 braced shearwall (plywood gusset plates) in terms of time and ease of construction. Applying the truss plate took significantly less time than trimming the framing members (both sides) and nailing for the application of the plywood gusset plates.

The braced shearwall relieved the sheathing-frame nailed connections by transferring forces through the brace, as long as mechanical connections were applied to attach the brace to the wall frame. Therefore, the use of braced shearwalls as part of a light-framed building is feasible. Further research would be needed to evaluate the expected strength and stiffness for use in design and construction.

4.2 Vertical Testing

Little difference existed in the slope (stiffness) of the curves for the three load cycles applied to the Type 1 walls. Loading the top-plate versus loading the top-plate and sheathing appeared to have little effect on the stiffness of the wall. This suggests that the center stud is the controlling factor in the deflection of the top-plate for the conventional Type 1 wall. Type 4 walls experience a lower stiffness when subjected to Cycle 2 loading. This suggests that sheathing-framing nails influence the deflection of the top-plate for the Type 4 wall.

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The Type 1 and Type 4 walls experienced vertical deflections in excess of L/480 (0.1-in.). All walls experienced permanent displacement after the release of Cycle 1. This was represented by Cycle 2 and 3 not originating at zero displacement in the load-displacement curves. The Type 1 walls had permanent displacements of 0.02 to 0.04-in. after the release of Cycle 1. The Type 4 wall had a permanent displacement of 0.02-in. after the release of Cycle 1. This permanent displacement could have been a product of the test apparatus. If the walls were not completely bearing on the continuous steel beam at the bottom of the wall, a permanent displacement would occur. This could partially explain the deflections in excess of 0.1-in. Further research and testing would be required to fully understand the vertical transfer of forces through the braced shearwall.

The vertical load path appears to incorporate the top-plate and sheathing through the sheathing-framing fasteners. Vertical deflection of the top-plate in the cross-braced wall is reduced by the semi-rigid end connections of Type 4 walls and does not exceed that of conventional shearwall design.

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Appendices

Appendix A – Wall Construction

Twelve 4 x 8-ft. shearwalls were constructed for lateral and vertical shearwall tests (Walls A1-A8 for lateral tests and Walls B1-B4 for vertical tests). Four shearwalls (Type 1) were built following conventional light-framed shearwall construction (Figure A1). The remaining eight shearwalls (Type 2, 3, and 4) were constructed similarly to the conventional shearwalls except 4x4 cross braces were used in lieu of the typical intermediate framing studs used in the conventional shearwall (Figure A2). Various construction techniques were used to attach the brace to the wall frame (Type 2, 3 and 4 braced shearwalls).

Fig. A1. Conventional Type 1 shearwall construction.

All 2x4 framing members were kiln-dried Douglas Fir-Larch Standard & Btr grade. Two 2x4 studs were used at each end of all walls. The conventional shearwalls also used one intermediate 2x4 stud at the center of the wall. Top and bottom-plates for all walls were constructed of kiln-dried 4x4 Douglas Fir-Larch No.2 lumber. Braces for the six braced walls kiln-dried 4x4

Douglas Fir-Larch No. 2 lumber. The sheathing panels (7/16-in. APA Sheathing Rated Exposure 1 OSB) were oriented vertically.

Fig. A2. Braced Type 2 shearwall construction.

The specific gravity, moisture content and modulus of elasticity were determined for each piece of lumber used in the wall construction. The pieces of lumber were assigned random numbers prior to construction and were used in the construction process without regard to the assigned number or measured stiffness.

The modulus of elasticity for all 2x4 lumber was determined using the Mertiguard Model 340 Transverse Vibration system. With this test system, one end of the lumber is supported by a load cell at a single contact point while the other end is supported by a pivot support. The load cell sends signals to the computer for processing. The dimensions of the lumber and the distance between the two supports are entered into the computer software program. Vibrations are initiated by gently striking the piece of lumber near midspan. The test system measures and records the weight of the specimen and the vibration energy loss. The measurements, along with the entered span and lumber dimensions are correlated to determine the modulus of elasticity. Width and depth dimensions were entered as nominal dimensions (1.5 in. x 3.5 in.) for all specimens. The use of nominal dimensions was justified by the fact that none of the measured dimensions varied by more than 1.5 percent from the nominal dimensions. Tables A2 and A3 show the modulus of elasticity values for the 2x4 lumber.

The modulus of elasticity of the 4x4 pieces was determined by a simple flexure test. Each specimen of lumber was simply supported at 7-ft. - 5-in. on center with two point loads applied to the specimen at 2-ft. - 8 1/2-in. from each support with a hydraulic actuator (Fig. A3). A small load was then applied to the piece (typically around 200 lb) and the deflection recorded. The load was then increased to around 800 lb and the deflection recorded again (Table A1). The modulus of elasticity (E) was determined by computing the stiffness of the piece from the results of the flexure test (Fig. A4).

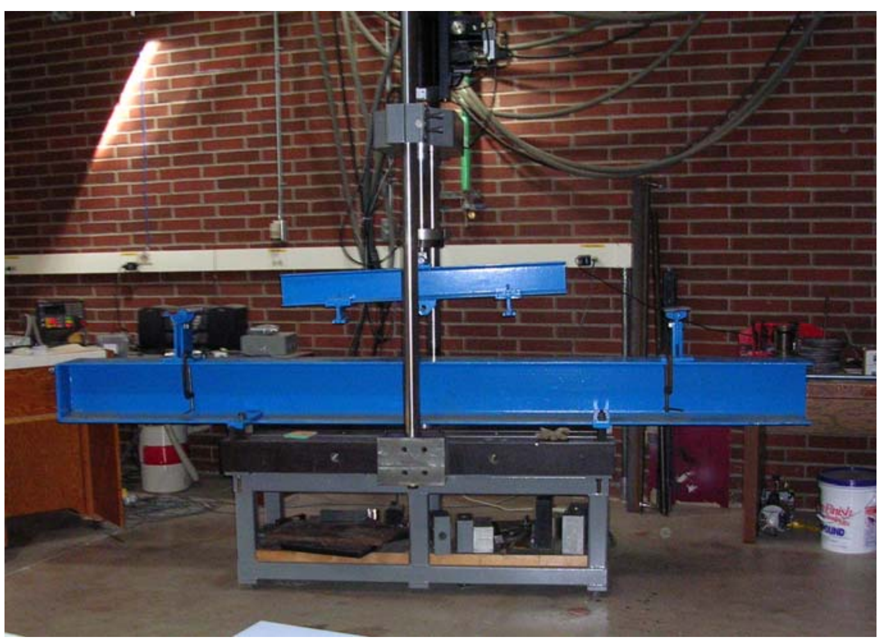
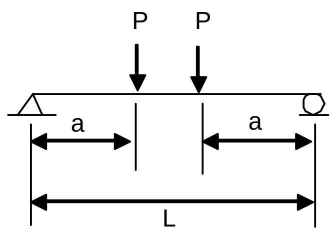


Fig. A3. Test set-up for 4x4 lumber stiffness testing.

Specific gravity was determined from dimensions of the entire piece by measuring the width, depth and length of each piece with calipers and a standard tape measure. Specific gravity was based on the moisture content of the lumber at the time of construction. Moisture content was measured for each piece of lumber using a non-destructive moisture meter prior to the construction of the walls. The values for dimensions, weight, density, specific gravity, modulus of elasticity and moisture content of the 2x4 and 4x4 lumber are summarized in Tables A2 and A3.



$$\Delta = \frac{Pa}{24EI}(3L^2 - 4a^2)$$

$$k = \frac{P}{\Delta}$$

$$E = \frac{ka}{24I}(3L^2 - 4a^2)$$

Fig. A4. Flexure test loading diagram and equations.

Equation variables (L, a, and P) are defined in the diagram; Δ = deflection at L/2; E = modulus of elasticity; k = stiffness; I = moment of inertia.

Table A1. Stiffness data for 4x4 lumber.

Specimen	Load 1 (lb)	Load 2 (lb)	Defl 1 (in)	Defl 2 (in)	Stiffness (lb/in)
1	201.0	799.0	0.130	0.467	887.2
2	200.0	801.0	0.266	0.750	620.9
3	200.0	799.0	0.154	0.610	656.8
4	201.0	500.0	0.348	0.599	595.6
5	202.0	802.0	0.234	0.569	895.5
6	207.0	800.0	0.039	0.360	923.7
7	198.0	800.0	0.125	0.390	1135.8
8	202.0	800.0	0.158	0.540	782.7
9	201.0	802.0	0.171	0.485	957.0
10	200.0	800.0	0.172	0.645	634.2
11	201.0	502.0	0.102	0.314	709.9
12	198.0	800.0	0.131	0.435	990.1
13	199.0	800.0	0.125	0.651	571.3
14	199.0	800.0	0.082	0.482	751.3
15	198.0	799.0	0.065	0.456	768.5
16	201.0	802.0	0.100	0.446	868.5
17	204.0	801.0	0.114	0.505	763.4
18	198.0	798.0	0.144	0.481	890.2
19	200.0	799.0	0.109	0.485	796.5
20	204.0	801.0	0.131	0.550	712.4
21	201.0	800.0	0.130	0.407	1081.2
22	199.0	798.0	0.141	0.457	947.8
23	202.0	800.0	0.117	0.575	652.8
24	200.0	500.0	0.119	0.354	638.3
25	199.0	800.0	0.143	0.512	814.4
26	201.0	801.0	0.176	0.499	928.8
27	201.0	800.0	0.179	0.572	762.1
28	202.0	800.0	0.167	0.453	1045.5
29	201.0	500.0	0.190	0.437	605.3
Average	**200.7**	**758.8**	**0.147**	**0.500**	**806.5**
Std Dev	**2.0**	**105.1**	**0.060**	**0.097**	**155.5**

Table A2. Physical dimensions and characteristics of 2x4 lumber.

Specimen	**Width (in)**	**Depth (in)**	**Length (in)**	**Weight (lb)**	**Density (pcf)**	**SG**	**MOE ($x10^6$ psi)**	**MC (%)**
1	1.4965	3.5155	96.1875	10.188	34.8	0.56	2.55	12.0
2	1.5105	3.5100	96.1250	9.228	31.3	0.50	2.11	12.0
3	1.5020	3.5085	96.0625	8.552	29.2	0.47	1.46	11.0
4	1.5045	3.5170	96.2500	10.038	34.1	0.55	2.13	12.5
5	1.5165	3.5070	96.1875	9.390	31.7	0.51	1.86	13.0
6	1.5145	3.5065	96.1250	9.382	31.8	0.51	2.15	13.0
7	1.5100	3.5260	96.2500	9.390	31.7	0.51	1.64	12.0
8	1.5005	3.4975	96.1250	10.142	34.7	0.56	1.81	14.5
9	1.5060	3.5100	96.1875	8.472	28.8	0.46	1.55	11.0
10	1.4905	3.5050	96.2500	9.736	33.5	0.54	2.27	11.0
11	1.5140	3.5065	96.0625	7.912	26.8	0.43	1.23	10.0
12	1.5065	3.5145	96.1875	8.422	28.6	0.46	1.68	10.0
13	1.5225	3.5490	96.1250	9.112	30.3	0.49	2.14	10.0
14	1.5135	3.5335	96.2500	9.700	32.6	0.52	2.10	11.0
15	1.4990	3.5000	96.1250	8.968	30.7	0.49	1.65	10.0
16	1.5195	3.5175	96.1875	10.312	34.7	0.56	2.18	12.0
17	1.5215	3.5200	96.1875	8.916	29.9	0.48	1.85	10.0
18	1.5125	3.5195	96.1875	8.120	27.4	0.44	1.46	9.5
19	1.5220	3.5165	96.2500	8.096	27.2	0.44	1.58	9.0
20	1.5085	3.4990	96.1875	7.590	25.8	0.41	1.06	10.0
21	1.5010	3.5050	96.0625	8.406	28.7	0.46	1.60	11.0
22	1.5155	3.5090	96.1875	8.498	28.7	0.46	1.84	9.5
23	1.5170	3.5120	96.1250	10.076	34.0	0.54	2.06	14.0
24	1.5080	3.5390	96.1875	8.438	28.4	0.46	1.56	12.5
25	1.5085	3.5080	96.1875	8.526	28.9	0.46	1.71	12.5
26	1.5175	3.5035	96.1875	8.906	30.1	0.48	1.65	12.0
27	1.5115	3.5065	96.0625	10.106	34.3	0.55	1.61	14.5
28	1.5055	3.5025	96.1875	8.566	29.2	0.47	1.64	11.0
29	1.5050	3.4985	96.1875	9.390	32.0	0.51	1.81	15.0
30	1.5090	3.4970	96.1875	8.140	27.7	0.44	1.38	13.0
31	1.5150	3.5105	96.1875	7.980	27.0	0.43	1.53	11.0
32	1.5130	3.5010	96.0625	8.840	30.0	0.48	2.08	12.0
33	1.5130	3.5110	96.2500	9.048	30.6	0.49	1.20	11.0
34	1.5060	3.4915	96.1875	10.522	35.9	0.58	2.11	16.0
35	1.4990	3.5065	96.2500	10.204	34.9	0.56	2.04	15.0
36	1.5180	3.5045	96.2500	8.716	29.4	0.47	1.52	13.5
37	1.4970	3.5055	96.2500	9.176	31.4	0.50	1.94	12.0
38	1.4905	3.5060	96.1875	10.624	36.5	0.59	1.80	14.5
39	1.5000	3.5070	96.2500	11.062	37.8	0.61	2.41	15.0
40	1.4965	3.5235	96.1875	9.558	32.6	0.52	1.60	12.0
41	1.5040	3.5235	96.1875	8.876	30.1	0.48	1.52	13.0
42	1.5080	3.5080	96.1875	10.522	35.7	0.57	1.95	16.0
43	1.5165	3.5065	96.1250	10.440	35.3	0.57	2.51	17.0

Table A2 (Continued). Physical dimensions and characteristics of 2x4 lumber.

Specimen	Width (in)	Depth (in)	Length (in)	Weight (lb)	Density (pcf)	SG	MOE ($x10^6$ psi)	MC (%)
44	1.4960	3.5110	96.1250	9.958	34.1	0.55	2.01	15.0
45	1.4945	3.4960	96.1875	9.232	31.7	0.51	1.10	12.0
46	1.5170	3.5020	96.1875	9.364	31.7	0.51	1.48	11.0
47	1.5025	3.4935	96.1875	9.806	33.6	0.54	2.01	15.0
48	1.5065	3.4935	96.1875	9.992	34.1	0.55	2.36	13.5
49	1.5205	3.4950	96.1875	9.396	31.8	0.51	1.49	15.0
50	1.5000	3.5035	96.1875	10.670	36.5	0.58	2.53	16.0
51	1.5090	3.5150	96.1875	9.062	30.7	0.49	1.35	13.5
52	1.4990	3.5040	96.1250	8.564	29.3	0.47	1.26	12.0
Average	**1.5079**	**3.5092**	**96.1767**	**9.276**	**31.5**	**0.50**	**1.79**	**12.5**
Std Dev	**0.0084**	**0.0112**	**0.0536**	**0.837**	**2.91**	**0.047**	**0.372**	**2.00**

Table A3. Physical dimensions and characteristics of 4x4 lumber.

Specimen	Width (in)	Depth (in)	Length (in)	Weight (lb)	Density (pcf)	*SG*	MOE ($x10^6$ psi)	MC (%)
1	3.5110	3.5405	120.5625	28.788	33.2	0.53	1.87	12.0
2	3.4970	3.4780	120.5625	28.912	34.1	0.55	1.31	12.0
3	3.4615	3.4745	120.4375	24.564	29.3	0.47	1.38	10.0
4	3.4630	3.4735	120.4375	22.992	27.4	0.44	1.25	13.0
5	3.4640	3.4895	120.5625	23.410	27.8	0.44	1.89	11.0
6	3.4965	3.4775	120.5000	30.970	36.5	0.59	1.95	13.0
7	3.4500	3.4545	120.5625	31.350	37.7	0.60	2.39	13.0
8	3.4710	3.4620	120.5625	28.710	34.2	0.55	1.65	12.0
9	3.4700	3.4745	120.5000	28.314	33.7	0.54	2.02	12.5
10	3.4935	3.4710	120.4375	26.510	31.4	0.50	1.34	12.0
11	3.4730	3.4715	120.4375	26.862	32.0	0.51	1.49	11.0
12	3.4900	3.4645	120.5000	32.222	38.2	0.61	2.09	15.0
13	3.4990	3.4730	120.3750	27.634	32.6	0.52	1.20	10.0
14	3.4505	3.4800	120.5000	23.860	28.5	0.46	1.58	10.0
15	3.4665	3.4350	120.4375	33.232	40.0	0.64	1.62	13.0
16	3.4300	3.4640	120.6250	25.752	31.0	0.50	1.83	10.0
17	3.5065	3.5415	96.6250	20.800	30.0	0.48	1.61	12.0
18	3.5745	3.5335	96.5000	22.820	32.4	0.52	1.87	11.5
19	3.5385	3.5750	96.6250	22.790	32.2	0.52	1.68	10.0
20	3.5955	3.5600	96.5625	23.256	32.5	0.52	1.50	22.0
21	3.5365	3.5300	96.7500	25.380	36.3	0.58	2.28	22.0
22	3.5050	3.4700	96.7500	23.914	35.1	0.56	2.00	13.0
23	3.5285	3.5160	96.6250	19.754	28.5	0.46	1.37	10.0
24	3.5230	3.5425	96.5625	22.024	31.6	0.51	1.34	9.5
25	3.5400	3.5285	96.5625	21.562	30.9	0.50	1.71	11.0
26	3.5680	3.5345	96.7500	24.798	35.1	0.56	1.96	12.0
27	3.5065	3.5515	96.6875	19.612	28.1	0.45	1.60	9.0
28	3.5600	3.5555	95.8750	32.044	45.6	0.73	2.20	14.0
29	3.5630	3.5930	96.4375	22.956	32.1	0.51	1.27	10.0
Average	**3.5046**	**3.5040**	**96.5625**	**23.208**	**33.0**	**0.53**	**1.70**	**12.8**
Std Dev	**0.0423**	**0.0421**	**12.1163**	**3.870**	**4.04**	**0.065**	**0.328**	**3.06**

All pieces of lumber were randomly used to construct the test walls. The 4x4 pieces labeled 18-30 were cut into two pieces to use for the top and bottom-plates of the walls, thus some members in the walls have the same number assigned. Due to the small sample size, the random assignment of members imposes a greater probability of experimental error. Figure A5 and Table A5 show the configuration of the constructed walls along with the pieces used for construction of each wall, respectively.

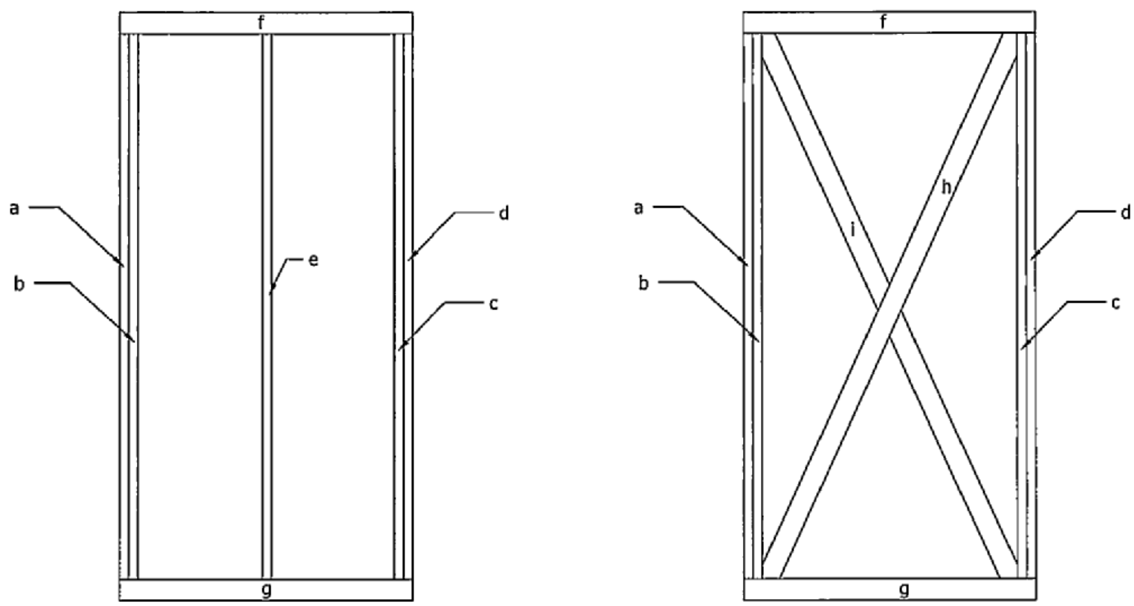


Fig. A5. Wall layout.

Table A4. Wall member layout.

Wall No.	Wall Description	2x4 lumber a	b	c	d	e	4x4 lumber f	g	h	i
A1	Type 1	12	47	31	29	1	18	24	-	-
A2	Type 1	50	38	4	17	45	25	24	-	-
A3	Type 2	42	5	32	46	-	21	19	11	5
A4	Type 2	2	3	21	9	-	22	22	4	6
A5	Type 3	19	35	18	15	-	18	25	1	3
A6	Type 3	49	30	28	27	-	28	17	9	12
A7	Type 4	43	51	52	20	-	21	17	7	8
A8	Type 4	36	48	24	25	-	26	26	10	2
B1	Type 1	41	33	22	16	44	27	29	-	-
B2	Type 1	11	37	14	39	23	29	19	-	-
B3	Type 4	6	7	10	26	-	20	23	13	16
B4	Type 4	13	40	8	34	-	23	20	15	14

For wall construction, 16d (0.162-in. diameter by 3 1/2-in. length) and 8d (0.131-in. diameter by 2 1/2-in. length) common nails were used. The double end studs were nailed together with one 16d at 24-in. on center (per 2003 IBC Table 2304.9.1). Double end studs were attached to the top and bottom-plate with four toe-nailed 8d nails. The intermediate stud on the conventional shearwall was attached to the top and bottom-plate with two toe-nailed 8d nails. Sheathing was attached to the framing of the conventional shearwall with 8d nails at 3-in. on center at the perimeter. Sheathing was attached to the framing of the braced shearwall with 8d nails at 3-in. on center at the perimeter and along the braces for a vertical length of 18-in. from the top and bottom of the wall. Field nailing was at 12-in. on center for all walls. The hold-downs were Simpson PHD5-SDS3® installed per the manufacturer's recommendations at each end of all the walls.

Eight walls were built for the lateral shearwall tests and two of the walls were built as Type 1 conventional shearwalls with studs at 24-in. on center (Wall A1 and A2). The six remaining shearwalls were constructed with a 4x4 internal brace. The brace was trimmed at both ends to fit within the wall frame. Three different connections (Type 2, 3, and 4) of the brace to the framing were constructed (two walls for each type of brace connection).

The Type 2 shearwall brace connection consisted of providing no mechanical connection between the brace and the framing (Walls A3 and A4). The brace was only connected to the wall framing through the sheathing nailing (Fig. A6).

The Type 3 shearwall brace connection (Walls A5 and A6) used 9-in. by 12-in. plywood gusset plates each side of the wall at the corners (Fig. A7). The plywood was 1/2-in. CD Exposure 1. The braces, top and bottom-plates and the end studs were dapped 1/2-in. on each side so that the outer face of the plywood was flush with the face of the connecting members (Fig. A8). In a practical sense, this would allow wall panels, such as gypsum board and exterior sheathing, to be installed without furring strips. In order to maximize the capacity of this connection, deformed shank nails were used to connect the plywood gusset plate to the brace and framing members. The deformed shank nails were Stanley Bostitch Sheather Plus^{TM} nails (0.131-in. diameter by 2 1/2-in. length). A total of 34 Sheather Plus^{TM} nails were used at each plywood gusset plate to attach it to the framing (14 nails to the top or bottom-plate, 8 nails to the double end stud and 12 nails to the brace). The nails were staggered in three rows on each framing member (Fig. A7).

The Type 4 brace connection (Walls A7 and A8) used 12-in. by 12-in. metal truss plates on each side of the wall at the corners (Fig. A9). The metal truss plates were applied to both sides of the wall at each corner with a hydraulic press. Pressure was applied until all teeth of the metal truss were embedded into the wood framing without causing any crushing of the framing. The 18-gage metal truss plates had holes 1/8-in. wide by 1/2-in. long spaced at 5/8-in. on center horizontally and 7/8-in. on center vertically. One tooth was punched from each hole so that there was approximately 540 teeth per square foot.

Four additional walls were constructed for the vertical wall tests. Two of the walls (Wall B1 and B2) were constructed as the Type 1 conventional shearwalls. The final two walls (Wall B3 and B4) were Type 4 braced shearwalls.

Fig. A6. Brace detail without mechanical connection.

Fig. A7. Brace detail with plywood gusset plate.

Fig. A8. Braces cut to accommodate plywood gusset plates.

Fig. A9. Brace detail with metal truss plates.

Appendix B – Lateral Load Tests of Shearwalls

Eight walls were tested laterally under cyclic loading. Two of each type of wall were tested (Type 1, Type 2, Type 3, and Type 4). The walls were tested in accordance with E 2126 (ASTM 2002) following the CUREE loading protocol, with a reference displacement of 3-in. (Fig. B1). Rather than testing each type of wall monotonically, the reference displacement was selected based on previous studies conducted at Oregon State University, as well as the limitations of the hydraulic actuator, and so that post peak behavior would be created.

The maximum stroke of the 150-kip capacity actuator was 10 in. The tests were run at 0.10 Hz. Load was measure with an electronic load cell mounted on the end of the actuator rod. The test-set up is a welded steel fixture, heavily bolted to the structural floor. The test wall was bolted to steel top and bottom-plates using two 5/8-in. bolts at the bottom in addition to the hold-downs and four 1/2-in. bolts at the top. The washers used on the bottom bolts were 2-1/2-in. square ¼-in. thick steel plates. The load was applied at the top of the wall using the hydraulic actuator.

Table B1 describes the wall labeling system used for the lateral tests. Fig. B1-B8 are the hysteresis curves for the walls under the CUREE loading protocol and Table B2 summarizes the test results.

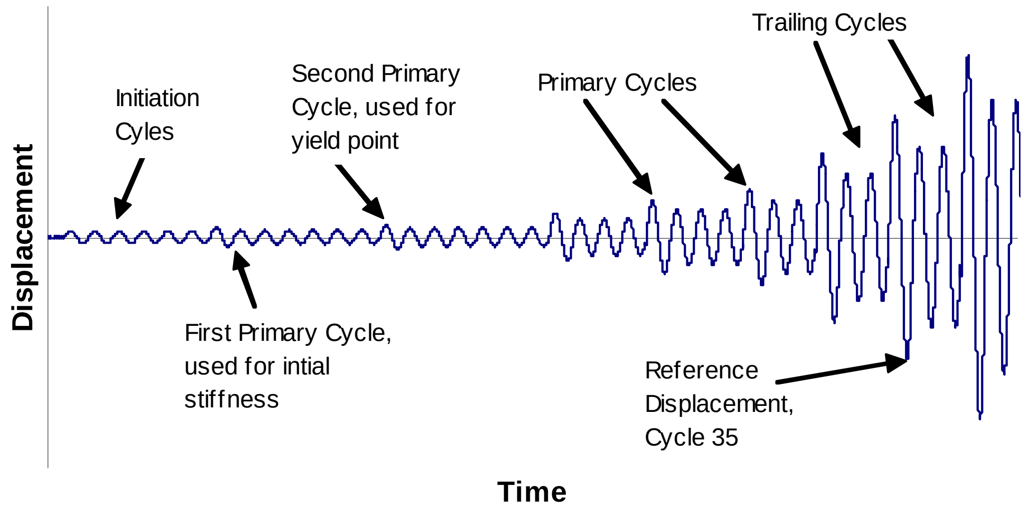


Fig. B1. CUREE loading protocol for shearwall tests.

Table B1. Wall descriptions.

Wall No.	Wall Description
A1	Type 1 - Conventional
A2	Type 1 - Conventional
A3	Type 2 - Brace: no mechanical connection
A4	Type 2 - Brace: no mechanical connection
A5	Type 3 - Brace: plywood gusset plate
A6	Type 3 - Brace: plywood gusset plate
A7	Type 4 - Brace: metal truss plate
A8	Type 4 - Brace: metal truss plate

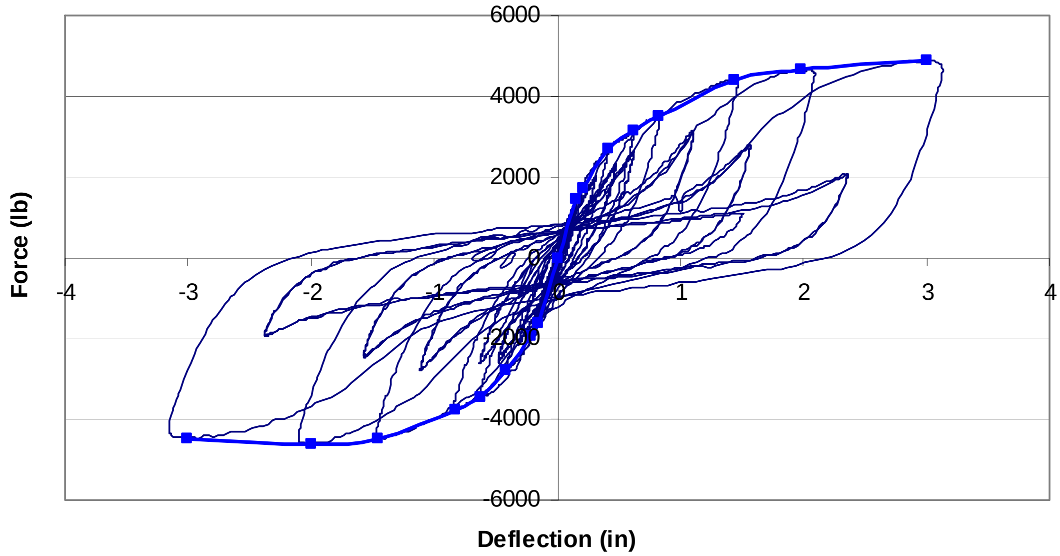


Fig. B2. Wall A1 hysteresis curve.

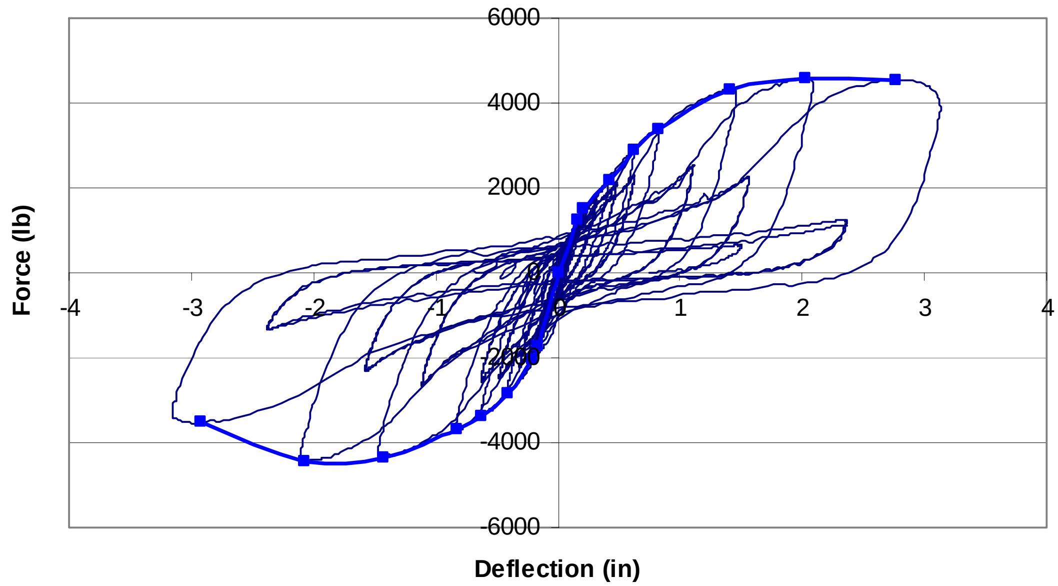


Fig. B3. Wall A2 hysteresis curve.

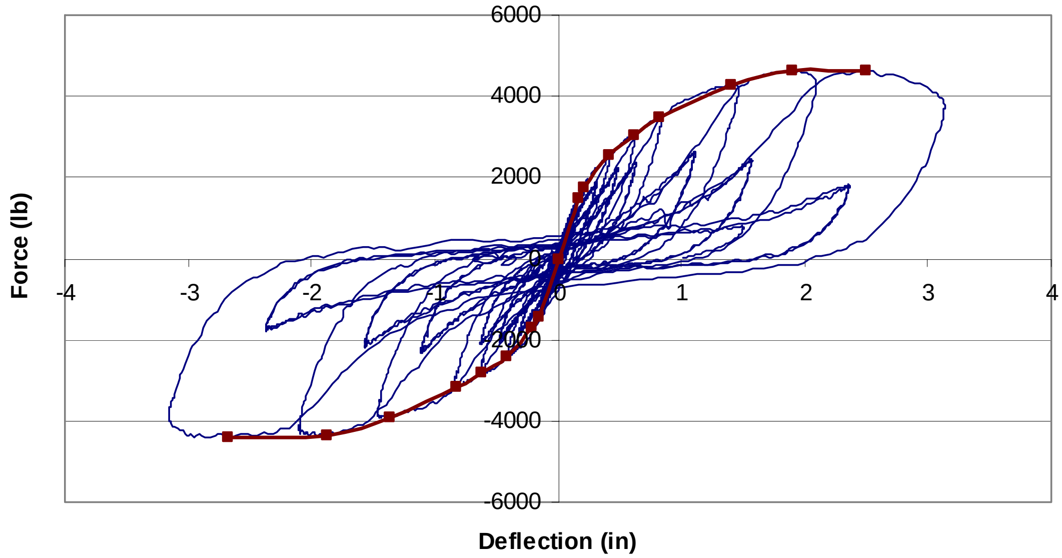


Fig. B4. Wall A3 hysteresis curve.

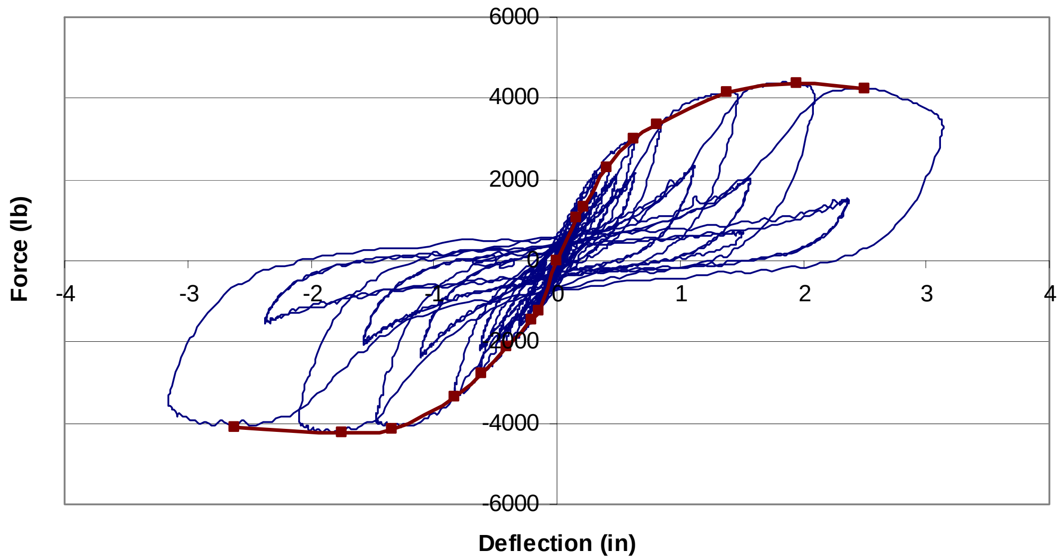


Fig. B5. Wall A4 hysteresis curve.

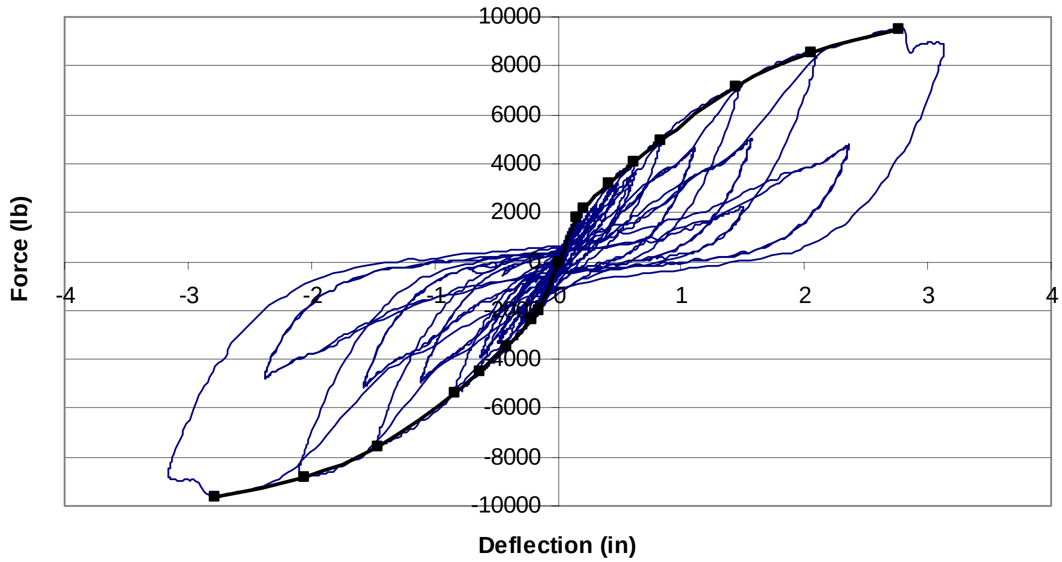


Fig. B6. Wall A5 hysteresis curve.

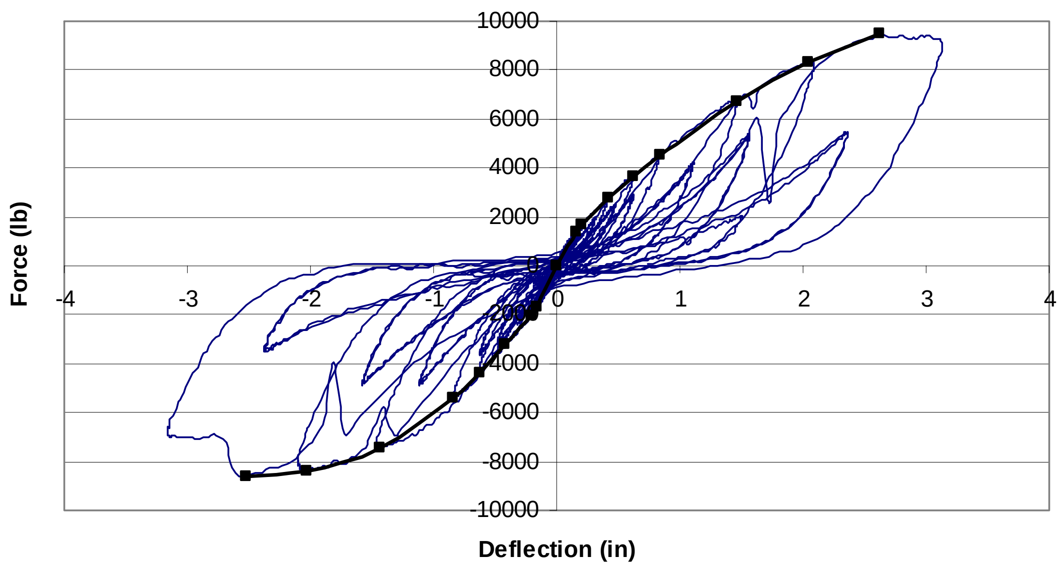


Fig. B7. Wall A6 hysteresis curve.

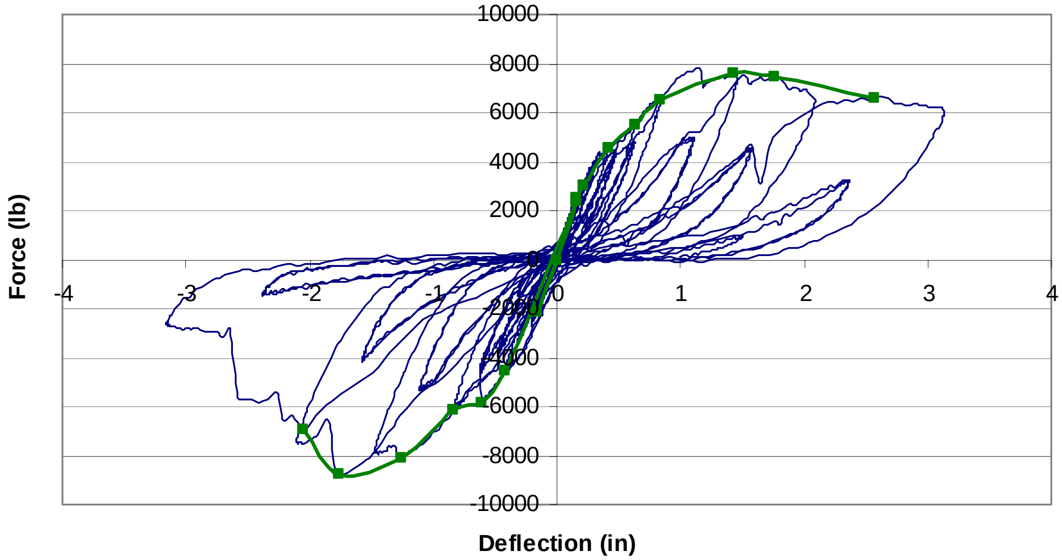


Fig. B8. Wall A7 hysteresis curve.

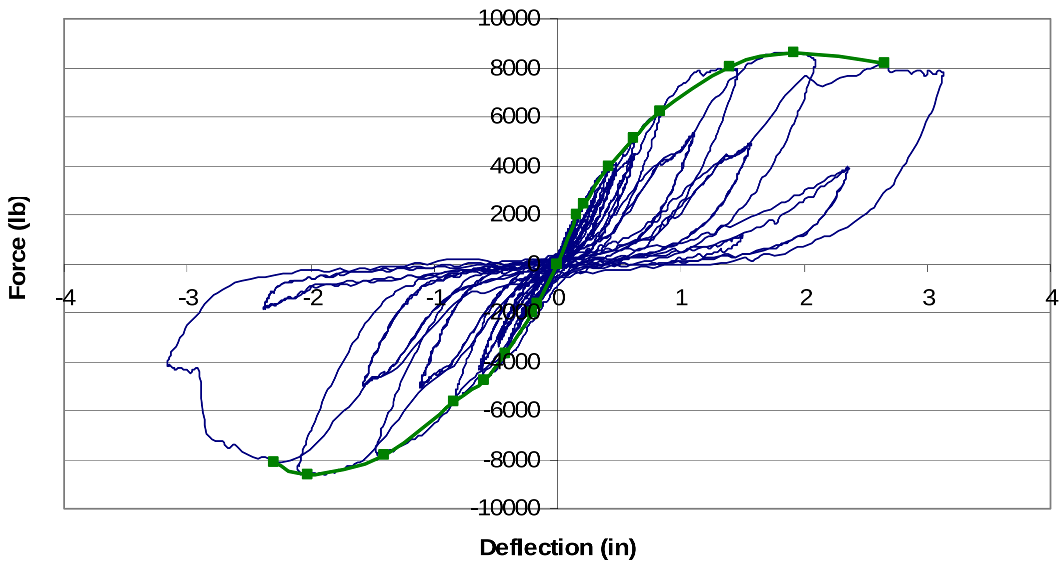


Fig. B9. Wall A8 hysteresis curve.

Table B2. Cyclic wall test results.

Wall		F_u (lb)	Δ_u (in)	K_0 (lb/in)	Energy (lb·in)	Ductility
Type 1	A1	4884	3.00	9167	72011	14.4
	A2	4584	2.03	7883	67628	9.8
	Mean	4734	2.52	8525	69819	12.1
Type 2	A3	4634	2.50	8593	58035	11.8
	A4	4408	1.85	7078	56332	9.0
	Mean	4521	2.17	7836	57184	10.4
Type 3	A5	9610	2.78	10720	94566	13.5
	A6	9484	2.62	8807	78872	12.6
	Mean	9547	2.70	9764	86719	13.1
Type 4	A7	8774	1.80	14411	72640	8.5
	A8	8645	1.92	11586	73196	9.1
	Mean	8710	1.86	12999	72918	8.8

The shearwalls exhibited various types of failure. Failure of the sheathing nails was witnessed on all walls. The most common modes of nail failure in the lateral shearwall tests were pull-through and withdrawal, with a few nails exhibiting fatigue failure. All nail failure was confined to the perimeter sheathing nails. The field sheathing nails showed no visible signs of failure. After each shearwall test, the sheathing nails were thoroughly examined and the failure modes were recorded. Examples of nail pull-through, withdrawal and fatigue failures are shown in Fig. B10. The damage on each shearwall is shown in Fig. B11-18. The field nails are not included in the summary. The nail conditions in Fig. B11-B18 are coded as withdrawal (W), pull-through (P), fatigue (F), and no failure (blank).

Fig. B10. Nail failure modes (left) withdrawal, (upper right) pull-through and (lower right) fatigue.

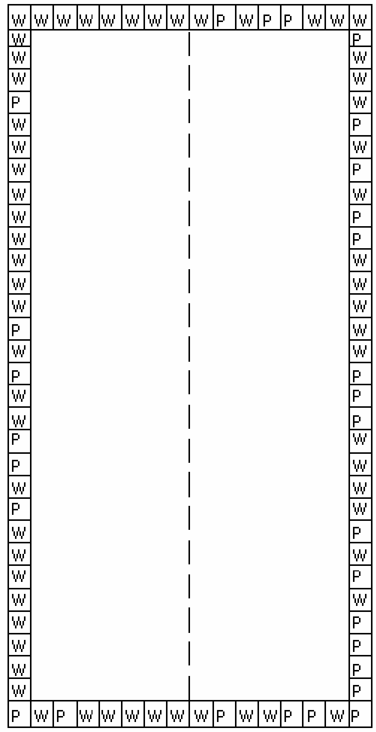


Fig. B11. Sheathing nail failure modes for Wall A1.

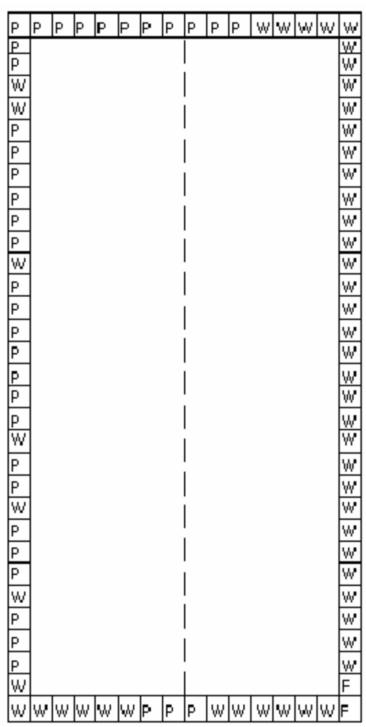


Fig. B12. Sheathing nail failure modes for Wall A2.

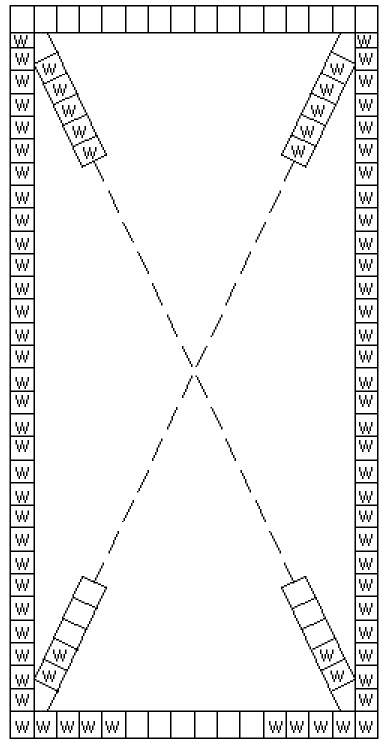


Fig. B13. Sheathing nail failure modes for Wall A3.

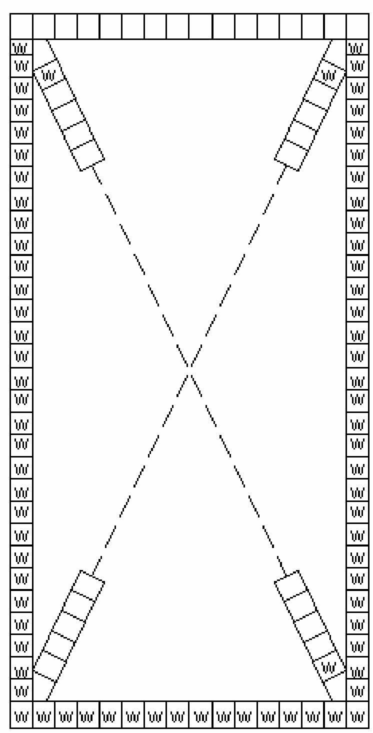


Fig. B14. Sheathing nail failure modes for Wall A4.

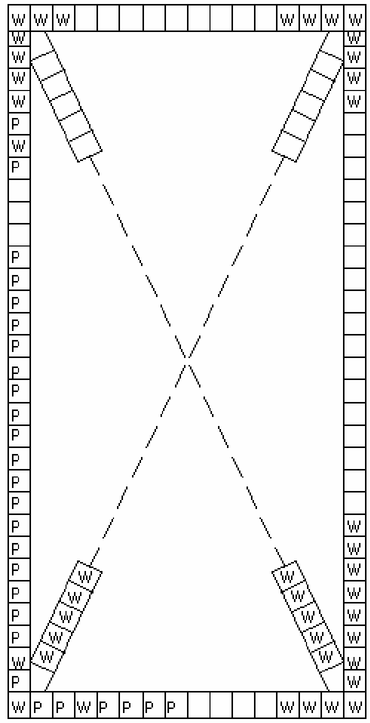


Fig. B15. Sheathing nail failure modes for Wall A5.

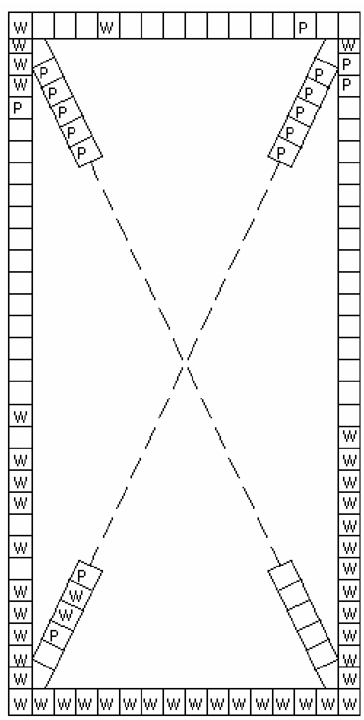


Fig. B16. Sheathing nail failure modes for Wall A6.

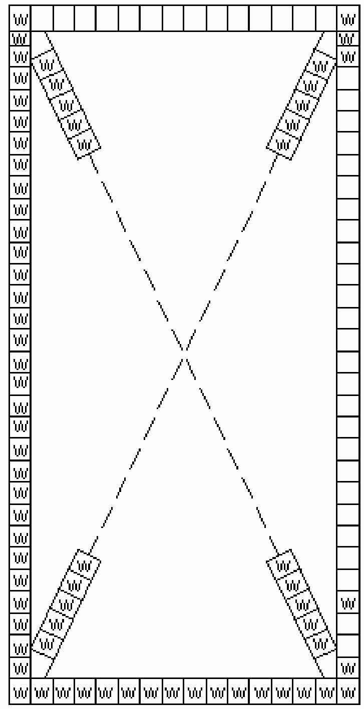


Fig. B17. Sheathing nail failure modes for Wall A7.

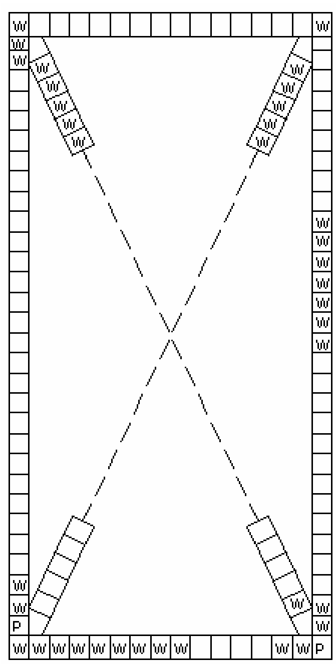


Fig. B18. Sheathing nail failure modes for Wall A8.

The nail failures reported in Figures B11-B18 did not express the degree of failure exhibited in the nails. Of importance to note is that nails in the Type 3 and Type 4 shearwalls did not withdrawal and pull-through as much as the nails in the Type 1 and Type 2 shearwalls. The withdrawal and pull-through of nails in the Type 1 and Type 2 shearwalls was clear in most cases just by looking at the wall. Many of the nails pulled all the way through the OSB in the Type 1 and Type 2 shearwalls, while the Type 3 and Type 4 shearwalls did not have any nails that pulled completely through the OSB. Similarly, nails in the Type 1 and Type 2 shearwalls withdrew by larger amounts (sometimes more than 1/2-in.), while withdrawal in the Type 3 and Type 4 shearwalls had to be determined by whether or not a folded piece of paper could slide between the OSB and the wood frame.

The Type 1 and Type 2 shearwalls performed similarly during the cyclic testing. The sheathing rotated relative to the wall frame and the top-plate pulled up from the end studs withdrawing the toenails connecting the top-plate to the end studs (Fig. B19). There was no indication of the center stud separating from the top-plate during the test for the Type 1 shearwalls. The braces in the Type 2 shearwalls, which were only attached to the sheathing, also pulled away from the end studs (Fig. B20). The movement of the brace and stud tended to loosen the hold-down connection on the Type 2 braced shearwalls. The majority of the nail failures occurred at the largest amplitude cycles of the test.

The plywood gusset plates on the Type 3 shearwalls exhibited failures under cyclic loading (Fig. B21-B22) at both the top of the wall and the bottom of the wall. At large cycles, after the plywood gusset plate failed, a small gap occurred between the brace and top-plate and the end studs and the top-plate. The hold-down also lifted at the ends of the wall under the large cyclic loads.

Fig. B19. Conventional shearwall top-plate separation.

Fig. B20. Braced shearwall top-plate and brace separation.

Fig. B21. Plywood gusset plate failure at top corner of wall.

Fig. B22. Plywood gusset plate failure at bottom corner of wall.

The metal truss plates on the Type 4 shearwalls buckled in compression when the applied force reached approximately 5000 lb (Fig. B23). These two walls also had significant tension perpendicular to grain failures of the bottom-plates (Fig. B24). The bottom-plate also exhibited compression and bending failures. Plate washers for the shear bolts at the bottom-plate embedded into the wood bottom-plate, crushing the wood member. The plate washers also experienced permanent deformations due to bending. Additionally, Wall A7 experienced a hold-down failure (Fig. B23). There was no indication of separation of the top-plate from the end studs or the brace from the top-plate or end studs.

Fig. B22. Failure of the hold-down and buckling of the metal truss plate.

Fig. B23. Bottom-plate failure.

The energy dissipated by the walls was calculated based on the area under each of the individual hysteresis loops of the test data. The sum of all of the curves is the cumulative energy dissipated by the walls (Table B3).

Table B3. Wall energy dissipated per cycle (lb·in).

Cycle	Wall							
	Type 1		Type 2		Type 3		Type 4	
#	**A1**	**A2**	**A3**	**A4**	**A5**	**A6**	**A7**	**A8**
1	55	63	62	46	70	56	47	54
2	45	55	57	41	59	46	44	43
3	44	55	56	41	58	43	43	39
4	43	56	55	39	57	41	41	36
5	43	57	54	39	57	41	41	37
6	43	55	53	38	57	39	40	37
7	110	126	121	83	142	97	101	94
8	62	74	72	50	81	51	51	43
9	59	72	65	47	76	47	50	39
10	57	70	68	46	73	45	44	39
11	57	69	67	46	72	45	48	41
12	56	69	62	46	72	45	48	41
13	58	66	67	46	71	43	48	40
14	203	222	208	146	234	162	170	143
15	114	127	120	80	139	83	87	77
16	109	114	113	77	133	78	78	69
17	108	119	111	73	129	75	76	69
18	110	113	110	94	127	76	75	68
19	107	118	109	104	129	75	76	66
20	110	119	111	97	129	75	72	68
21	821	948	794	681	840	703	1098	725
22	399	481	361	296	379	280	354	207
23	374	439	342	340	354	264	261	191
24	366	425	337	368	351	260	262	183
25	1705	1738	1407	1390	1632	1323	2581	2039
26	758	958	619	593	711	572	787	571
27	705	908	582	554	671	544	496	399
28	688	894	568	539	662	537	471	394
29	2535	2636	2010	1947	2657	2125	3043	2740
30	1109	1369	821	816	1120	861	1173	968
31	1065	1323	791	763	1007	839	740	611
32	7211	7140	5824	6127	8360	7500	9879	8763
33	2599	2895	1814	1829	3381	2178	2931	2657
34	2392	2719	1646	1670	3107	2035	2406	2388
35	10540	10296	8680	9028	14204	12823	14614	13621
36	3874	4157	2865	2868	5166	3787	2926	3196
37	3641	3892	2665	2625	4794	3386	2403	2772
38	17257	15187	15416	14697	27411	25599	19517	23805
39	5909	3746	4293	3925	7832	6035	2996	3193
40	5425	3065	3777	3382	6898	5082	2102	2293
41	1047	593	683	616	1066	877	318	337
Total	72011	67628	58035	56332	94566	78872	72640	73196
Average	69819		57184		86719		72918	
Std Dev	3099		1204		11097		393	

Appendix C – Vertical Load Tests of Shearwalls

To evaluate the feasibility of using the braced shearwalls as a part of conventional light-frame construction the vertical load performance, in addition to the lateral performance, had to be tested. In conventional shearwalls the floor and roof joists are supported by stud members in compression. The configuration of the braced shearwall omits the intermediate stud, changing the load path that is typically used for floor and roof joists load transfer to the lower floors or foundation. To enhance the vertical performance of braced shearwalls a 4x4 top-plate was used, in lieu of the typical double 2x4 top-plate.

A total of four 4 by 8-ft. shearwalls were constructed for the vertical shearwall tests. Two walls (B1 and B2) were built as Type 1 conventional shearwalls. The two remaining walls were built as Type 4 braced shearwalls with metal truss plates at the corners each side of the wall. The walls were constructed as described in Appendix A. The Type 3 braced shearwall with and the Type 4 braced shearwall both showed better lateral performance than the braced shearwall without corner connections. Even though the Type 3 braced shearwall had a higher maximum capacity and dissipated more energy, the Type 4 braced shearwall was chosen for the vertical tests. The primary reason for choosing the Type 4 braced shearwall over the Type 3 braced shearwall was the ease of construction. Easier construction, which amounts to cost savings, would increase the likelihood of the wall being used in actual light-frame construction.

One of the major concerns with vertical loading is serviceability, particularly top-plate deflection. Frequently serviceability issues, rather than strength, dictate the size of structural members. The 2003 IBC does not include specific requirements for top-plate deflection; however, it does give guidance for deflection limitations that are considered good practice (2003 IBC, Section 1617.4.2). Typically, live load deflections are limited to L/360 for floors and L/240 for roofs (where L is the length of the span). Similarly, live load plus dead load deflections are limited to L/240 for floors and L/180 for roofs.

In conventional shearwall design, the studs are typically spaced at 24-in. or 16-in. on center. These studs act as columns to support roof or floor joists. The vertical capacity of the wall is typically determined by performing column calculations on the studs. Axial deformation of a stud in compression parallel to the grain is generally very small, and therefore neglected. For this reason, deflection of the top-plate is not usually considered in conventional light-frame design. This should not be the case for the braced shearwall where there is no intermediate stud to act as a column. The top-plate must act as a bending member.

The bending capacity of a 4x4 under vertical loading is easily solved. The allowable bending stress value (F_b) and the size factor (C_F) for a No.2 Douglas Fir-Larch 4x4 can be obtained from Table 4A in the 2001 NDS Supplement (AF&PA 2001). The bending capacity calculation assumed the top-plate was simply supported, conservatively ignoring the effects of the metal truss plates at the corners of the wall and the composite action of the sheathing, which is nailed to the top-plate. For a span length of 4-ft. with two point loads at 16-in. on center the maximum allowable point load is 603 lb. Normal loading duration was used in these calculations. A uniform load of 452 pounds per lineal foot (plf) is equivalent to point loads continuously spaced at 16-in. on center. Typical roof loads for a light-framed building in Oregon might be 15 pounds per square foot (psf) dead load and 25 psf live (snow) load. This is the equivalent of having roof joists at 16-in. on center with over 11-ft. of tributary width going to the top-plate of the wall (452 plf / (15+25 psf) = 11.3 ft.). Assuming the joists are simply supported between two walls this implies a roof joist span of approximately 22 feet. Similar calculations performed for floor joists (assuming 15 psf dead load and 40 psf live load) produced a floor joist span of approximately 16-ft. These are relatively long spans for wood joists and it is unlikely that many light-framed wood buildings would have joists spans longer than this. Additionally, if strength was a problem, a more complex analysis could be performed including the effects of the metal truss plates, sheathing and the load duration to give a less conservative estimate of the vertical capacity of the top-plate.

Another potential serviceability issue relates to deflection of the top-plate as it affects the interior wallboard panel. Small local deflections in the top-plate could cause cracking of the ceiling wall joist or in the wall panel below the joint. This study does not address this potential problem since we do not have a method of calculating the local deformation of the interior wall panel or a method for predicting the level of deformation at which cracking will occur.

From the prior calculations, strength was excluded as a controlling factor in the vertical assessment of the shearwalls. Deflection was considered the main criteria for this study in evaluating the vertical performance of the shearwalls. Determining deflection for a simply supported beam under vertical loading is also a problem that is easily solved. However, the effects of the metal truss plates and the sheathing are not as straight forward. The main focus of the vertical testing was to determine the stiffness of the wall under typical service conditions.

No standard testing protocol exists for vertical load performance of light-framed walls so an apparatus had to be developed to perform vertical load tests. The test apparatus was horizontally orientated so that the structural floor could serve as reaction for the apparatus foundation and loading fixtures. The test apparatus consisted of a series of steel wide-flange beams and columns to support the wall and restrict movement similar to the confines of building construction. A wide-flange beam was placed along the bottom-plate similar to a continuous foundation support. The continuous wide-flange beam was supported at the ends and midspan by wide-flange sections bolted to the laboratory floor (Fig. C1). The modulus of elasticity for steel is

29,000,000 psi, while the average modulus of elasticity for the 2x4 lumber used in the wall construction was 1,790,000 psi and 1,700,000 psi for the 4x4 lumber (Appendix A). Since the modulus of elasticity of the steel is so much higher than wood, the deflection of the wide flange beam supporting the bottom of the wall was assumed to be negligible. The test walls were supported by wood stickers to eliminate the possibility of the floor providing frictional resistance.

Fig. C1. Vertical load shearwall test apparatus.

A hydraulic actuator was set up on the end of the wall opposite the continuous wide flange beam support. The hydraulic cylinder was attached to a steel beam with two points of contact to the top of the wall. The points of contact were spaced at 16-in. on center about the centerline of each wall. Each contact point had an adjustable steel bearing plate to apply the loads to the top of the wall.

The following displacements measured describe the movement of the wall under vertical loading (Fig. C2 and C3):

- Bottom slip displacement – the movement between the OSB sheathing and the bottom-plate.
- Top slip displacement – the movement between the OSB sheathing and the top-plate.
- Top reference displacement – the top-plate displacement relative to a stationary point.

Three linear variable differential transformers (LVDT) were attached to the wall to assess movement during loading. The electrical signal from the LVDT was sent to a controller that converted the information to a displacement. The displacement information was recorded for each of the LVDT's used for testing.

The steel beam near the top of the wall limited accessibility to apply an LVDT directly to the top-plate of the walls, therefore, a steel angle was screwed to the top-plate. One LVDT was fixed at the laboratory floor and measured the deflection of the angle. The deflection of the steel angle was assumed negligible. Two additional LVDT's were attached to the sheathing to measure the relative displacement slip between the sheathing and the top and bottom-plates. The movement of the LVDT's was recorded at 1-second intervals. Fig. C2 and Fig. C3 show the general set-up of the vertical test apparatus and the location of the LVDT's for the braced wall and conventional shearwall, respectively. The truss plates are not shown on the braced wall for clarity.

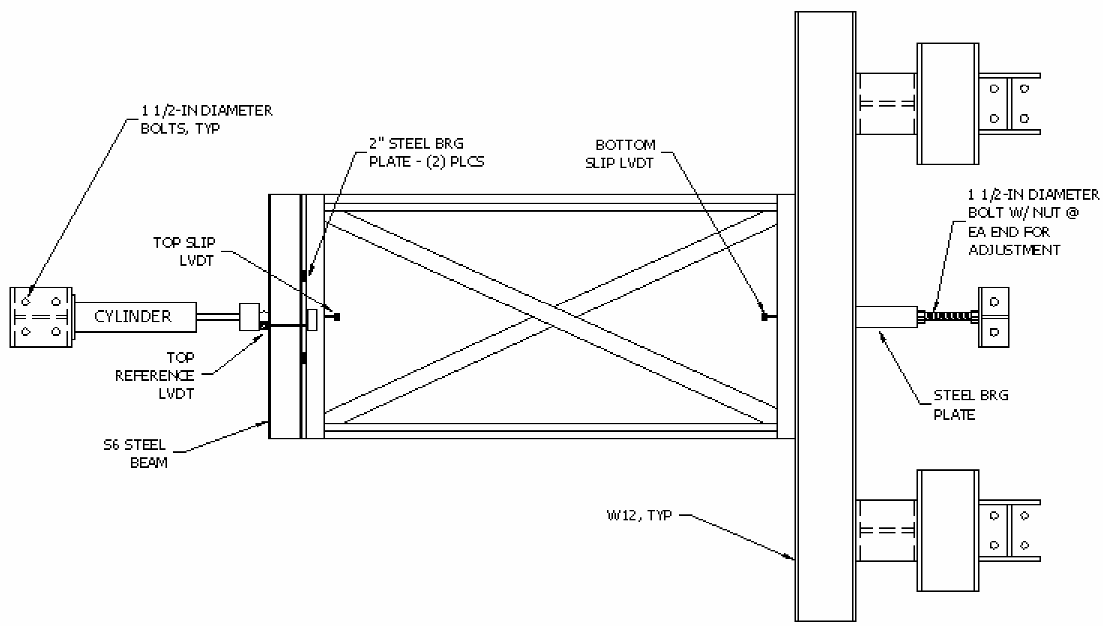


Fig. C2. Braced wall vertical load test set-up.

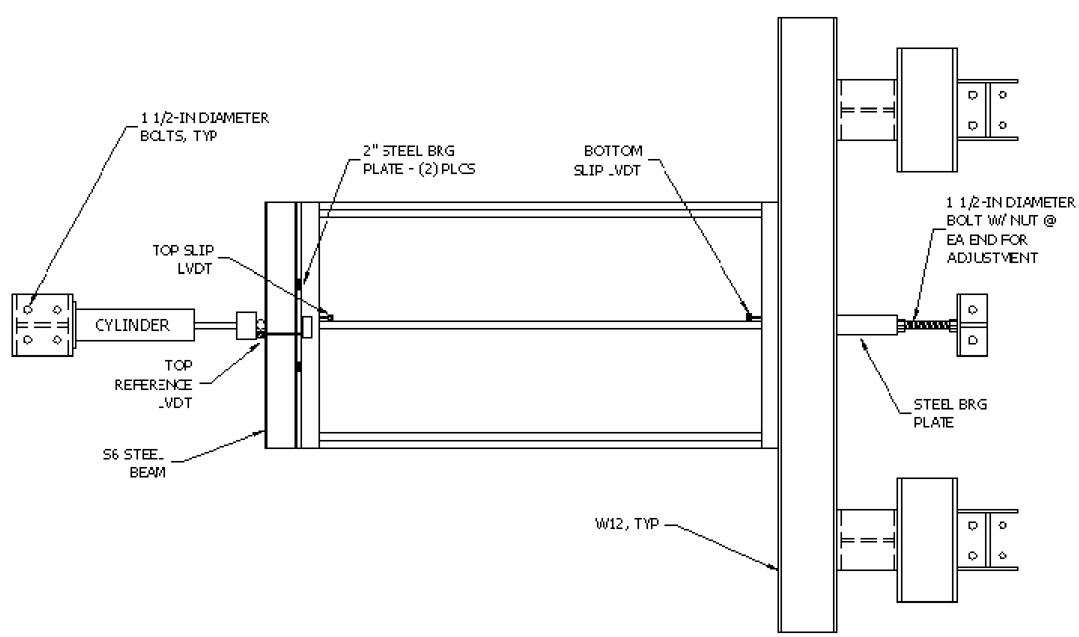


Fig. C3. Conventional wall vertical load test set-up.

Without the guidance of a standard test procedure, a vertical loading protocol was developed. The goal of the vertical testing was to determine the stiffness of the wall, which is directly proportional to deflection. The wood members are designed using allowable stress design, and in allowable stress design, wood members stay within their elastic limits for given service loads. For the wall assembly to remain within its elastic limit, loads must be used that would not create plasticity in the fasteners or the wood materials. Two different loading conditions were performed to simulate the different possible configurations of roof and floor joists on a wall. At the roof level of a light-framed wood building, the joist commonly bears on the top-plate and the sheathing, while at the floor level the joist commonly bears only on the top-plate.

A load of approximately 700-lb was initially applied to both the sheathing and top-plate through the steel bearing plate contacting the entire width of the top-plate and sheathing. The load was then released and another load of approximately the same magnitude was applied to the top-plate only by adjusting the steel bearing plate to only contact the top-plate. Once more the load was released and a third load was applied, again of the same magnitude, to the top-plate and sheathing. Fig. C4 is an example of the loading for each of the vertical tests.

Table C1 shows the maximum displacements recorded at each of the locations. The maximum displacement recorded at the bottom of the wall was approximately two-thousandths of an inch. Deflection measured at the bottom of the wall was very small given the size of the wall and the magnitude of the other displacement measurements, and was therefore neglected in any further analysis.

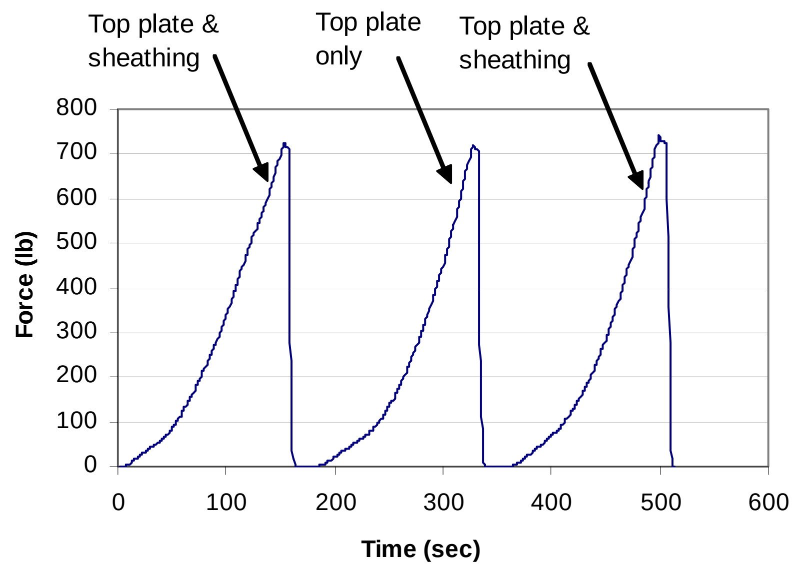
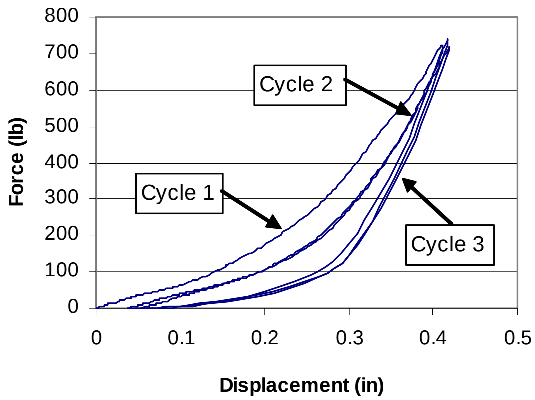


Fig. C4. Typical vertical load test force-time diagram.

Table C1. Maximum displacements from wall tests.

		Maximum Displacement (in.)		
		Bottom	Top	Top
Wall		Slip	Slip	Reference
Type 1	B1	0.0021	0.1354	0.4197
	B2	0.0011	0.0646	0.2941
	Mean	0.0016	0.1000	0.3569
Type 4	B3	0.0004	0.0906	0.2062
	B4	0.0004	0.0085	0.0591
	Mean	0.0004	0.0495	0.1327

Problems were experienced with the test apparatus when testing one of the Type 4 walls (Table C1). Fig. C5 – C8 show the top reference displacement and top slip displacement of each wall as a function of the applied load. Walls B1 and B2 are the Type 1 conventional shearwalls and Walls B3 and B4 are the Type 4 braced shearwalls.



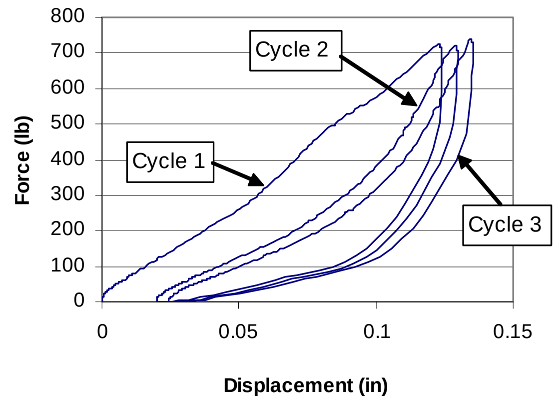
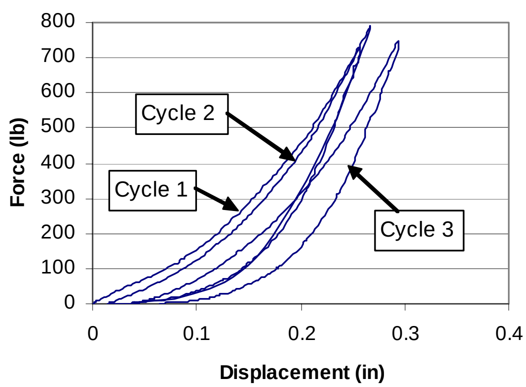


Fig. C5. Wall B1 top reference displacement (left) and top slip displacement (right).



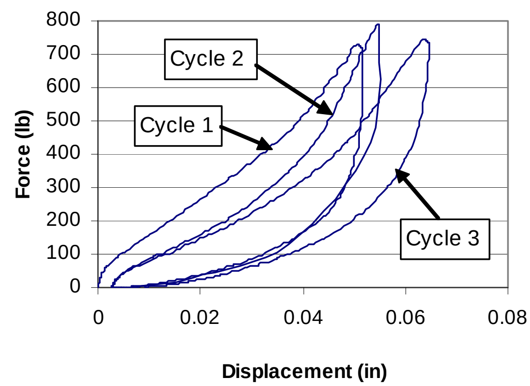
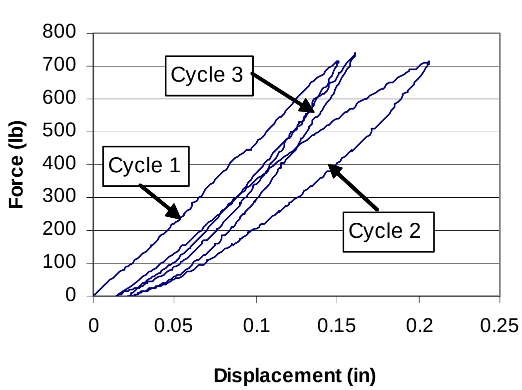


Fig. C6. Wall B2 top reference displacement (left) and top slip displacement (right).



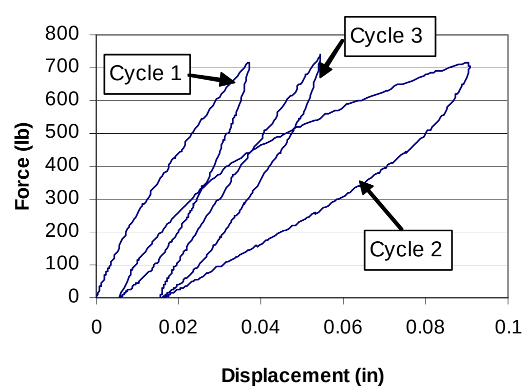


Fig. C7. Wall B3 top reference displacement (left) and top slip displacement (right).

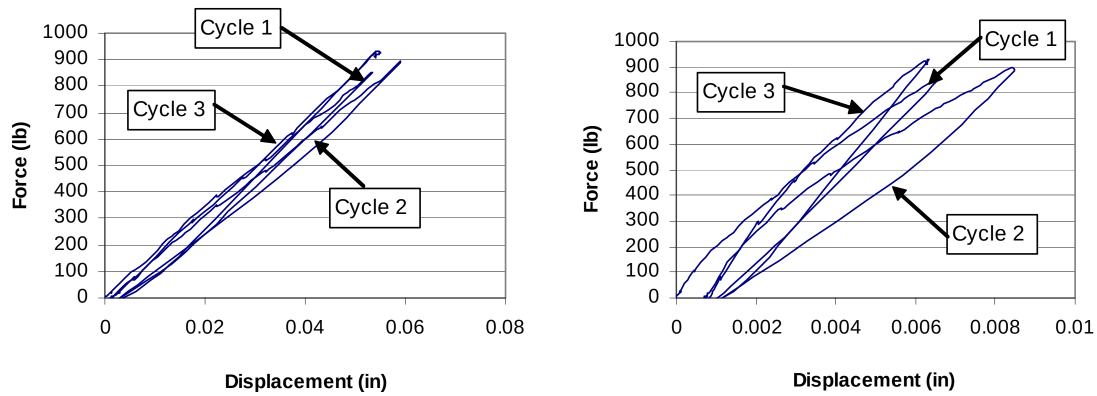


Fig. C8. Wall B4 top reference displacement (left) and top slip displacement (right).

The three cycles in the graphs represent the load-displacement curves for the three cycles of loading. The first cycle (Cycle 1) corresponds to loading of the top-plate and sheathing, the second cycle (Cycle 2) corresponds to loading of the top-plate only and the last cycle (Cycle 3) corresponds to loading of the top-plate and sheathing. Although the measured displacements varied largely from one another, it appears there is very little difference in the slope of the curves for the three loops for an individual wall. Loading the top-plate only versus loading the top-plate and sheathing appears to make little difference to the stiffness (represented by the slope of the plotted force-displacement data) of the wall. This suggests that the vertical load path of the forces does not change if the sheathing is loaded with the top-plate or not.

The stiffness was determined by calculating the slope from the load-displacement curves for each load cycle using the top reference displacement data. Initial stiffness refers to the stiffness observed at the onset of testing (for Type 1 walls only). The initial stiffness was calculated using data for the first 100 lb of loading (0 – 100 lb). Structural stiffness refers to the stiffness of the wall without regard to the initial stiffness. The structural stiffness of the Type 1 walls was calculated using data for the loads over 300 lb (300 lb – P_{max}), while the structural stiffness of the Type 4 walls was determined using the entire range of data (0 lb - P_{max}). Table C2 summarizes the stiffness for each wall.

The Type 1 walls exhibited two distinct regions of different stiffness (Fig. C5 and C6). The initial stiffness, characterized by the initial slope, is much lower than the structural stiffness. Nuisances of the testing set-up and construction of the walls likely caused the lower initial stiffness. If the wall was not perfectly snug against the wide-flange beam settlement would occur upon the initial loading. Similarly, if there were any gaps in the studs to top or bottom-plate connections these would close causing additional displacement that might not otherwise typically occur under vertical loading conditions.

The slip measured between the top-plate and the sheathing was much larger than the slip measured between the bottom-plate and the sheathing for all the walls. The slip observed between the top-plate and the OSB could have occurred in the sheathing-to-framing connections at the top-plate. All walls experienced some permanent displacement during each load cycle. This is represented by the second and third load-displacement cycles not originating at zero displacement. Cycle 2 corresponds to the load applied only to the top-plate. The load-displacement curves show higher magnitudes of slip displacement at the top-plate in the Type 4 walls in Cycle 2 (Fig. C7 and C8), while the Type 1 walls did not show a higher magnitude of slip at Cycle 2. The lack of an intermediate stud at the Type 4 braced shearwalls allowed the top-plate to displace more relative to the sheathing than the top-plate at the conventional shearwalls when only the top-plate was loaded.

Type 4 walls appeared to have a constant stiffness throughout the duration of the testing. Gaps between the studs and the top and bottom-plates would not compress initially (as seen in the Type 1 walls) since the truss plates were applied to both sides of the walls at each corner. The truss plates provided additional fixity at the corners by preventing vertical movement at these locations.

Table C2. Wall stiffness (lb/in).

	Loop 1		Loop 2		Loop 3		Average
	Initial Stiffness	Structural Stiffness	Initial Stiffness	Structural Stiffness	Initial Stiffness	Structural Stiffness	Structural Stiffness
B1	685	2842	614	3842	683	4106	3597
B2	1373	4011	1330	4921	1234	4619	4517
B3	N/A	4824	N/A	3879	N/A	5256	4653
B4	N/A	15903	N/A	15261	N/A	17399	16188

The average structural stiffness for the Type 1 walls (B1 and B2) and the Type 4 wall (B3) is relatively consistent, ranging from approximately 3600 lb/in to 4600 lb/in. This suggests that deflection would be similar in the top of the wall regardless of whether a conventional shearwall or braced shearwall was used. The addition of the truss plates reduces the effective span of the top-plate for the braced shearwalls and likely helps distribute the load to the end studs.

The stiffness of Wall B4 is nearly four times higher than the other three walls. It is unclear what caused the stiffness in this particular wall to be so high. The modulus of elasticity of the top-plate for Wall B4 was 1,370,000 psi, while the average modulus of elasticity for all the 4x4 members used in all the wall construction was 1,700,000 psi (Appendix A). Therefore, the increased stiffness was not caused by a difference in the modulus of elasticity of the top-plate. The increased stiffness of Wall B4 might been caused by an error in testing or recording that went unnoticed during testing, possibly a component that got hung up during the test and could not move freely.

Appendix D – Statistical Analysis

SPSS Version 14.0 (SPSS, Inc. 2005) software was used to perform the statistical analysis. A one-way analysis of variance was used to determine significant differences in means for F_u, Δ_u, K_0, Energy, and Ductility. Table D1 and Table D2 are the output files from the SPSS analysis.

Table D1. SPSS output for one-way analysis of variance.

ANOVA

		Sum of Squares	df	Mean Square	F	Sig.
F_u	Between Groups	41260276	3	13753425	633.8	0.000
	Within Groups	86797	4	21699		
	Total	41347073	7			
Δ_u	Between Groups	0.830	3	0.277	1.576	0.327
	Within Groups	0.702	4	0.175		
	Total	1.531	7			
K_0	Between Groups	58468665	3	19489555	6.455	0.052
	Within Groups	12076491	4	3019123		
	Total	70545156	7			
Energy	Between Groups	882625075	3	294208358	8.759	0.031
	Within Groups	134360835	4	33590209		
	Total	1016985910	7			
Ductility	Between Groups	21.164	3	7.055	1.871	0.275
	Within Groups	15.085	4	3.771		
	Total	36.249	7			

Table D2. SPSS output for Tukey multiple comparisons.

Multiple Comparisons

Tukey HSD

Dependent Variable	(I) Wall	(J) Wall	Mean Difference (I-J)	Std. Error	Sig.	95% Confidence Interval	
						Bound	Bound
F_u	1	2	213	147.31	0.538	-386.6625	812.6625
		3	-4813.0(*)	147.31	0.000	-5412.663	-4213.338
		4	-3975.5(*)	147.31	0.000	-4575.163	-3375.838
	2	1	-213	147.31	0.538	-812.6625	386.6625
		3	-5026.0(*)	147.31	0.000	-5625.663	-4426.338
		4	-4188.5(*)	147.31	0.000	-4788.163	-3588.838
	3	1	4813.0(*)	147.31	0.000	4213.3375	5412.6625
		2	5026.0(*)	147.31	0.000	4426.3375	5625.6625
		4	837.5(*)	147.31	0.016	237.8375	1437.1625
	4	1	3975.5(*)	147.31	0.000	3375.8375	4575.1625
		2	4188.5(*)	147.31	0.000	3588.8375	4788.1625
		3	-837.5(*)	147.31	0.016	-1437.163	-237.8375
Δ_u	1	2	0.34	0.419	0.847	-1.365	2.045
		3	-0.185	0.419	0.968	-1.89	1.52
		4	0.655	0.419	0.485	-1.05	2.36
	2	1	-0.34	0.419	0.847	-2.045	1.365
		3	-0.525	0.419	0.631	-2.23	1.18
		4	0.315	0.419	0.872	-1.39	2.02
	3	1	0.185	0.419	0.968	-1.52	1.89
		2	0.525	0.419	0.631	-1.18	2.23
		4	0.84	0.419	0.320	-0.865	2.545
	4	1	-0.655	0.419	0.485	-2.36	1.05
		2	-0.315	0.419	0.872	-2.02	1.39
		3	-0.84	0.419	0.320	-2.545	0.865
K_0	1	2	150.5	1737.6	1.000	-6922.867	7223.8667
		3	-2119	1737.6	0.648	-9192.367	4954.3667
		4	-6545.5	1737.6	0.064	-13618.87	527.8667
	2	1	-150.5	1737.6	1.000	-7223.867	6922.8667
		3	-2269.5	1737.6	0.605	-9342.867	4803.8667
		4	-6696	1737.6	0.060	-13769.37	377.3667
	3	1	2119	1737.6	0.648	-4954.367	9192.3667
		2	2269.5	1737.6	0.605	-4803.867	9342.8667
		4	-4426.5	1737.6	0.190	-11499.87	2646.8667
	4	1	6545.5	1737.6	0.064	-527.8667	13618.867
		2	6696	1737.6	0.060	-377.3667	13769.367
		3	4426.5	1737.6	0.190	-2646.867	11499.867

* The mean difference is significant at the .05 level.

Table D2 (Continued). SPSS output for Tukey multiple comparisons.

Dependent Variable	(I) Wall	(J) Wall	Mean Difference (I-J)	Std. Error	Sig.	95% Confidence Interval	
						Lower Bound	Upper Bound
Energy	1	2	12636	5795.7	0.271	-10957.49	36229.487
		3	-16899.5	5795.7	0.134	-40492.99	6693.9873
		4	-3098.5	5795.7	0.946	-26691.99	20494.987
	2	1	-12636	5795.7	0.271	-36229.49	10957.487
		3	-29535.5(*)	5795.7061	0.024	-53128.99	-5942.013
		4	-15734.5	5795.7061	0.162	-39327.99	7858.9873
	3	1	16899.5	5795.7061	0.134	-6693.987	40492.987
		2	29535.5(*)	5795.7061	0.024	5942.0127	53128.987
		4	13801	5795.7061	0.223	-9792.487	37394.487
	4	1	3098.5	5795.7061	0.946	-20494.99	26691.987
		2	15734.5	5795.7061	0.162	-7858.987	39327.987
		3	-13801	5795.7061	0.223	-37394.49	9792.4873
Ductility	1	2	1.7	1.94197	0.818	-6.2055	9.6055
		3	-0.95	1.94197	0.957	-8.8555	6.9555
		4	3.3	1.94197	0.428	-4.6055	11.2055
	2	1	-1.7	1.94197	0.818	-9.6055	6.2055
		3	-2.65	1.94197	0.577	-10.5555	5.2555
		4	1.6	1.94197	0.841	-6.3055	9.5055
	3	1	0.95	1.94197	0.957	-6.9555	8.8555
		2	2.65	1.94197	0.577	-5.2555	10.5555
		4	4.25	1.94197	0.268	-3.6555	12.1555
	4	1	-3.3	1.94197	0.428	-11.2055	4.6055
		2	-1.6	1.94197	0.841	-9.5055	6.3055
		3	-4.25	1.94197	0.268	-12.1555	3.6555

* The mean difference is significant at the .05 level.

Appendix E – Notation

A	cross-sectional area
C_F	size factor
C_s	seismic coefficient
C_{vx}	vertical distribution factor
Ductility	Δ_u divided by Δ at P_{yield}
E, MOE	modulus of elasticity
Energy	sum of energy dissipated by the walls based on the area under each of the individual hysteresis loops of the test data
F	global force vector
F_b	allowable bending stress
F_u	maximum load, peak capacity
h	height of the wall
h_i, h_x	height from building base to level i or x
I	moment of inertia
I	importance factor
K_0	initial stiffness of the hysteresis
k	period exponent
L	length
n	number tested samples
P_{max}	maximum load (vertical)
P_{yield}	maximum load of the second primary cycle
R	response modification factor
S_{DS}	design spectral response acceleration
SG	specific gravity
v_a	allowable unit shear force (plf)
v_u	ultimate unit shear force (plf)
V	base shear

W	effective seismic weight
w_i, w_x	effective seismic weight assigned to level i or x
Δ_m	displacement at which the applied load drops for the first time below 0.8 of the maximum load
Δ_{ref}	reference displacement for the CUREE load protocol, $0.6\Delta_m$
Δ_u	the displacement at the ultimate load