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Monotonic Testing of Fully and Partially Anchored Wood-Shear Walls

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MONOTONIC TESTING OF FULLY AND PARTIALLY ANCHORED WOOD SHEAR WALLS

by

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Chapter 1

Introduction

Currently in the United States of America (USA) there are around 125 million single-family dwellings (SFDs), most of which consist of wood-frame construction (SSA 2015). Traditionally wood frame construction SFDs have performed well in seismic events but more efficient designs against earthquake loads is desired. More than 143 million Americans are living in seismic regions and nearly 28 million live in high seismic regions (SSA 2015). During an earthquake event, the lateral forces created are characterized as cyclic and random. The main lateral force resisting system for these wood-frame SFDs is the shear wall, therefore it is critical that the shear wall has the ability to resist cyclic and random lateral forces. This study focuses on the effects of earthquake loads on residential wood shear walls.

Most wood-frame SFDs were constructed and designed to building code provisions before seismic requirements were introduced. Even after building code accounted for seismic provisions, the design values for shear walls were based on static loading tests. Static tests are not able to capture the same effects of an earthquake. During a static test, the shear wall is loaded at a constant rate in one direction until failure occurs. This does not accurately reflect the random reversal or cyclic loads exhibited during an earthquake (Folz 2001).

It was not until the 1994 Northridge earthquake in southern California that revisions to the design of wood shear walls began to be researched. The Northridge earthquake is one of the expensive and life taking earthquake events in the U.S. $40 billion in economic loss, 60 casualties, 7000 injured, and over 40,000 houses damaged were estimated as a result of the Northridge earthquake. After the Northridge earthquake, the Federal Emergency management Association (FEMA) funded the "CUREE-Caltech wood-frame project" (Kirkham et al. 2012). The purpose of the project was to focus research on mitigating damage to wood frame houses during an earthquake through appropriate cyclic-testing protocols. Research was directed towards more accurately representing earthquake cyclic and random reversal loads.

There are currently two design approaches for residential wood-frame single family dwellings (WFSFDs) to resist seismic lateral loads. The more commonly applied is the International Residential Code (IRC). The latter being International Building Code (IBC). The main difference between the two is IRC assumes overturning moments and tension forces are resisted by self-weight dead loads and perpendicular walls. As a result, IRC does not require hold-down ties installed. However, many times the same wall designed by IBC code would require hold-down ties. Shear walls with hold-down ties are considered fully anchored while shear walls with no hold-downs are considered partially anchored.

One of the objectives of this study is to observe and compare the performance of partial and fully anchored wood shear walls subjected to cyclic loading. In order to achieve this objective, a test frame was designed to test a full scale 8ft x 12ft wall typical of WFSFDs. A low-frequency actuator along with a data acquisition device is used to subject the wall to the cyclic loading procedure. In order to replicate fully dynamic and spectral response loading patterns which is a goal for this project later on, ASTM first requires parameters from the monotonic loading procedure. The parameters found from monotonic loading are a function of the loading pattern for fully dynamic tests. Therefore, this study is the first phase of a multi-phase research project to study the performance of shear walls under earthquake loads with this paper focusing on monotonic loading.

1.2 LITERATURE REVIEW

Wood construction is unique due to it being a very complex organic material. Wood is orthotropic, thus its strength and stiffness properties are different in all three directions. Wood can also have strength reducing characteristics such as knots, shakes, and splits (Ritter 1990). Over time, it will also shrink due to natural loss of water content and creep effect of dead loads. All of these factors require a different design approach for wood unlike other construction materials.

WFSFDs were considered to be very safe in earthquakes (Li and Ellingwood 2007; Skaggs and Martin 2004). Structures built before the 1970s were more traditional one story, usually simple rectangular shape with a continuous roof structure. These WFSDs performed well with regards to life safety and little structural damage. It was not until more modern architectural styled houses started to become more popular that dwellings suffered more damage during earthquakes (Li and Ellingwood 2007). The new architectural style of WFSFDs consisted of multiple stories, segmented roof levels, and irregular shaped framing. Noticeable damage to the new dwellings were observed during the 1971 San Fernando and the 1989 Loma Prieta earthquake. The 1994 Northridge earthquake was the tipping point. The life loss and economic damage was enough to lead to federal grants for wood-frame housing research projects (Kirkham et al. 2012).

In 1998, FEMA announced that it would be funding a $12.1 million, three year research project to study earthquake hazard mitigation in wood-frame structures (CUREE 2002). The project became known as The Consortium of Universities for Research in Earthquake Engineering (CUREE) and worked with the California Institute of Technology (CalTech). The goal of the project is to improve the seismic performance of wood-frame construction through development of cost-effective retrofit strategies, changes in design and construction procedures, and education (Hall 2001). The project produced approximately 30 reports divided into five elements: testing and analysis, field investigations, building codes and standards, economic aspects, and education and outreach. CUREE advanced the knowledge at the time with state-of-the-art research facilities being able to replicate full scale dynamic tests. Though it produced answers to many of the questions of the day, it also provided direction for further research in areas that it could not answer within the project timeframe (Cobeen et al. 2004a,b).

Once, the CUREE project was finished, the Network for Earthquake Engineering Simulation-Wood (NEESWood) began. The National Science Foundation (NSF) funded a $1.2 million grant as a multi-year project to study how wood-frame structures respond to seismic forces. NEESWood continued the work begun by the CUREE-Caltech project by performing and analyzing a series of experiments based on the CUREE prototype buildings (Van de Lindt et al. 2006a, b). The NEESWood project focused on larger scale experiments. One experiment included a full scale six story wood building subjected to an earthquake magnitude of 7.5 through a shake table. Though much larger than typical houses, the experiments with this structure helped to evaluate some of the wood structural systems and elements that are also used in houses (Kirkham et al. 2012). The summary of results from the CUREE and NEESWood projects relating to conventional shear wall testing for purpose of this study is shown in Table 1.

Of the studies, monotonic testing of fully and partially anchored shear walls on 8 ft. x 8 ft. specimens performed by Peter Seaders et al. was reviewed closely. Seaders et al. paper on anchorage effects closely reflects some objectives this study is attempting to accomplish. Seaders et al. performed tests on 8 partially and 2 fully anchored wood shear walls according to ASTM E564. Loading protocol was 1.18 in./min. for and partially anchored walls and 1.77 in./min. for fully anchored walls. The fully anchored shear walls exhibited an average peak load of 5.11 kips and average yield load of 4.58 kips. The partially anchored shear walls exhibited an average peak load of 2.17 kips and average yield load of 1.86 kips.

The limitation of Seaders et al. study and many others for this project is that they did not utilize 12 ft. x 8 ft. shear walls which are the standard size found in residential wood-frame construction and did not take code anchorage effects into account. Seaders et al. compared anchorage effects but on a 1:1 wall to height ratio which is not commonly seen in practice. This study focuses on monotonic loading of fully and partially anchored wood shear walls on 12 ft. x 8 ft. specimens.

Table 1. Conventional Shear Wall Testing Analysis (Kirkham et al. 2012)

Reference	Method/loading	Focus of research
Oliva and Wolfe (1988); Oliva (1990)	ASTM E564. monotonic, static cycles at 1 Hz, dynamic at 5 Hz	Tested 59 gypsum SW for racking resistance. 2.4 m (8 ft) long walls confirmed codes, but longer walls, and horizontal sheets were better. Gluing increased
Thurston and King (1994)	Racking resistance	Ten SW, varying wall returns, openings and materials w/o hold-down
Seible et al. (1999)	Analytical study	CUREE workshop on testing, analysis and design
Karacabeyli et al. (1999)	Static and dynamic	Compares static and dynamic SW test results
Merrick (1999)	cyclic, nonincreasing	7 tests of plywood, OSB, gypsum wallboard SW to evaluate energy dissipation
Salenikovich and Dolan (2000, 2003a, b)	Monotonic, cyclic at 0.25 Hz (ISO 1998)	Investigates the strength of anchored SW, 2.4 m (8 ft) tall, 4:1, 2:1, 1:1, 2:3 aspect ratios
NAHB Research Center (2001)	Static	Strength and deflection of SW with corners and openings
Gatto and Uang (2002, 2003); Uang and Gatto (2003)	Dynamic and cyclic	Standard construction 2.4 × 2.4 m (8 × 8 ft) wood-frame shear walls were tested using: monotonic, CUREE-Caltech standard (CUREE), CUREE-CalTech near-fault, SPD, and International Standards Organization test protocols
McMullin and Merrick (2002)	Cyclic	Six shear walls of grade CD plywood, OSB and gypsum wallboard, includes tests of different types of drywall screws
Kim (2003); Rosowsky and Kim (2004a, b); Kim and Rosowsky (2005a, b)	Reliability analysis	Develops fragility curves for various SW materials
Langlois et al. (2004)	Static, cyclic	Applied monotonic (ASTM E564) and cyclic (CUREE) testing protocols to SW
Ni and Karacabeyli (2004)	Analytical study	Presents equations for evaluating deflection of unblocked SW and horizontal diaphragms
van de Lindt et al. (2004); van de Lindt and Rosowsky (2004)	Reliability analysis	Tested 12 SW designed and evaluated for reliability with ASCE 16
Seaders et al. (2004, 2009a, b)	Monotonic (ASTM E564), cyclic and earthquake loads	Two sets of tests of eight partially and two fully anchored 2.4 × 2.4 m (8 × 8 ft) SW with 38 × 89 mm (nominal 2 × 4 in.) DF studs at 610 mm (24 in.) o.c. 2 OSB with 8d nails and GWB
van de Lindt (2004)	Literature review	Details 31 SW tests, modeling and reliability analysis
Williamson and Yeh (2004)	SPD (SEAOSC, FME = 3 cm)	SW w/openings (portal frames)
Dean and Shenton (2005)	ASTM E564 modified to exceed design allowable before the final half-cycle	Ten 2.4 × 2.4 m (8 × 8 ft) SW with 11 mm (7/16 in.) OSB and applied vertical load
Lebeda et al. (2005)	Static, cyclic	Thirteen 2.4 × 2.4 m (8 × 8 ft) SW with misplaced hold-downs (CUREE)
White (2005); White et al. (2009, 2010)	Earthquake records	Tested 34 identical 2.4 × 2.4 m (8 × 8 ft) walls of 38 × 89 mm (nominal 2 × 4 in.) kiln-dried DF. Studs were spaced at 610 mm (24 in.) o.c. Half were partially anchored, half fully anchored
Johnston et al. (2006)	Cyclic	Compares effects of vertical load and hold-down placement
Seaders et al. (2004); White (2005); van de Lindt and Gupta (2006); White et al. (2009)	Three SAC response spectra	2.4 × 2.4 m (8 × 8 ft.) SW with 11.1 mm (7/16 in.) OSB and 12.5 mm (1/2 in.) gypsum panels
Leichti et al. (2006)	CUREE	Tested SW with different nail strengths
Mi et al. (2004, 2006)	Monotonic and ASTM E2126	Eight 4.9 × 4.9 m (16 × 16 ft.) SW with 12.5 mm (1/2 in.) plywood
Winkel (2006); Winkel and Smith (2010)	Static	14 tests of shear walls with combined racking, uplift and bending loads
Yasumura et al. (2006)	1940 El Centro	Two-story 3 × 3 × 6 m (9 × 9 × 18 ft.) 7.5 mm (5/16 in.) plywood with openings
McMullin and Merrick (2007)	Monotonic and CUREE-CalTech	11 tests. Discusses seismic damage thresholds for gypsum wallboard
Ni and Karacabeyli (2007)	ISO 16670, ASTM 2126	16 SW w/ diagonal or transverse horizontal lumber sheathing and gypsum sheathing varying hold-downs, vertical load, and width of sheathing
van de Lindt (2008)	Shake-table tests	24 shake-table tests of SW, some w/gypsum, some w/ corner walls
Hart et al. (2008)	Cyclic, varying by author	Discusses 195 drywall and stucco sheathing tests done by APA, Merrick, City of Los Angeles and McMullen and Pardoen for CUREE
McMullin and Merrick (2008)	Cyclic CUREE-Caltech	17 tests w/ screws and nails w and w/o window openings
Sinha (2007); Sinha and Gupta (2009)	Monotonically (ASTM E564)	Tested 16 standard 2.4 × 2.4 m (8 × 8 ft) walls, 11 were sheathed with OSB on one side and GWB on the other, and 5 walls were tested without GWB. Digital image correlation was used for data acquisition and analysis which is a full-field, noncontact technique for measurement of displacements and strains
Zisi (2009)	Monotonic and cyclic w/increasing amplitude	Tested brick veneer on wood-frame walls with OSB and gypsum
Ni et al. (2010)	Monotonic and cyclic (ISO 16670)	Tested 20 configurations of 1.22, 2.44, or 4.88 m long SW with 9.5 mm OSB or 12.7 mm GWB, some 4.88 m SW with a 2.44 opening, some 2.44 m walls with 1.22 or 0.61 m perpendicular bracing walls

Chapter 2

Test Setup

In order to evaluate the performance of wood shear walls subjected to earthquake loads; a proper testing apparatus, loading procedure, data acquisition, and analysis is required. Since there was no testing system for this research already set up in the lab, part of this study was to design a system able to apply pseudo-earthquake loads to standard size residential shear walls. The design of the testing system and assembly of shear wall specimens were followed in accordance with ASTM E564-12 – Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Building. The scope of ASTM E564 states that the practice describes methods for evaluating the shear capacity of a typical section of framed wall, supported on a rigid foundation and having load applied in the plane of the wall along the edge opposite of the rigid support and in the direction parallel to it. For this paper, the anchorage effect of utilizing steel hold-downs which is required design by International Building Code (IBC) is compared to the International Residential Code (IRC) which does not require the use of hold-downs.

2.1 WALL SPECIMEN ASSEMBLY

Shear walls constructed for testing were designed in accordance to the 2015 IRC and IBC. All tests were performed on 8 ft. x 12 ft. walls. Framing members were made up of 2 in. x 4 in. nominal heat treated Spruce-Pine-Fir (SPF) at lengths of 91 in. for the studs and 144 in. for the top and sill plate. Both the IRC and IBC codes require studs to be placed at a maximum of 16 in on center. A total of 12 studs plus one sill plate and a double top plate were used for framing. The chords at the both ends consisted of two studs. Three 4 ft. x 8 ft. oriented strand board (OSB) with 3/8th in. thickness heat treated panels were used as sheathing. The panels were placed as for the grain to run parallel to the studs in order to utilize the strong axis.

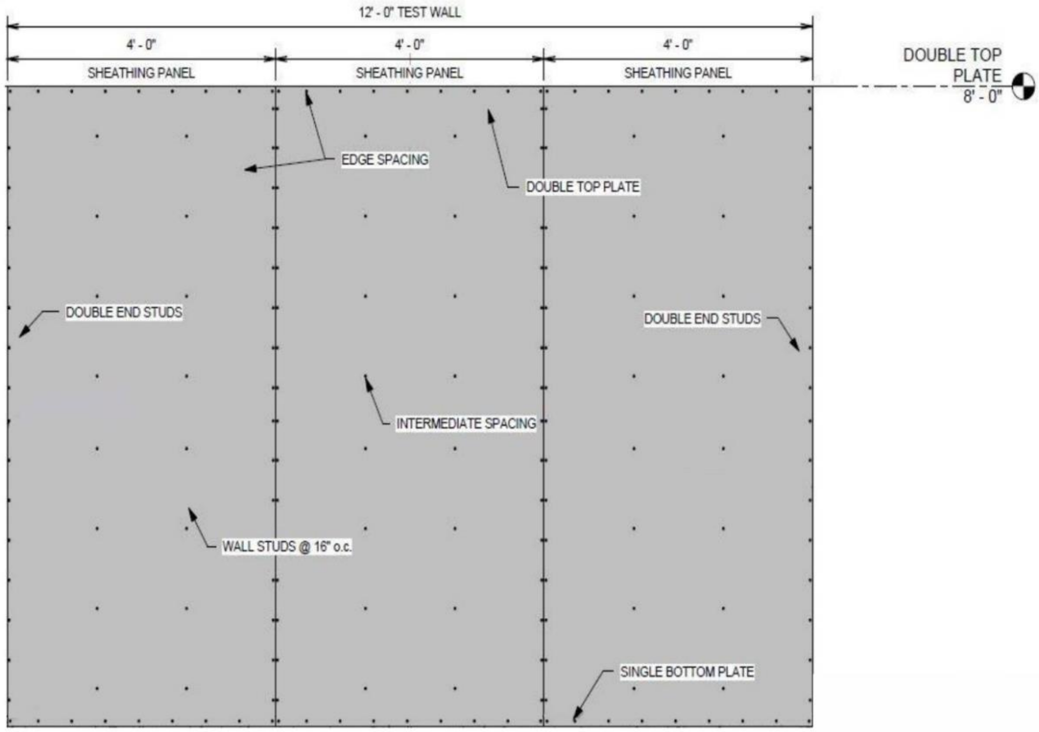

Figure 1. Wall specimen schematic

Nails were in accordance with ASTM F1667- Standard Specifications for Driven Fasteners. Framing nails were full round head smooth shank and sized at 3.25 in. x 0.131 in. Sheathing nails were full round head smooth shank and sized at 2.375 in. x 0.131 in. The perimeter or edge spacing of the sheathing consisted of a 4 in. spacing nailing pattern. Sheathing to stud or intermediate spacing consisted of a 6 in. nailing pattern. The two double end studs were connected using framing nails in. a 4 in. staggered pattern. Two framing nails were used in the top and bottom of each stud to connect the studs to bottom and top plates. Figure 1 shows the nailing pattern and design schematic of the wall. A Hitachi NR90AE 3.5 in. plastic collated full-head framing nail gun was used to pneumatically drive nails.

2.2 WALL ANCHORAGE

Wall specimens were designed under two conditions to evaluate anchorage effects. SIMPSON HTT5 steel hold-down anchors are commonly used in IBC design but not in IRC. For this study, the design of the shear walls were the same except for the addition of the hold-down anchors for IBC shear walls which will be referred to use as fully anchored in this study. A schematic of the two different hold-down patterns is shown below in Figure 2. Partially anchored shear walls refer to IRC design without hold-downs.

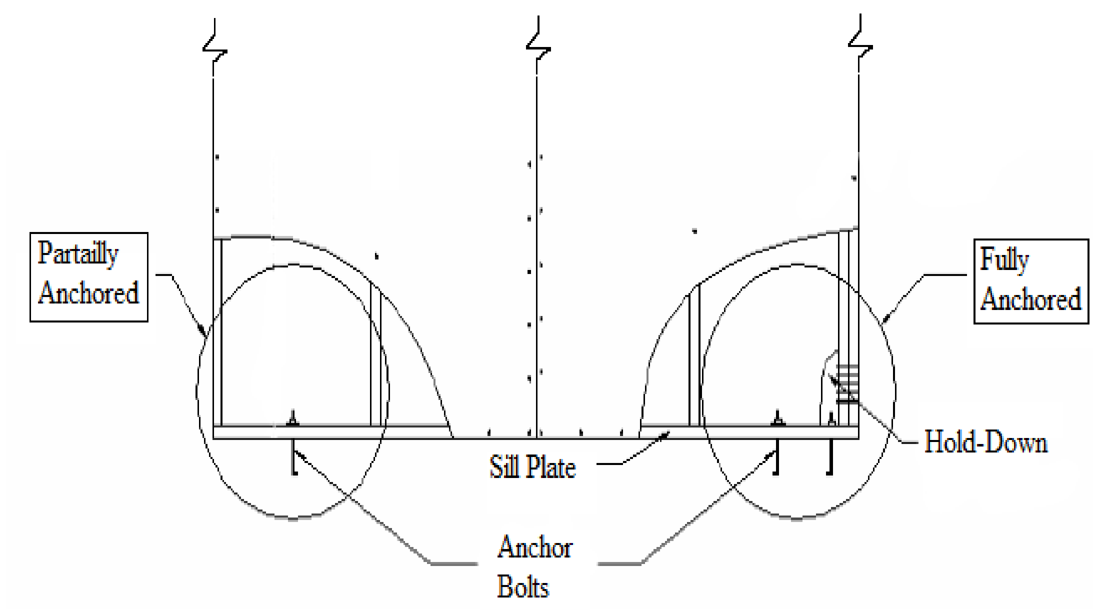

Figure 2. Partially vs fully anchored

2.2.1 PARTIALLY ANCHORED

The more frequently used anchorage design system follows IRC 2012 section R403.1.6. According to this section: "there shall be a minimum of two bolts per plate section with one bolt located not more than 12 in. or less than seven bolt diameters from each end of the plate." All the test setups use; A307 with 0.5 in. diameter anchor bolts placed at 3.5 in. from the inside studs and 48 in. on center (a total of five bolts per wall). All of the anchor bolts are threaded to the foundation beam which is anchored to the concrete foundation. This replicates equivalent anchorage conditions that would be seen in typical residential wood frame houses. This anchorage system is considered to be partially anchored. Figure 3 shows a picture of a completely constructed partially anchored wall ready for testing.

Figure 3. Partially anchored shear wall ready for testing

2.2.2 FULLY ANCHORED

The IBC designed wall specimens were tested under fully anchored conditions. These walls are considered fully anchored due to the addition of SIMPSON HTT5 Strong-Tie at each double stud chord element. The HTT5 strong-tie is 16 in. x 2.5 in. made from galvanized 11-gauge steel that weigh 2.17 lb. each. Each hold-down is connected to the double stud with 26 framing nails in a two row staggering pattern and connected to the foundation with an anchor bolt. The location of the anchor bolt from the hold-down is 1 inch from the inside of the double stud. A close up picture of the SIMPSON hold-down in place after construction can be seen in Figure 4. Figure 5 shows a picture of a completely constructed fully anchored wall.

Figure 4. Simpson HTT5 Hold-Down

Figure 5. Fully anchored shear wall ready for testing

2.3 TESTING FRAME AND EQUIPMENT

Testing of the shear wall specimens was conducted at University of Memphis' Department of Civil Engineering Structural Research Laboratory. Figure 6 shows the design drawing of the test set up for the monotonic loading. The main components of the set up are the strong-frame, actuator, bottom foundation beam, top loading beam, the in-plane restraint system, and shear wall specimen. A steel bottom beam with threaded mount points for anchor bolts is used to simulate the connection to foundation. The top load beam is connected to the actuator at the front of the wall with five ¾ inch diameter bolts and distributes the load to the wall. 2 sets of rollers are used, each set at 12 inches from each end of the wall and restrain the wall to in-plane motion only. A 10 kip hydraulic actuator capable of a 10 inch stroke was utilized to simulate earthquake load by pushing and pulling the top beam horizontally. Figure 6 shows the design layout of the testing system which consists of:

1. Strong Frame
2. Actuator
3. Load Beam
4. Shear Wall
5. In-Plane Restraint System
6. Bottom Foundation Beam

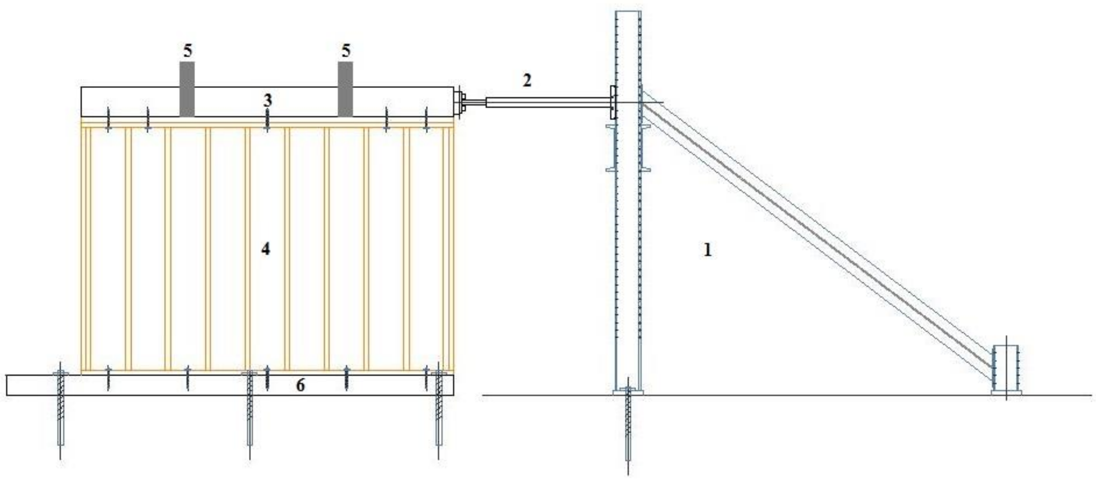

Figure 6. Layout of Shear Wall Testing System

2.3.1 STRONG FRAME

The whole testing system depends on the strong frame's ability to withstand the reaction forces created during loading. It is crucial to have a rigid strong frame that allows no racking during a test so the data obtained is accurate. The strong frame consists of W10x49 columns and beams. One column is in plane with the shear wall and is connected to the actuator and there are two columns (behind the wall in Figure 6.) offset laterally from the wall that are used to connect the in-plane restraint system. The strong frame is anchored to the concrete foundation.

2.3.2 ACTUATOR

For testing, the load is applied to the shear the wall with a 92E Series Single Ended Shore Western actuator. It is a fatigue rated actuator that is ideally suited for static to low-cycle structural testing. Figure 7 shows the actuator which is capable of a 10 inch stroke and a force rating of 10 kips. A 28 in. x 10 in. x 1 in. steel plate connects the actuator to the strong frame with four ¾ inch diameter bolts. A strap connected to the in-plane restraint system above the actuator is used to support the cantilevered end. The cantilevered end is bolted to the load beam which is on top of the shear wall. Both of actuator's connection to the strong frame and to the load beam are pinned, free to rotate. The actuator is allowed to angle itself vertically but it restricted to in-plane movement from the in-plane restraint system which is discussed in section 2.3.4. Figure 8 shows the setup of connections of actuator to the strong frame and shear wall.

Figure 7. 92E series single ended Shore Western actuator

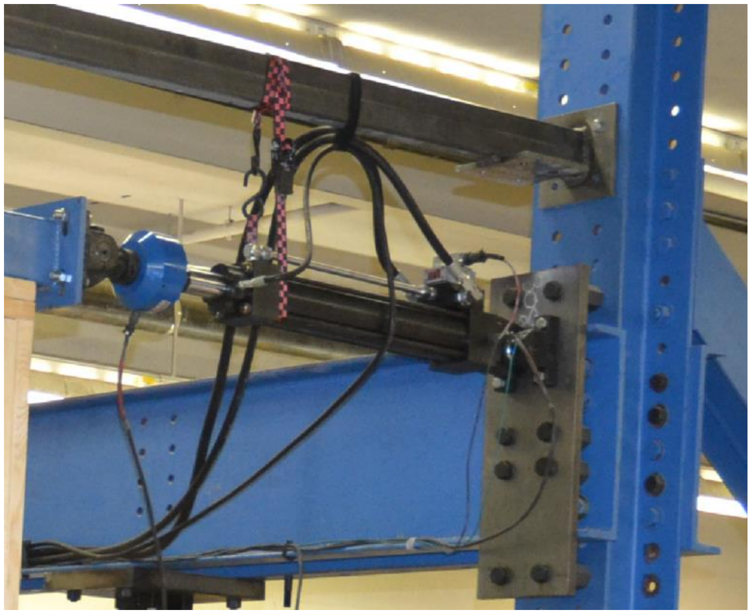

Figure 8. Actuator connections

2.3.3 LOAD BEAM

In order to distribute the load from the actuator to the shear wall, a 14 ft. HSS3x5 load beam was utilized. Five anchor bolts of ½ in. diameter spaced evenly were used to connect the load beam to the shear wall. A 5 in. x 5 in. slotted steel plate was welded at the end to connect the actuator. The self-weight of load beam was the only dead load applied to the shear wall. As discussed later this in turn is a conservative design compared to having a higher dead-load force which is normally seen in practice. There is a welded handle in the middle of the beam, this is to allow for the crane to hook on and lift the shear wall in to place for testing.

2.3.4 IN-PLANE RESTRAINT SYSTEM

In order to simulate how a real residential wood shear wall would react during an earthquake event, displacement should be restricted to in-plane only. In practice, a shear wall would have laterally supporting members such as a perpendicular wall at the end where a corner of a house would be, therefore in-plane displacement would not be found in a real life scenario.

An in-plane roller system was designed to restrict the wall from out of frame deformation during a test. Two sets of rollers are cantilevered from adjacent columns to laterally support the load beam. The load beam sits on top of the two top plates of shear wall. Rollers were chosen because they provided the least amount of friction as a support if the wall came in contact with them and allow for movement in the in-plane direction only. Figure 10 shows a drawing of the restraint system. This in-plane restraint system allows the wall to react how it would normally in a real-life scenario.

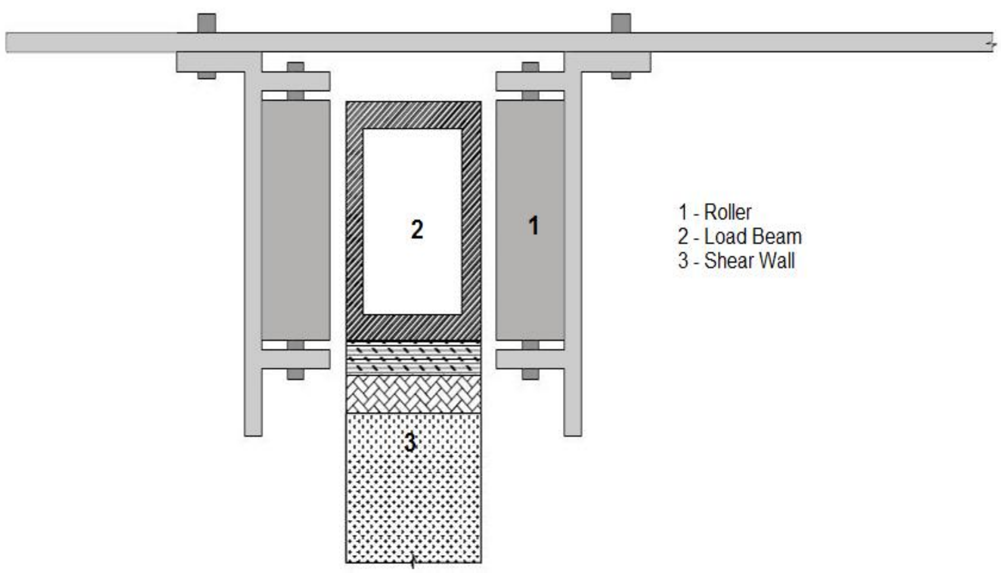

Figure 10. Schematic of in-plane restraint system

Chapter 3

Monotonic Loading Procedure

Shear walls were subjected to cyclic loading under ASTM E564 - 06 (2012) "Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings." No dead loads were applied except for the self-weight and the weight of the top loading beam. The shear wall strength values are considered more conservative because in practice, the shear wall would have a roof applying more force as a dead load which contributes to resisting overturn. ASTM E564 describes methods for evaluating the shear capacity of a typical section of a framed wall, supported on a rigid foundation and having load applied in the plane of the wall along the edge opposite of the foundation. The objective of ASTM E564 is to provide a determination of the shear stiffness and strength of any structural light-frame wall configuration to be used as a shear-wall on a rigid support (ASTM E564).

3.1 NUMBER OF TESTS REQUIRED

According to ASTM E564, a minimum of two shear wall assemblies must be tested to determine the shear capacity of a given construction. If the strength or shear stiffness of the second test is not within 15 % of the results of the first test, a third wall assembly with the wall oriented in the same manner as the weaker of the two test values should be tested. This is the reason for an uneven number tests performed. The partially anchored shear walls did not fall within 15% in the first two attempts so a third test was conducted that satisfied ASTM E564.

3.2 LOADING PROCEDURE

Loads were applied laterally and to the top of the wall, in-plane of the frame, using the hydraulic actuator. For the static pushover test, ASTM E564 states that loads shall be applied at a constant rate of displacement to reach the limiting ultimate displacement in no less than 5 minutes. To meet this requirement, a constant rate of displacement of 1.0 in./min. is utilized.

Chapter 4 Data

Acquisition

One of the goals of this study is to create a framing system that can subject shear walls to earthquake loads and collect accurate data. Having proper instrumentation that collects reliable and accurate data consistently is necessary for any experimental research. For this study, multiple displacement and loading force points are measured according to ASTM E564 and can be seen in Figure 11. These parameters are used in quantifying the shear strength and stiffness of the wall diaphragm.

4.1 LOAD MEASUREMENT

The only force measured is the horizontal shear force being applied from the actuator to the top of the wall. This force is used to for creating a load-displacement curve needed in order to evaluate shear strength parameters of the wall. A 40 kip rated load cell built into actuator head reads the horizontal force applied to the shear wall.

4.2 DISPLACEMENT MEASUREMENTS

One of the main parameters needed in evaluating shear wall performance is the horizontal shear displacement. This is the wall racking deformation from a result of the load being applied. Shear displacement measurement of the wall was recorded to within 0.01 in. which is in compliance with ASTM E564. Calculating accurate shear displacement can be complicated by the fact that the wall assembly tends to rotate and translate as a rigid body. During a test, the loading transforms the wall from a rectangular to a skewed parallelogram shape. There are two approaches provided by ASTM E564 when estimating the shear deformation: direct measurement or by diagonal elongation of the frame.

Direct measurement is based off the four displacement points seen in Figure 11. For this method, data acquisition instruments are used to directly measure slip at the base (1), uplift (2), top plate horizontal displacement (3), and vertical displacement (4). These four data points give the necessary information to calculate the horizontal shear displacement of the wall.

Measurement of the diagonal elongation of the frame is the second approach provided by ASTM E564. Horizontal shear displacement is calculated on the basis of the diagonal elongation of the wall which simplifies the test by eliminating the need to measure rigid body rotation and horizontal translation of the wall. Figure 12 illustrates the calculation of the horizontal shear displacement using diagonal elongation.

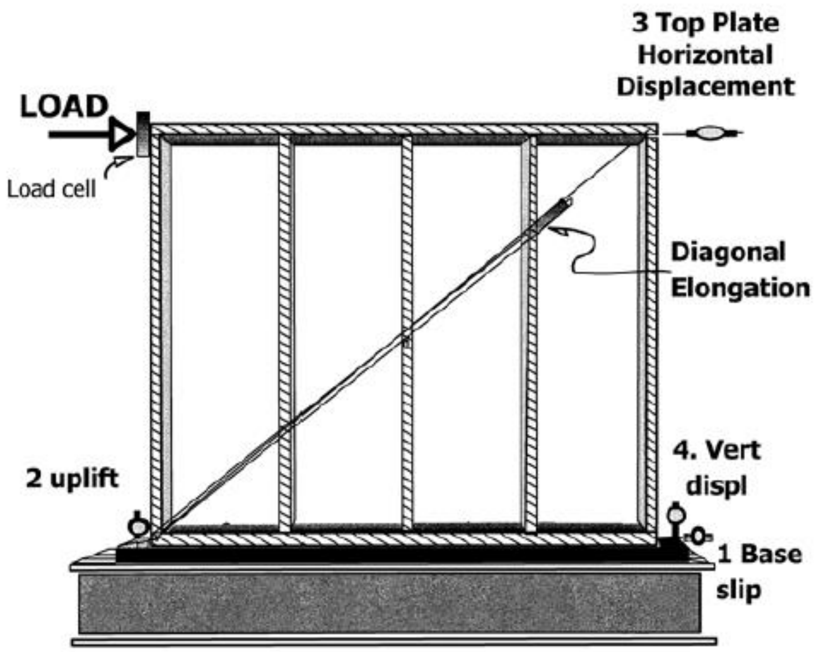

Figure 11. ASTM E564 Data Points (ASTM E564)

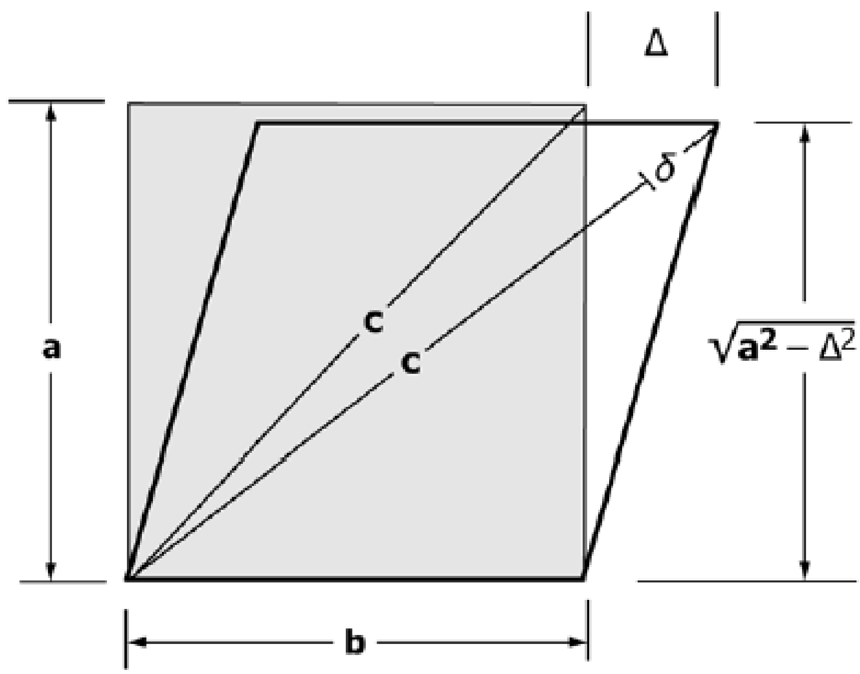

Figure 12. Horizontal Shear Displacement (ASTM E564)

$$(C+\delta)^2=(b+\Delta)^2+(a-\Delta^2) \quad (1)$$

Substituting: $$a^2+b^2=c^2 \quad (2)$$

gives: $$2c\delta^2+\delta^2-2b\Delta=0 \quad (3)$$

and: $$\Delta=\frac{(2c\delta+\delta^2)}{2b} \quad (4)$$

Both of these techniques are considered contact methods in which instrumentation such as dial gauges or piezometers are attached to the specimen. This type of method is used for load data acquisition where the actuator comes in contact with the wall. A non-contact technique is used for acquiring displacement data.

4.3 INSTRUMENTATION

Data was collected through two different techniques as stated before. The first method was utilizing the actuator's linear variable differential transducer (LVDT) to record horizontal displacement and the load cell attached at the head of the actuator to record applied force. The second method was a non-contact technique using a camera and a digital image correlation (DIC) software called Ncorr which was used analyze the pictures taken of the wall to measure displacements of the entire wall.

A non-contact method was sought out after trial runs of using piezometers to acquire data resulted were damaged during loading. A LVDT long enough to cover a 12ft. x 8ft. diagonal was not ideal either so a digital imaging correlation was used as a non-contact method to acquire displacement data.

4.3.1 ACTUATOR

The load cell recorded the force being applied to the load beam from the actuator while LVDT recorded the horizontal displacement at the top of the wall. This displacement value corresponds to point 3 in Figure 11. The force recorded from the actuator's load cell was calibrated with DIC data and used in the load-displacement curve discussed in the analysis section. A data acquisition system attached the actuator stored the recorded data.

4.3.2 NCORR

The other method used to collect data was a non-contact method using digital imaging correlation to calculate displacements. Ncorr has been used in different fields of research for geotechnical and biomedical applications, wood joints, and crack tip experiments. It has also been verified against other commercial DIC packages by R Harilal and M Ramji (2014). In their study, a comparison of displacement and strain fields obtained by Ncorr and VIC-2D (a commercial DIC software) was performed. The results in Figure 13 demonstrates an accurate agreement for displacement values found between Ncorr and VIC-2D.

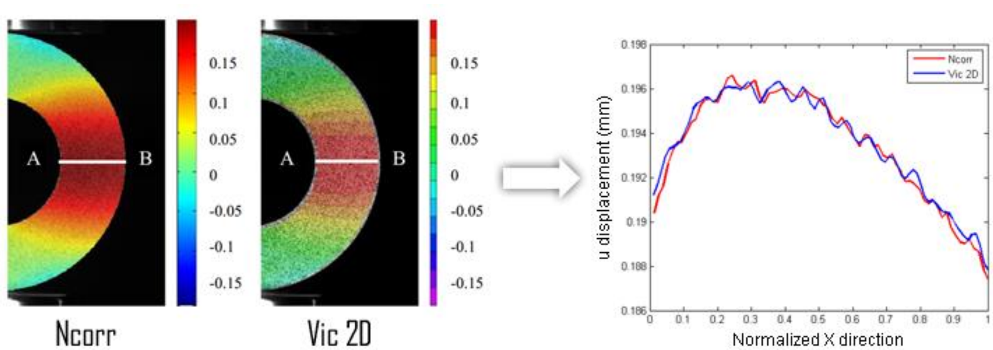

Figure 13. Ncorr and VIC-2d comparison (Harilal 2014)

This non-contact method was performed by taking photos of the wall with a Nikon D7000 16.2 MP camera. During a test, a picture would be taken before the test began as a reference image and at the local maximum displacement points of the loading pattern. Ncorr V1.2, an open source 2D digital image correlation MATLAB software, was used to analyze the photos taken during the test and was able to display the total wall deformation.

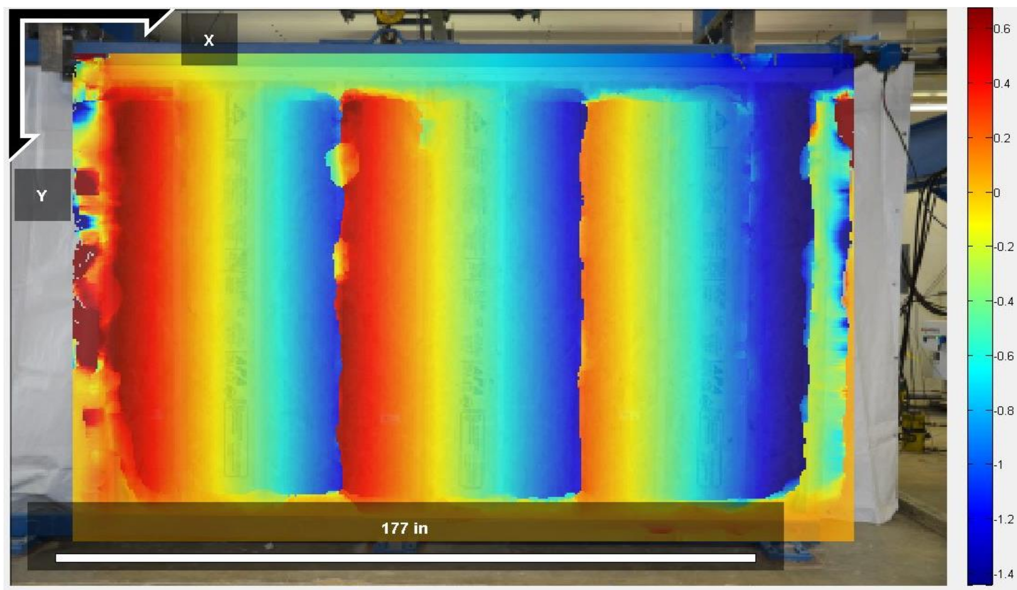

Figure 14. Horizontal field displacement results found by Ncorr

One of the advantages to using this method is that it displayed the deformation of the entire wall in one picture, giving a better understanding of how the wall reacts locally and globally. Ncorr produces horizontal and vertical displacement fields that are color coordinated referring to the corresponding displacement calculated. This allows for a more in depth evaluation by being able to visually see how the panels, studs, and plates are performing during a test. Ncorr also calculates the true horizontal shear displacement and vertical uplift without having to account for rigid body movement as discussed before. Figure 14 shows the horizontal and vertical displacement field results found by Ncorr after analysis.

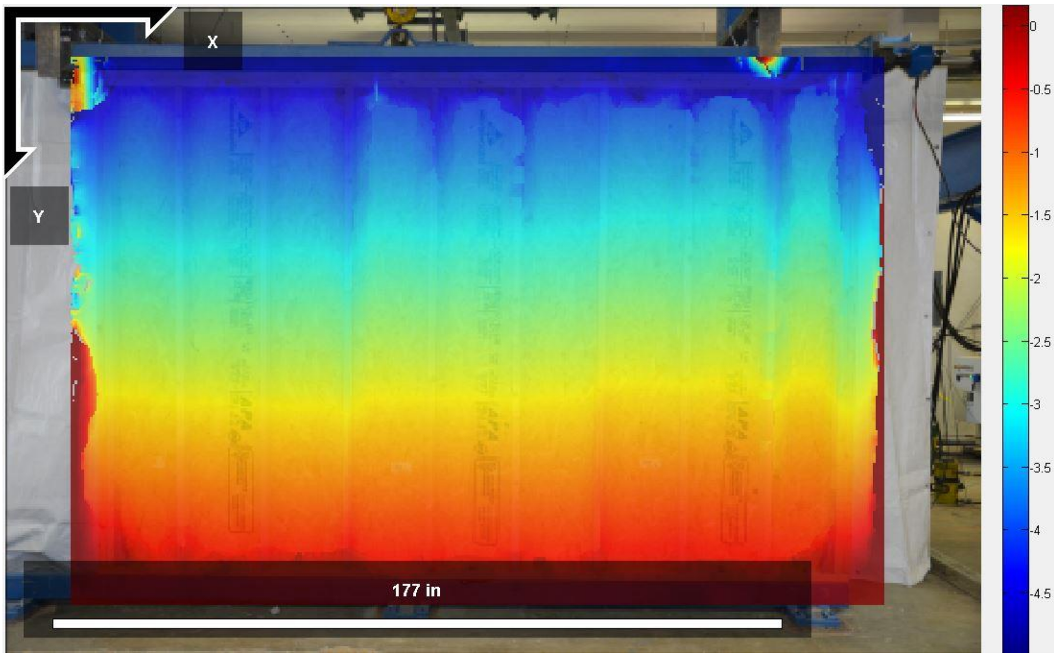

Figure 14. Horizontal field displacement results found by Ncorr

As it can be seen the software analyzes the whole wall for displacements, therefore there is no need to for gauges or any contact data acquiring devices. This also gives the ability to perform both direct measurement and diagonal elongation method if one wanted. Ncorr can provide the true horizontal displacement of the shear wall without any extra calculations. This value is used to calibrate the data from the actuator to obtain the true horizontal shear deformation for the load-displacement curve. This is important because as will be discussed later on, the load-displacement curve is the main data used in evaluating the shear walls for earthquake performance parameters.   A more detailed description of the Ncorr analysis process used to measure shear deformations is discussed in the Analysis section.

Chapter 5

Analysis

5.1 Earthquake Strength Parameters

The purpose of the tests performed is to evaluate earthquake strength parameters of the wood shear wall as the lateral force resisting system under different code anchorage conditions. These parameters include shear stiffness, shear strength, ductility, and toughness.

Shear stiffness is the resistance of the shear wall to deformation in the elastic range before the first major event is achieved. First major event (FME) is defined by ASTM as the point at which the specimen enters the inelastic range. Shear stiffness can be expressed as a slope measured by the ratio of the resisted shear load to the corresponding displacement. It is important for the shear wall to have adequate stiffness to resist smaller and more frequent cyclic forces. For a structure that is subjected to smaller more frequent earthquakes, the structure should not have any significant damage or exceeds the serviceability limit state. Displacements and inter-story drifts are less than the damage threshold and structure is able to continue operation with proper shear stiffness (Fajfar et al. 2004).

Shear Strength is the capacity of the shear wall against load resistance and can be expressed as the initial yielding strength recorded after the first major event is achieved. Shear strength is a key parameter in order for the shear wall to resist earthquake load adequately. It is considered when designing for medium to high magnitude earthquakes due to the higher forces created. Damage is limited to non-structural elements with no significant damage to structure but repairs may be needed. For example, a window might break or brick veneer may fall but structural components are not damaged.

Ductility and toughness are parameters that express the ability of the shear wall to deform beyond the elastic range. Ductility can be calculated as the ratio of the peak displacement to yield displacement. Toughness describes the ability of a structure to sustain excursions in the nonlinear range without critical decrease in strength. Toughness is calculated as the area under the load-displacement curve. Toughness can be considered to be the amount of energy required to fail a system. These factors are considered when designing for large magnitude earthquake with regard to life safety of the occupants. Collapse prevention is the main goal, keeping plastic deformation and rotation less than ultimate capacity. Structural damage is limited to that which can be repaired economically. Displacement and drifts are limited to avoid instability (Fajfar et al. 2004).

The earthquake strength parameters of shear stiffness, shear strength, and toughness will be further defined using the load-displacement envelope and the equivalent energy elastic-plastic curve.

5.2 LOAD-DISPLACEMENT CURVE

The load-displacement curve is utilized for evaluating the earthquake strength parameters of the shear wall. According to ASTM E564, inter-story drift which is found by taking the difference of the top displacement minus the displacement of the bottom plate should be used for generating load-displacement graphs.

Base slippage was recorded by Ncorr and applied to the top displacement found by the actuator to calculate inter-story drift.

Load-displacement curves are different for monotonic and cyclic tests. In this study, monotonic graphs for this study are considered always positive and produce data characteristic in one direction and for constant loading. Figure 14 shows an example of a typical monotonic load-displacement. Cyclic tests produce what is called a hysteresis loop and is made up of two distinct curves, one is positive for pushing and the other half is negative for pulling of the actuator and consists of many loops corresponding to the loading pattern used. An envelope curve is produced by connecting the peak loads of each displacement cycle and can be compared to the monotonic curve. For this study, only monotonic loading will be applied to shear walls.

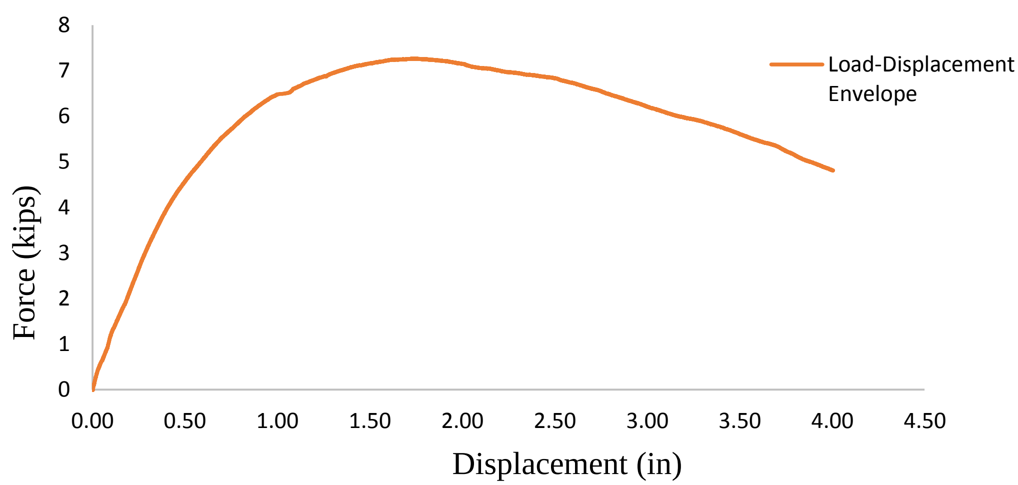

Figure 14. Typical monotonic load-displacement envelope curve

5.3 EEEP CURVE

Wood shear walls have a significantly different load-displacement curve than most construction materials. Unlike steel which can exhibit a close to perfect elastic-plastic response, wood does not display a distinct yield load. The determination of the yield point has been contentious and a wide range of definitions and methods to determine yield for wood structures has been introduced (Foliente 1996). In this study, the yield load was determined by deriving the equivalent energy elastic-plastic (EEEP) curve from the load-displacement data. The EEEP curve is defined by ASTM E2126 as an ideal elastic-plastic curve circumscribing an area equal to the area enclosed by the envelope curve between, the ultimate displacement (Δ_u), and the displacement axis. For monotonic tests, the observed load-displacement curve is used to derive the EEEP curve. Figure 15 provides an example of the parameters needed to create an EEEP curve.

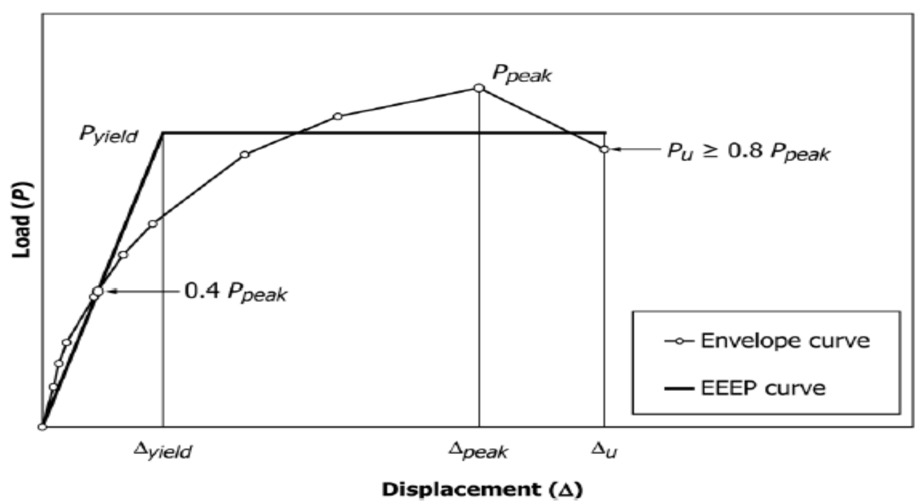

Figure 15. ASTM E564 EEEP curve

The EEEP curve allows for a more accurate representation of the elastic-plastic response of the wall by approximately calculating the energy dissipated by the wall. Energy dissipated is measured by calculating the area under the load-displacement curve and is a key parameter in finding the yield strength of the wall. Using the area under the load-displacement curve as a measure of the energy dissipated by the structure is a key concept of the EEEP method (Lawless 2014). For monotonic testing, the area under the load-displacement curve found is used. The elastic portion of the EEEP curve contains the origin and has a slope equal to the elastic shear stiffness (K_e). The plastic portion is a horizontal line equal to the yield load (P_{yield}).

5.3.1 ELASTIC SHEAR STIFFNESS

Elastic shear stiffness (K_e) is defined by the slope of the secant passing through the origin and 40% of the peak load (P_{peak}) on the load-displacement curve. This slope defines the elastic portion of the EEEP curve and is also used to determine other parameter calculations discussed later. The elastic stiffness is a good representation of the stiffness that a wall would exhibit when subjected to low to moderate displacements (Salenikovich 2000). Elastic stiffness is determined as:

$$\mathrm{K}_e = \frac{0.4\mathrm{P}peak}{\Delta 0.4\mathrm{P}peak} \tag{4}$$

5.3.2 YIELD LOAD AND DISPLACEMENT

The plastic portion of the EEEP curve is defined by the horizontal line equal to the yield load and extends until failure of the specimen. The yield load, P_{yield}, is calculated based off the load-displacement curve in order for the EEEP curve to have to the same area under the curve as the envelope curve, this is a key factor for the Equal Energy Method. The yield load, P_{yield}, is a function of elastic stiffness, toughness, and failure displacement and is calculated by ASTM E564:

$$P_{yield} = \left(\Delta_u - \sqrt{\Delta_u^{\ 2} - \frac{2A}{K_e}} \right) K_e \tag{5}$$

If $\Delta_u^{\ 2} < \dfrac{2A}{K_e}$, it is permitted to assume $P_{yield} = 0.85\,P_{peak}$

Where:

P_{yield}	=	yield load, kips
A	=	the area under envelope curve from zero to ultimate displacement (Δ_u) of the specimen, kip-in.
P_{peak}	=	Maximum absolute load resisted by the specimen in the given envelope, kips
Δ_u	=	Displacement of the top edge of the specimen at ultimate displacement, in.
Δ_e	=	Displacement of the top edge of the specimen at $0.4P_{peak}$, in.
K_e	=	$\dfrac{0.4P_{peak}}{\Delta_e}$, in.

Once P_{yield} is determined, the yield displacement or FME can be estimated as:

$$\Delta_{yield} = \frac{P_{yield}}{K_e} \tag{6}$$

5.4 NCORR

The digital image correlation software Ncorr was used to analyze the shear wall displacements. An optical measurement instrument was utilized to capture grey scale images of the wall at local maximum and minimum displacements while the wall is being subjected to the loading pattern. After testing of a specimen, the recorded images were analyzed by Ncorr to obtain displacements.

In order for the software to calculate deformation at any point, a reference subset of pixels from the wall before loading was utilized. This subset consists of a unique light intensity pattern in which Ncorr searches for the best matching pattern in the deformed specimen images. Once it matches the subset in deformed specimen, mathematical correlation is used to calculate displacements and a full field displacement view of the wall is obtained.

Ncorr provides a horizontal and vertical displacement field graphs of the shear wall. These graphs can be used to read the four displacement points needed for direct measurement method or to measure the diagonal elongation as discussed in the data acquisition section. It is also used to verify the displacement data recorded by the actuator because the actuator cannot account for base slip.

Chapter 6

Monotonic Test Results

In this study, a total of 2 fully and 3 partially anchored monotonic tests were performed. Only 2 of the 3 partially anchored walls were used for data evaluation as the third did not fall within the 15% deviance requirement by ASTM E564. All specimens were constructed with 12 ft. x 8 ft. dimensions and tested according to ASTM E564 (2012). A comparison of IRC and IBC design code for wood shear walls were performed using the shear wall testing system. The main difference between the two codes is the use of steel hold-down anchors which are required by IBC. These hold-downs are placed at the bottom corners of shear walls and connected to the double ended studs with 26 nails and to the foundation with A307 ½ in. diameter bolt. Recall that shear walls with the hold-down are considered "fully anchored" and "partially anchored" without hold-downs. Fully anchored walls are designed for residents living in higher seismic risk zones to prevent the shear wall from pulling out of the foundation. Pull out of the studs is the typical failure mode seen for shear walls subjected to earthquake loads without hold-downs. Simpson Strong Ties are manufactured to hold the end columns to the foundation preventing this pullout failure. This study compares the effects of fully anchored shear walls using Simpson Strong Ties to partially anchored shear walls. Figure 16 presents the average load-displacement curve found for fully and partially anchored shear walls subjected to monotonic loading.

The data presented is based on the average of the tests that fall within 15% of each other. According to ASTM E564, if fewer than 10 tests of a single shear wall configuration is used, the evaluation of the wall performance characteristics are based on the mean values found.

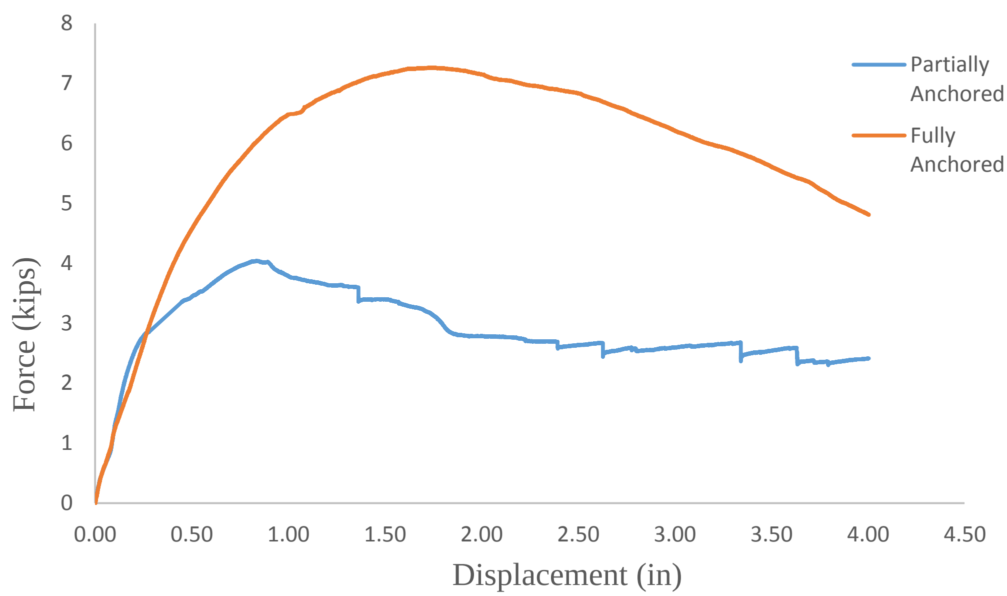

Figure 16 Average Load-Displacement Curve for Fully and Partially Anchored Shear Walls under monotonic loading

6.1 PEAK LOAD AND DRIFT

The peak load is taken as the maximum recorded force on the load-displacement curve and is used in allowable stress design method to determine design values with a factor of safety applied. Peak drift is the corresponding displacement value found at the maximum load.

Table 1. Peak load and drift values

		P_{peak} (kips)	Δ_{peak} (in.)
Partially Anchored	Test 1	4.02	0.46
	Test 2	4.3	0.59
	Average	4.16	0.53
Fully Anchored	Test 1	7.32	1.62
	Test 2	7.28	1.67
	Average	7.3	1.65

In Table 1, it can be observed that the average maximum load for the partially anchored shear walls was found to be 4.16 kips and 7.30 kips for fully anchored walls. The peak displacement values were found to be 0.53 and 1.65 in respectively. The addition of the 2 Simpson Strong Tie increased the maximum load by 75% and peak displacement by 311%.

6.2 ELASTIC SHEAR STIFFNESS

The elastic shear stiffness is the ability of the shear wall to resist shear deformation of a specimen in the elastic range and before the first major event is achieved. As stated before, the elastic stiffness is the secant line passing through the origin and calculated as the ratio of the load to displacement at 40% of the peak load, $0.4P_{peak}$. Elastic stiffness is a key parameter to account for when designing for earthquake loads. Adequate stiffness is required where smaller more frequent earthquakes occur where the structure would be more likely performing in the elastic range. Table 2 lists the value of elastic shear stiffness, as defined by ASTM E2126, for the monotonic test.

Table 2. Elastic shear stiffness values

		$0.4P_{peak}$ **(kips)**	$\Delta_{0.4Ppeak}$ **(in.)**	K_e **(kips/in.)**
Partially Anchored	Test 1	1.61	0.09	17.87
	Test 2	1.72	0.07	24.57
	Average	1.67	0.08	21.23
Fully Anchored	Test 1	2.93	0.19	15.41
	Test 2	2.91	0.29	10.04
	Average	2.92	0.24	12.73

Partially anchored shear walls recorded an average elastic shear stiffness of 21.23 kips/in while fully anchored was found to be 12.73 kips/in. This is interesting as it was assumed that the fully anchored shear walls would have a greater elastic shear stiffness from the addition of the hold-downs. The higher stiffness exhibited by the partially anchored walls could be contributed to the way ASTM calculates elastic shear stiffness. ASTM calculates the stiffness as the ratio of load to displacement at 40 % of the peak load. The values for the corresponding load and displacement for fully and partially anchored shear walls tested are shown in Table 2.

In Table 2, it can be seen that the average 40% of the peak load for fully anchored walls is less than twice of partially anchored but the corresponding displacement is 3 times as much. This is because the addition of the hold-downs allows for greater shear capacity and energy dissipation which can be observed by the large area under the load-displacement curve for fully anchored walls. Partially anchored walls have less area under the load-displacement curve and does not absorb as much energy. This results in the 40% displacement for partially anchored walls to be much smaller than the corresponding fully anchored wall which then provides the higher elastic shear stiffness values.

The ASTM E2126 stiffness values can be misleading when comparing partially and fully anchored walls if one is not familiar with how they are derived. For this study, it is noted that the average elastic shear stiffness values for the points found on the load-displacement curve were computed to take the total average of all the points up to 40% of the peak load. This allows for a better representation of the elastic region for fully anchored walls. This method resulted in an average elastic shear stiffness of 18.93 kips/in. for fully anchored walls and 21.33 kips/in. for partially anchored walls.

6.3 ULTIMATE LOAD AND DISPLACEMENT

As expected, fully anchored shear walls exhibited a larger force capacity when compared to partially anchored walls. The ultimate load was calculated according to ASTM E2126 as the point on the load-displacement curve once the shear wall had lost 20% of its peak force capacity. Table 3 lists the results from the monotonic tests.

Table 3. Ultimate load, displacement, and area values

		P_u (kips)	Δ_u (in.)	A (kip-in.)
Partially Anchored	Test 1	3.22	1.75	5.87
	Test 2	3.44	1.25	4.01
	Average	3.33	1.5	4.94
Fully Anchored	Test 1	5.87	3.37	20.78
	Test 2	5.89	3.14	18.59
	Average	5.88	3.26	19.69

The ultimate displacement is the corresponding value on the load-displacement curve when the ultimate load is reached. This is an important parameter as it provides the boundary condition when calculating the energy absorbed by the shear wall. The lateral forces that are exerted on a structure and transferred to the shear wall produce large amounts of energy that must be absorbed in order to avoid failure (Toothman 2003). The energy absorbed and dissipated by the shear walls is measured by the area under the curve from the origin to the ultimate displacement. The calculated values can be seen in in Table 3.

In Table 3, it is observed that fully anchored walls dissipate five times as much energy than partially anchored walls. The addition of two hold-downs provides a significant increase in strength and energy dissipation. The reasoning for this increase is based on how the hold-downs change the failure mechanism of the shear wall. The main failure mechanism for partially anchored walls was observed to be pullout of the studs from the foundation. The largest uplift pullout displacement was found to be at the two end studs. The main problem with this failure type is that it does not allow for efficient use of the sheathing panels which are there to support against shear force. The studs pullout and fail before sheathing can contribute any additional support. The addition of two hold-downs in each corner of the shear wall prevents the end studs from pulling out. This changes the failure mechanism of the shear wall from stud pullout to sheathing and nail pullout. Because the studs cannot pullout from the foundatin, the load paths next step is the sheathing. The additional support from the sheathing provides the shear wall with a larger shear force capacity. Figure 17 shows a comparison of fully anchored and partially anchored shear wall behavior using averaged test data.

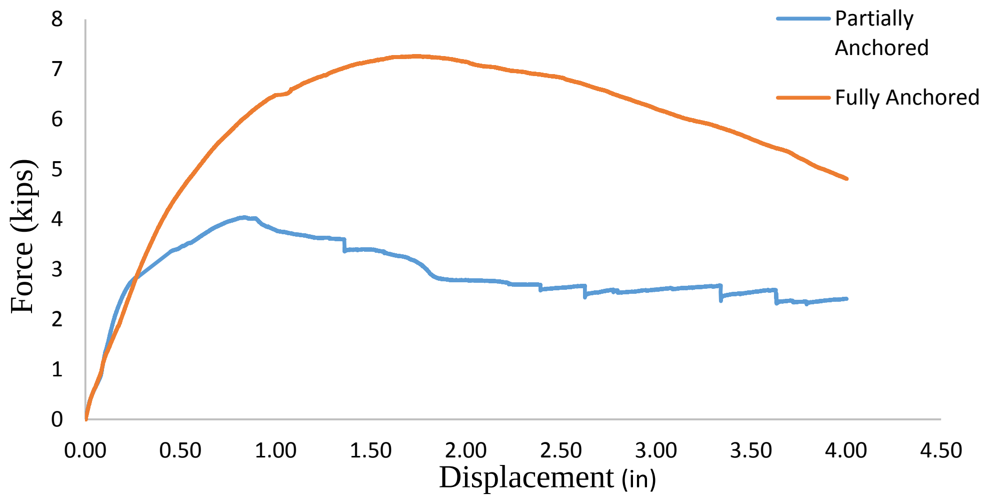

Figure 17. Average load-displacement curve from monotonic testing

6.4 YIELD LOAD AND DRIFT

Due to wood's complex strength characteristics and irregular elastic-plastic response, the EEEP curve method is utilized to determine the yield strength of wood shear wall (ASTM E2126). The yield strength of EEEP curve, defined in Equation 5 is a function of elastic stiffness, ultimate displacement, and energy dissipated. The yield load determined in this study is an approximation of the first major event (point at which the shear wall starts to deform in the inelastic region).

Fully anchored shear walls exhibited an average yield load of 6.61 kips with the first major event occurring at 0.59 in. Partially anchored shear walls provided an average yield load of 3.48 kips with the first major event occurring at 0.17 in.

Figure 18 and 19 shows EEEP curves and parameter values computed from average test data for fully anchored and partially anchored shear walls, respectively.

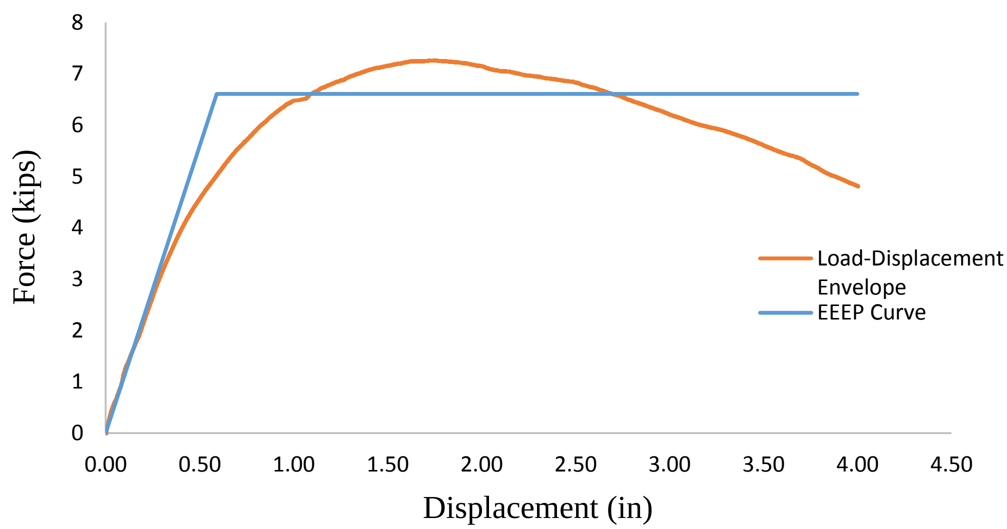

Figure 18. Average EEEP curve parameters for fully anchored wall

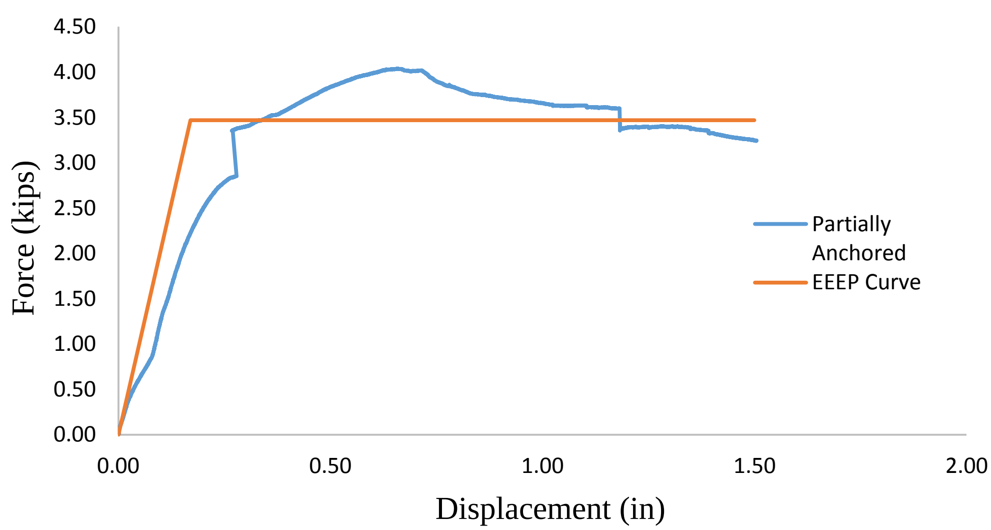

Figure 19. Average EEEP curve parameters for partially anchored wall

The addition of the two hold-downs allows the shear wall to have an increased yield load compared to the partially anchored walls without hold-downs.

Since the two hold-downs prevent the main failure mechanism of stud pullout, the wall is allowed to withstand higher forces by utilizing additional shear strength in the shear strength. When stud pullout occurs, the sheathing panels are not allowed to provide their full shear strength potential. This can be seen in the envelope curve in Figure 19. The partially anchored envelope curve has sudden loss of strength in portions of the load-displacement curve which can be seen in Figure 19 at a displacement of around 1.25 in. This occurs when the pullout force on the stud resulting from the shear force is increasing, the studs will slowly start to separate from the bottom plate. The main resisting force to this pullout failure is from the two framing nails connecting the bottom plate to studs. Once the stud pull out the full length of the framing nails, the capacity has a sudden decrease in the strength until the force follows the load path to the next stud.

6.5 FAILURE MODES

There were two main modes of failure found from the test. Partially anchored shear walls consistently failed at the bottom sill plate from either stud pullout or from sill plate splitting. Figures 20 and 21 show pictures of stud pullout and sill plate failure modes exhibited by partially anchored walls, respectively.

The failure modes found are consistent with that found by Seaders et al. (2008). In their findings, partially anchored walls also failed from stud pullout and fully anchored walls failed at the sheathing from nail pullout.

Figure 20. Stud pullout from bottom sill plate failure mode

Figure 21. Bottom sill plate splitting failure mode

Pullout of studs is the most common failure mode seen in houses after an earthquake and what the Simpson Strong Ties are designed to resist. Fully anchored walls were observed to have sheathing and nail slip as the main failure mechanism. The Simpson Strong Ties restrict the studs from pulling out so the load is redirected towards the sheathing and nails. When this occurs, the shear force is carried thru the nails to the sheathing and when the nails slip out or tear out, the sheathing is no longer effectively attached to the frame and the wall loses shear capacity. Figure 22 show typical failure mode of nails slipping or pulling out of the sheathing.

Figure 22. Sheathing failure due to nail slip

Chapter 7

Summary and Conclusion

7.1 Summary

A shear wall testing apparatus designed to subject full scale residential wood shear walls was designed and constructed for this study. A total of five tests were performed on 12 ft. x 8 ft. wood shear walls. All walls were tested using monotonic loading procedures at a constant rate of 1.0 in./min. Shear walls were tested under two anchorage conditions, fully and partially anchored. The purpose of the tests was to evaluate the effect of earthquake loads and anchorage effects of residential wood shear walls. Data analysis involved calculation of earthquake performance parameters such as capacity, yield strength, elastic stiffness, and energy dissipation.

7.2 CONCLUSIONS

Overall, the addition of the Simpson Strong Ties produced a dramatic change in the overall behavior and performance of the shear wall as expected. By comparing average test values found from monotonic testing of fully and partially anchored walls, it is apparent that fully anchored shear walls produced a large increase in the load carrying capacity, shear stiffness, and energy dissipation.

The main reason for the difference in performance of partially and fully anchored wood shear walls is how the load is distributed. Partially anchored walls typically fail from stud pullout and sill splitting which does not allow for full use the sheathing shear strength. Hold-downs prevent stud pullout failure from occurring which allows for sheathing shear capacity to be used more efficiently.

The behavior of fully vs. partially anchored shear walls subjected to monotonic loading is listed:

1. Fully anchored walls exhibited an average yield load of 6.61 kips with the first major event occurring at 0.59 in. Partially anchored shear walls exhibited an average yield load of 3.48 kips with the first major event occurring at 0.17 in.
2. Fully anchored walls provided a peak capacity of 7.30 kips at 1.65 in. drift. Partially anchored walls provided a peak capacity of 4.16 kips at 0.53 in. drift.
3. Average energy dissipated by fully anchored shear walls was found to be 19.69 kip-in. Average energy dissipated by partially anchored shear walls was found to be 4.94 kip-in.
4. Fully anchored walls exhibited an average ultimate load of 5.88 kips at an ultimate drift of 3.26 in. Partially anchored walls exhibited an average ultimate load of 3.33 kips at an ultimate drift of 1.50 in.
5. Fully anchored wall's load-displacement curve was smooth without sudden jumps due to nail pullout which was frequently seen in partially anchored shear wall's load-displacement curve.
6. Recommendation of changing ASTM E216 calculation of elastic shear stiffness to be based off average up to 40% instead of at 40% of peak load
7. The main failure mechanism for partially anchored walls was pullout of the studs from the bottom sill or splitting of sill. The main failure mechanism for fully anchored walls was sheathing failure to due nail slip

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