SHEAR WALL SILL PLATE BEHAVIOR IN WOOD FRAMES

By

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FRAMES

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Abstract: In the Midwest region of United States of America tornados are considered to be the most brutal form of natural disaster for the building houses where many of these houses are built out of wood. Newcastle-Moore tornado that happened in 2013 severely damaged a high number of houses in its path. Damage to a single component of a structure can cause the whole building to collapse. Failure of garage doors lets the wind enter the building and changes building envelope to a partially enclosed building where walls are subjected to uplift force that eventually cause the wall or sill plate to fail in connections. Additionally, the overturning moment in partially anchored shear walls put uplift forces on the bottom chord.

In this report, a summary review have been done to determine the failure causes for damaged houses that were observed by post tornado assessment team in Moore 2013. A further in-depth study of sill plate behavior in wood frames is a core focus of this report. Sill plate failure mode found throughout this and previous studies describes most of the actual failures that happened on the site during the Moore, 2013 tornado.

The sill plate failure modes can be described in three distinct scenarios. 1) Failing along bottom face due to bending moment from the sheathing to the connection 2) failure along the edge that is due to high tensile stress perpendicular to grains. 3) Failure of connections that can be between anchor bolts and foundation or sheathing to sill plate nailing. All these modes were tested in the lab, and the outcome results give the idea that a few critical factors in construction can significantly change failure modes. These factors are noticed in washer size, pith orientation, nailing spacing, bolt distance from sheathing and presence of a layer of the metal plate beneath the sill plate. The larger washer size or close bolt position reduces the moment arm and changes the failure from bending on the bottom face to failure along the edge. Pith orientation significantly affects the bending capacity of the sill plate. Metal plate at the bottom face of sill plate prevent the failure in bending and enhances the overall capacity. Four test configurations observed and analyzed in this study. Each pair consists of the plate with and without a metal connector. The first pair was in ideal condition while the two other was applied maximum possible bending stresses. It was determined that overall capacity is higher when bolts are at the center of the plate and metal plates at the bottom. This enhancement can give up to 30% more strength to the sill plate. However, in maximum bending stress metal plate acted more efficiently and increased the capacity by 60% more, but the overall applied loads were less than those with bolts at center. Conclusion and recommendation in this report are to place bolts at center, and metal plate connector on the bottom face to get the maximum capacity out of sill plate.

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CHAPTER I

INTRODUCTIONS

In this research, the primary purpose of this study is shear wall sill plate behavior in wood frames. To find out what factors affect in failure modes of the sill plate. Additionally, how to enhance the capacity of sill plate subjected to uplift forces.

The idea and concept for this topic came from the incident, Newcastle-Moore Tornado, which happened in the city of Moore in May 2013. Many houses were damaged, and some other were fully collapsed.

Chapter I of this report, discuss the importance of wood structures, natural loads on building structures and failure and damage of houses in Moore 2013 tornado.

Chapter II is detailed studies of the shear wall and sill plate, failure modes of the sill plate and factors affecting the failure mode as well review of previous works done in this area.

Chapter III is more focusing on theoritical approach for failure mode capcity, testing materials and procedures, setups and configurations for tests of the sill plate in the lab. Furthermore, metal plate connectors are also provided in these test.

Chapter IV presents the test results and discusses comparison between samples. While in Chapter V conclusion and recommendations from this research is discussed.

Appendices contain additional details that could not fit in above five chapters.

1.1 Wood Structures

Wood shelter is among the primary materials in the early ages used by humankind to protect themselves from the environment. Later timber frames used by ancient civilization for wider purposes. The history of early usage of long timber frames goes back to 500 to 1000 B.C. used by Egyptians and Romans. By the end of 500 A.D, the usage of wood structure buildings increased significantly in Europe, and the main reason was further development in construction and availability of wood in the area. However, the development of wood was not only limited in Europe since fabulous timber structures were built in North East Asia at the time as well (Blue Ridge Timberwright, n.d.).

During the colonization of Europeans in America, timber structures increased in number in the United States. There are many wooden monuments built by British in America. Later in the mid-18th century by developing new tools in the wood industry the smaller pieces of wood, lumbers, produced and the usage of wood as building materials gone skyrocketing. Later, in 1970s wood was considered as environmental friendly material in building industry (Blue Ridge Timberwright, n.d.). Wood construction in housing is most favorable in America due to the vast availability of wood material, easy construction techniques and its classic look.

1.2 Loads on Building Structures

All building structures must undergo gravity and lateral loads. Gravity loads are in the direction toward the ground due to gravity pull and lateral loads, horizontal loads, are caused by an earthquake or the wind. When it comes to gravity loads, there are very precise tables to estimate loads, but for lateral loads it mostly depends on many other factors that are classified based on regional and environmental conditions. Buildings are designed for lateral loads based on potential extreme load either wind (tornadoes, hurricane) or earthquake whichever controls the ultimate load combination. In the United States, west coast regions are designed for the earthquake, the east coast for hurricane and Midwest for extreme tornadoes load.   The following figure 1-1 & figure 1-2 are a good indicator of natural hazardous based on regions.

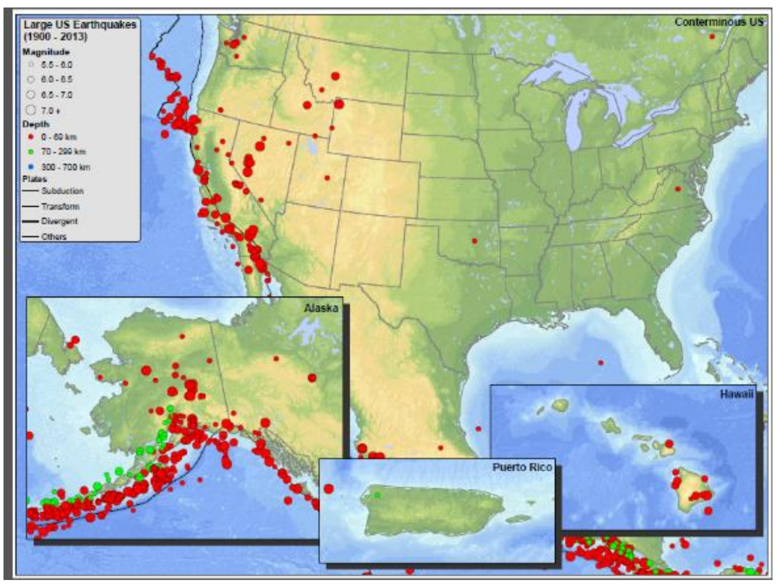


Figure 1-1 Earthquake Regions in the United States (U.S Geological Survey)

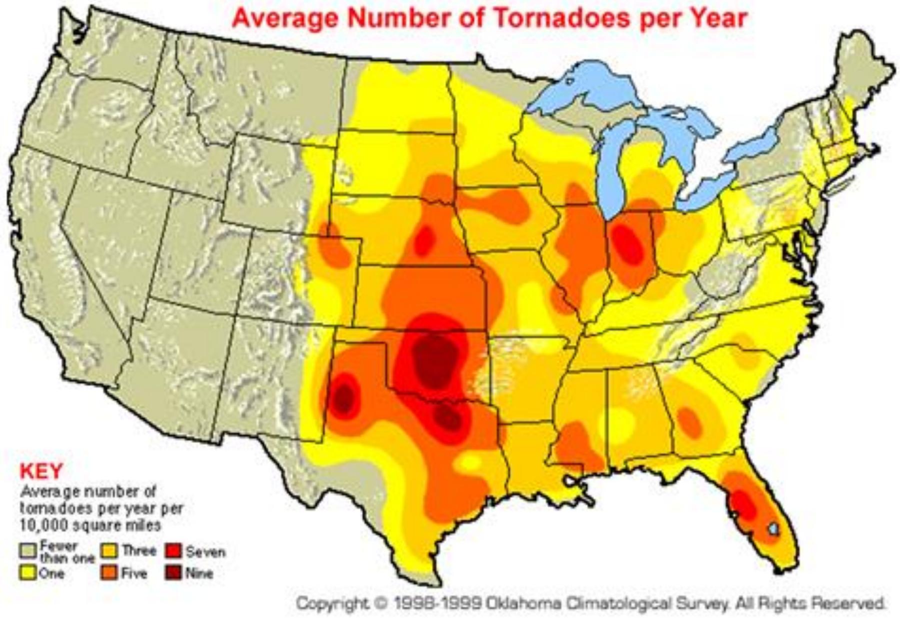


Figure 1-2 Map of Tornado occurance frequency across united states (Oklahoma Climatological Survey)

1.2.1 Tornadoes

Tornadoes are the circulating form of moving air at a very high speed where in some occasions it exceeds 500km/hr. (310mph). Tornadoes happen from the collation of warm, moist air with cold air where intense heating of earth surface develops updrafts in a severe thunderstorm. Tornadoes consolidate air to make enormous pressure that is approximately 800 mill bars and it is considered to be very high pressure on many man-made objects. It is enough to rip off any light weight building structure. There are few thing need to be estimated to determine tornadoes damage scale, how fast, how wide, how long the path as well how long does it last? It is broken in 5 degrees which are classified according to Fujita tornado intensity scale. Table 1-1 gives a detailed description of each scale.

Table 1-1 Fujita tornado intensity scale.

F-Scale	Category	Kilometers per Hour (Miles per Hour)	Comments
0	Weak	65-118 (40-73)	Damage is light. Chimneys on houses may be damaged; trees have broken branches; shallow-rooted trees pushed over; some windows broken; damage to sign boards.
1	Weak	119-181 (74-112)	Shingles on roofs blown off; mobile homes pushed off foundations or overturned; moving cars pushed off roads.
2	Strong	182-253 (113-157)	Considerable damage. Roofs torn off houses; mobile homes destroyed; train boxcars pushed over; large trees snapped or uprooted; light-objects thrown like missiles.
3	Strong	254-332 (158-206)	Damage is severe. Roofs and walls torn off better constructed homes, businesses, and schools; trains overturned; most trees uprooted; heavy cars lifted off ground and thrown some distance.
4	Violent	333-419 (207-260)	Better constructed homes completely leveled; structures with weak foundation blown off some distance.

5	Violent	420-513 (261-318)	Better constructed homes lifted off foundations and carried considerable distance where they disintegrate; trees debarked; cars thrown in excess of 100 meters.

Tornadoes are happening in most locations on earth, but the United States is where most of them happen, and the main reason is its location in between Mexican Gulf and cold northern region. In the United States, tornadoes occur between late spring and summer season. Figure 1-3 depicts the frequency of storm in the United States.

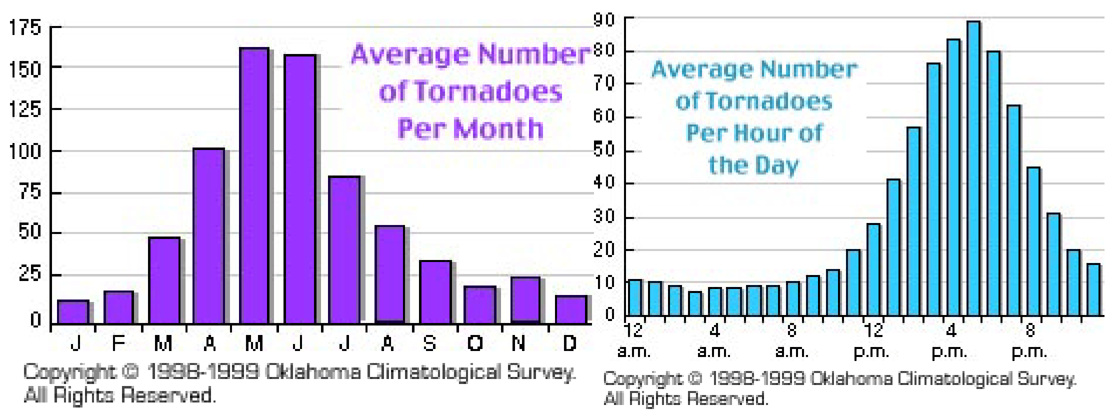


Figure 1-3 a)Average Number of Tornadoes Per Month b) Average Number of Tornadoes Per Hour of the Day (Oklahoma Climatological Survey)

In the United States, the highest number of tornadoes occurs in the Midwest region. Table 1-2 collected the most brutal form of tornadoes happened in the last century.

Table 1-2 Ten deadliest tornado events in the United States.

Date	**Location(s)**	**Deaths**
March 18, 1925	Missouri, Illinois, Indiana	689
May 6, 1840	Natchez, Mississippi	317
May 27, 1896	St. Louis, Missouri	255
April 5, 1936	Tupelo, Mississippi	216
April 6, 1936	Gainesville, Georgia	203
April 9, 1947	Woodward, Oklahoma	181
April 24, 1908	Amite, Louisiana and Purvis, Mississippi	143
June 12, 1899	New Richmond, Wisconsin	117
June 8, 1953	Flint, Michigan	115
May 11, 1953	Waco, Texas	114

The Oklahoma States is very famous in tornadoes worldwide. This state has the most record of tornados touchdown that caused severe damages in the past. All scale from F1 to F5 have been recorded in Oklahoma. Records show that on average nearly 45 tornadoes occurs annually in the state of Oklahoma (Webmaster, 2015). The most recent severe tornado occurred on 20th may 2013 and entitled as Newcastle-Moore tornado where the scale of intensity reached F5 and left behind a very high number of fatalities and damages.

1.3 Newcastle-Moore Tornado

Based on Tornado Damage Assessment in the aftermath of the May 20th, 2013 Moore Oklahoma Tornado Report On May 20th, 2013 city of Moore was evidence of a brutal tornado. The scale of this wind damages goes to F5 with the high speed of 210 miles per hour and traveled 17 miles across rural farmlands and created a 1.3mile wide swath of destruction. This 40 minutes long tornado left enormous chaos behind that cost the city $2 billion losses in the economy and took 24 lives and injured nearly 350 of the resident. This incident is considered the most brutal one after the one happened in 1999 Bridge Creek-Moore Tornado, which recorded 317 miles per hour speed. With the newly developed method of design and construction and safety precautions, the number of fatalities is dropped significantly but the damage and economic losses are still growing up.

The May 20th tornado reported a damage of 12,000 homes and 33,000 people were displaced or severely affected by the incident. Tornado path is shown in figure 1-4. Typical damages to residential structures occurred failure at the garage door opening, roof and the connections.

Figure 1-4 Tornado Path during 1999, 2003 & 2013 Tornadoes in Moore City, Oklahoma (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

1.3.1 Moore Tornado Failure Progression in Residential Structures

(Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014) Findings described destruction levels of the Tornado. Tornado leaves damage to any structure where it comes on its path. In what scale the building is damaged depends on what zone it locates. To classify damage level of tornadoes, it is divided into three bands as shown in figure 1-5.

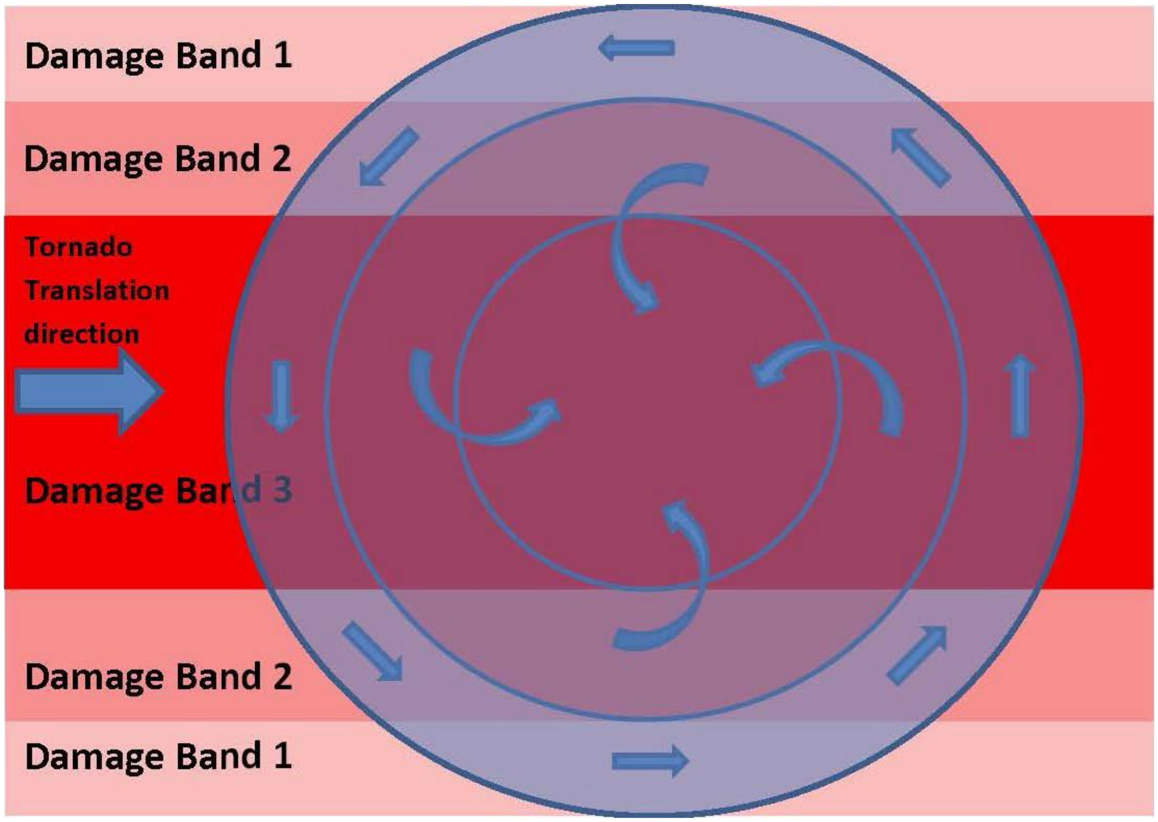


Figure 1-5 Layers of Tornado divided in band classes (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

The outer band, Band 1, is the least intense pressure from the tornado, and in this band only horizontal wind pressure is applied to structures, more similar to hurricane winds. Buildings in this zone experience horizontal pressure on wind direction, in wood houses, mostly garage doors goes undergo the windward pressure and roof sheathings fails on leeward wind pressure.

Figure 1-6 Failure of Garage Doors (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

The band 2 is the location in between the core and outer band, and its characteristics contain both horizontal and uplift pressures. The horizontal wind pressure cause damage to the garage door in windward direction and the horizontal leeward wind plus the wind uplift pressure cause the roof sheathing to fall apart. Figure 1-7 is a good indicator of this band.

Figure 1-7 (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

The most dangerous and extreme wind pressure is in the center of the tornado where it is named band 3. In this radius, there is absolute horizontal and uplift wind pressure. Unlike two other bands in this area, there is wind pressure in all direction of an object. Horizontal and uplift pressure is in its ultimate magnitude and causes severe damage to the structure. It fully destroys the garage door, and the whole building will be gone.

1.3.2 Failure of Residential Houses in Moore Tornado

Building structures goes under wind pressure based on its shape. If a building is more open to the wind, an uplift wind pressure is created to the roof of structures. It means there is no uplift force to building unless the wind is not entering the building through the openings. Wood frame houses during the Moore tornado leads to this conclusion that the wind first damaged the garage door then the roof and at the end the walls.

The roof structure is providing lateral supports to the walls. If the roof is destroyed or removed from the walls then the wall itself buckles and damaged. On the other hand, the existence of roof in uplift pressure pulls the walls upward and tries to fracture the sill plate that is attached to the foundation. This report will discuss further and in-depth fracture mechanics of sill plate in upcoming chapters.

In the (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014) report based on damaged houses and lab tests, it is found that there are four likely failure of wood houses during tornado wind pressure. The mechanism of failures is depended to other structural elements. Failure of garage door cause roof destruction and lack of roof lateral support cause walls to buckle and damage.

1. Failure of light-gage metal garage doors. Particularly on garages that extend out from the house. This type of garages led to pressurization on the roof, subsequently causes loss of a roof over the garage, and then to the collapse of the garage walls.

Figure 1-8 Light Grage Door Failure (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

Once the garage is gone, then the inside of building is more vulnerable to the wind. The building envelope changes from enclosed building to partially enclosed building (See Figure 1-9) where a combination of positive and negative pressure from inside and outside cause damage to the roof and side walls.

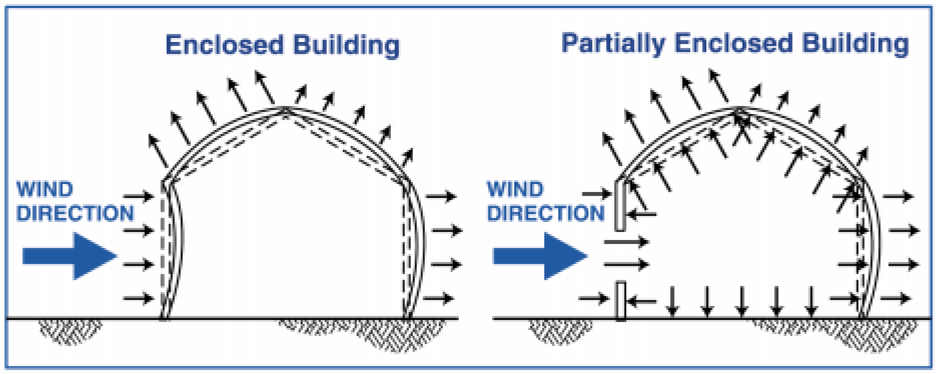


Figure 1-9 Building Envelope (FEMA, 2015)

Damage to the roof can cause lateral disability to walls as roof structure is acting lateral bracing to the top of the wall. Figure 1-10 describes how the garages are destroyed, and walls are ripped-off together.

Figure 1-10 (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

In figure 1-12 the wall was anchored by sill plate to the concrete foundation. The Anchorage is useful to resist uplift forces, but not torsion plus uplift, this is the worse condition to fail in wall foundation. Further discussions on sill plate will be discussed in details in Chapter II.

Figure 1-12 (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

Figure 1-11 (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

2. Prefabricated wooden roof trusses resisted uplift better than Rafter/ridge-beam roofs. However, it is only true for low-speed winds e.g. hurricanes. Figure 1-13 is showing two roofs where one is made of rafter/ridge beam and the other from prefabricated trusses. The prefabricated left in a better condition in compare to other. The truss provides lateral support to top of the wall and rescues the wall from collapse.

The better performance of prefabricated trusses is because of the rigid connections.

Figure 1-13 (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

3. Removal of roofs supported by prefabricated roof trusses left the tops of walls with little lateral support. On the other hand, removal of rafter/ridge-beam roofs typically left the separate ceiling joists in place, and thus the tops of the walls were still laterally supported. In the high-speed wind, it is favorable because prefabricated trusses are gone at all while Rafter/ridge beams still exist but in poor condition. Figure 1-14 shows the ceiling.

Figure 1-14 (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

4. “High profile” roofs had relatively long unsupported spans (compared to other roof types) leaving them more vulnerable to uplift. Fewer intermediate supports led to larger bending moments and shear forces in the rafters. Longer distances between lateral support to the rafter bottom edge (compression edge for uplift) decreased resistance to lateral torsional buckling of the rafters.

Figure 1-15 (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

1.3.3 Foundation Failure of Houses in Moore Tornado

Shear wall is the main structural component in wood building houses that carries both gravity and lateral loads. A detailed sketch of shear wall is shown in figure 1-16

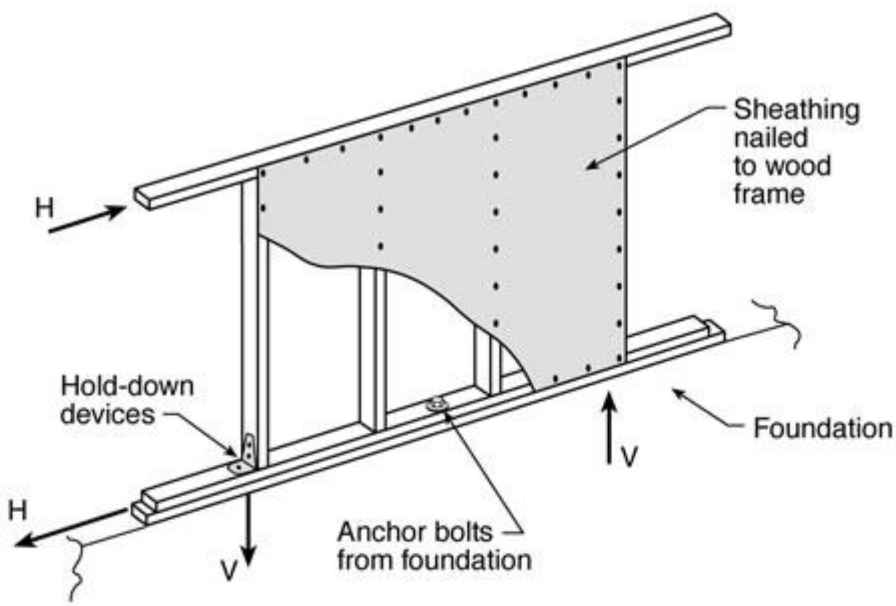


Figure 1-16 (Rainer & Karacabeyli, 2000)

The bottom plate also called sill plate, is attached by anchor bolts and nails to the concrete foundation. Design construction of bottom plate is crucial for the whole walls stability eventually for whole building. The topic of this research is to study in depth the failure of the sill plate. For now we focus on how sill plate and anchor bolts failed in Moore tornado. Figure 1-17 is detailed drawing of sill plate elements.

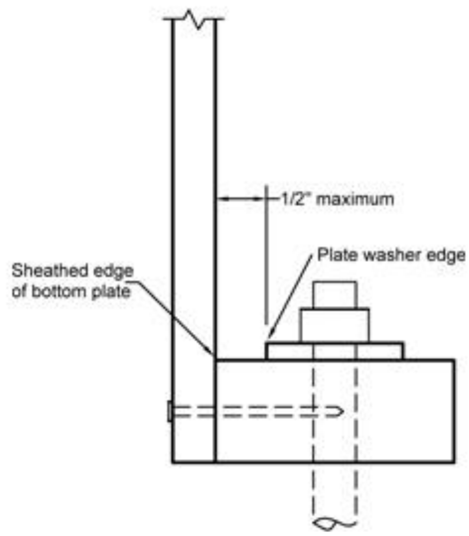


Figure 1-17 Sill Plate Connection Details

A wall to sill plate can fail in three ways.

1. Sheathing to sill plate failure due to lack of bond between wood pieces and nailing.
2. Failure of sill plate itself, this can happen due to exceeding loads than wood capacity. The piece of timber splits apart and then wall collapse.
3. Failure due to the anchor bolt. Anchor bolts primary responsibility is to resist the lateral loads exerted by the wall. It is shear capacity defines anchor bolt capacity. However, in case of tornado uplift pressure, anchor bolt washer resist the uplift pressure.

The assessment team in the 2013 Moore tornado did not find a house that was shifted from its foundation while remained intact as in figures 1-18. “FEMA (1999) stated that residential structures built in Moore were required to meet design requirements listed in the one and two-family dwelling building code published by the Council of American Building Officials (CABO). However, houses built prior to 1995 were governed by a less restrictive building code.”

Figure 1-18 House shifted off of foundation in Joplin, MO after 2011 Tornado. *(Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)*

Figure 1-19 House shifted off of crawlspace foundation in Tuscaloosa, AL after 2011 Tornado. *(Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)*

When the wind speed exceeds 125-135 mph, the weight of uplift pressure is increased, and this cause the building to put more pressure on the sill plate. Once the uplift pressure exceeds the building weight, then the horizontal force of wind pushes the building to slide from the foundation. On the other hand, the uplift pressure breaks apart the washer on top of the anchor bolts. Once it is gone, then the wall can easily slide and leave the foundation within very less magnitude horizontal force. This scenario could have happened in the past, as anchor bolts were found without washer and nuts in the area. (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014).

The post-tornado assessments photos show all failure scenarios mentioned above. It is observed that some houses were destroyed, and all walls collapsed, but the sill plate was remained undamaged. This type failure occurs when the failures are in cases 1 and 3. Figure1-20

Figure 1-20 Undamaged Sill Plate (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

Case scenario one is also observed in Moore 2013 tornado. This scenario happens when the sheathing to sill plate fails. There can be many reasons for failure the size and spacing of nails as well thickness of plywood. In this case, the sill plate is in its best's condition, and bolts are not deformed and damaged. Figures 1-21 and 1-22 are the photos taken from Moore 2013.

Figure 1-21 Damaged Sill Plate (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

Figure 1-22 Broken Sill Plate Along the Edge (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

Figure 1-23 Undamaged Sill Plate, Failure of Sheathing to Sill Plate (Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado, 2014)

It has also been observed that the whole sill plate is gone, but the bolts are remained on its plate, but it is deformed, or nothing happened at all. When the plate splits then, anchor bolts can come in action to resist the loads. The split can be to the bottom of plate or side attached to sheathing. Figures 1-24 and 1-25 are photos taken in Moore 2013.

Figure 1-24

Figure 1-25

The objective of this research is to find out what factors cause the failure in these houses and study in depth sill plate behavior in uplift pressure. The following chapters covers in details about sill plate behavior in different

CHAPTER II

LITERATEURE REVIEW

2.1 Loads in Building Structures

Building houses must resist both gravity and lateral loads. The load path defines how the load distributes throughout the building. FEMA (HOME BUILDER'S GUIDE TO COASTAL CONSTRUCTION, 2010) raise four key issues in load path concept in structural building.

- Loads acting on a building follow many paths through the building and must eventually be resisted by the ground, or the building will fail.
- Loads accumulate as they are routed through key connections in a building.
- Member connections are usually the weak link in a load path.
- Failed or missed connections cause loads to be rerouted through unintended load paths.

The distribution of vertical loads in a wood house follows the traditional "post-and-beam" concept. In case of lateral loads it is different, there is a various approach to resisting lateral load in a building. The most three acceptable systems are Moment Frame, Vertical Truss (braced frame) and Shearwall. (Breyer, Fridley, & Cobeen, 2007). Figure 2-1 describes how loads are transferred to ground in a wood building house.

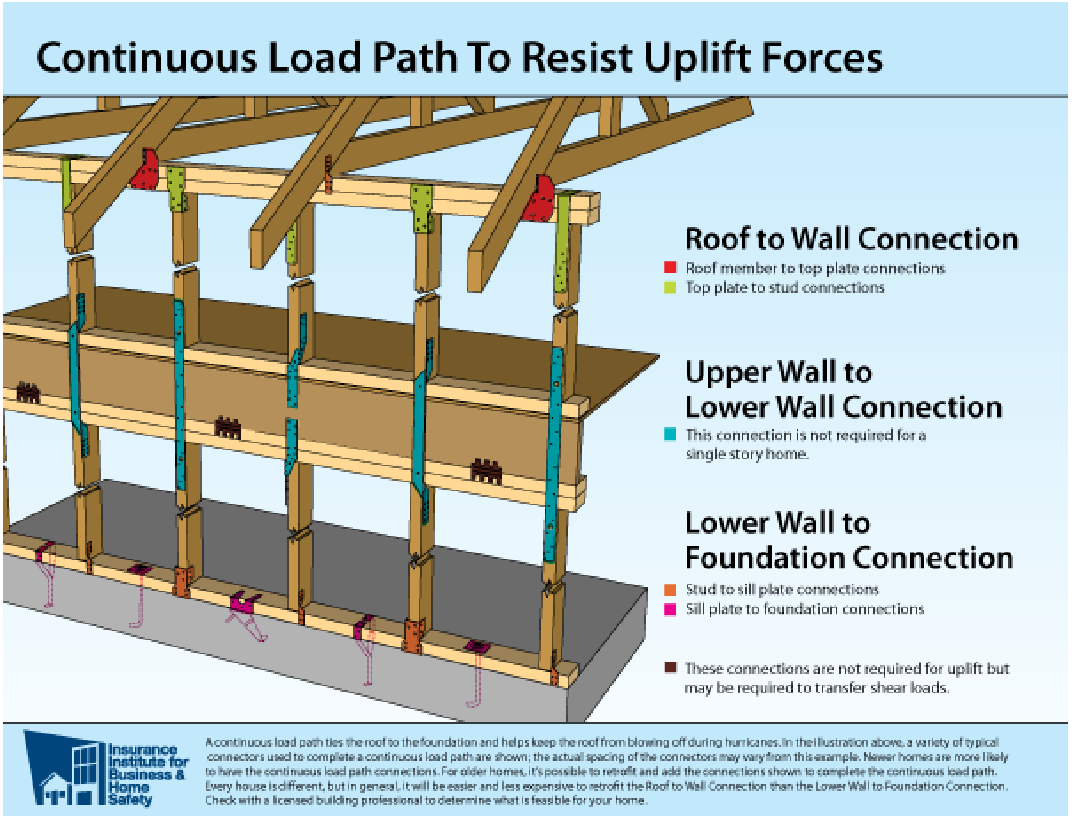


Figure 2-1 Load Path on Timber Frame Building (Safety, 2012)

2.2 Shear Wall

Shear wall essentially is a vertical structural component from foundation to story level that resist lateral loads. They support the diaphragm and transfers the loads from it to the base of building and then to the foundation as shown on figure 2-2. Wood diaphragm often used along with shear walls in wood frame buildings. (Breyer, Fridley, & Cobeen, 2007) States that Shear-wall in wood-frame building can be constructed from various material types, but few are very common as below:

- Wood structural panels [e.g., plywood and oriented strand board (OSB)]
- Gypsum wallboard (drywall)
- Interior and exterior plaster (stucco)
- Fiberboard (including fiber-cement panels)

- Lumber sheathing (diagonal or horizontal sheathing)

Among all above the mentioned materials wood, structural panels provide better resistance for a Shear wall. In cases where design shear forces are high special nailing and sheeting are taking into account. In this report, only plywood sheathing wall is tested and evaluated.

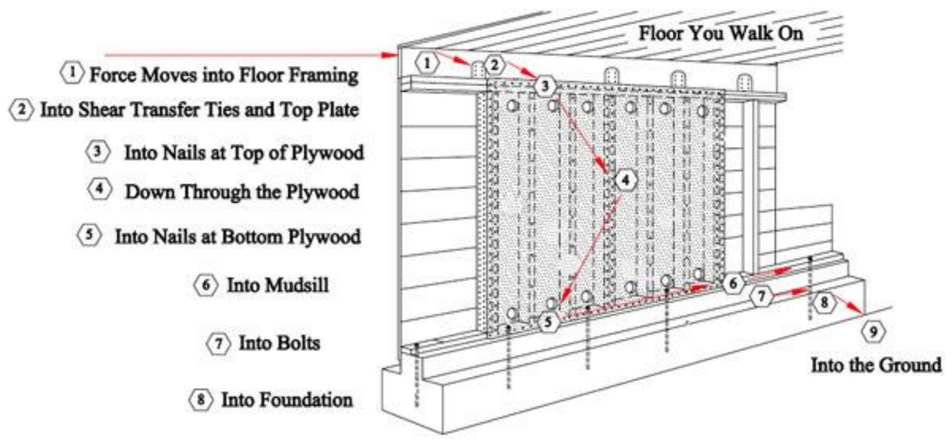


Figure 2-2 Shear wall Components (Retrofit, n.d.)

In IBC Chapter 23 determines a segment of wall consider to be Shear wall where the ratio of height-to-width meets the table. As shown in Table 2.1

Table 2-1 Shear Wall Height-to-Width Ratio

TYPE	**MAXIMUM HEIGHT-WIDTH RATIO**
Wood structural panels or particleboard, nailed edges	For other than seismic: 3½:1 For seismic: 2:1[a]
Diagonal sheathing, single	2:1
Fiberboard	1½:1
Gypsum board, gypsum lath, cement plaster	1½:1[b]

Shear wall should be designed to resist the applied shear force and overturning moment. The sheathing is the main component that resist the shear, and the top and bottom cord are intended to resist the overturning moment created by the lateral force. Some items should be considered in the design of Shear wall as follows:

- Sheathing thickness
- Shear wall nailing
- Chord design (tension and compression)
- Collector or strut design (tension and compression)
- Anchorage requirements (hold-downs and shear)
- Shear panel proportions
- Deflection

The overturning moment at the base of Shear wall is resolved in couple between two cords and are designed for both compression and tension where tension mostly controls the design. The bottom cord for the Shear wall is also called the sill plate. In this report, the main focus of study is the bottom cord (sill plate) where in-depth studies are done.

2.3 Sill Plates

The bottom cord of a Shearwall which is also called by many other names e.g. Sill Plate, Mudsill or Bottom Rail is horizontal board laid and anchored on the surface of the concrete foundation. Shear wall components such as studs, sheathing, nailing, and connections are attached to this wood member. Sill plates are the framing elements that are exposed to weather and placed on concrete where moisture exists therefore it is required that the material should be pressured treated woods. The typical sill plate is shown in figure 2-3.

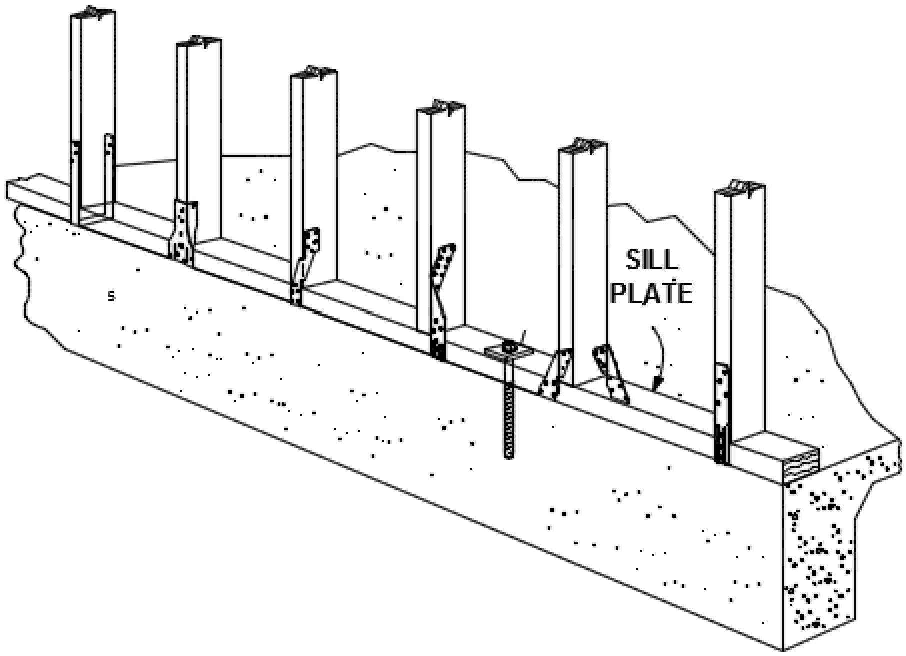


Figure 2-3 Sill Plate on Concrete Foundation (Strong-Tie, n.d.)

Usually bending stresses parallel to the grain of wood members are desirable, and the wood member has higher tension and bending capacity stresses in the direction of grains. However, there are some situations where tension or bending occurs in cross-grain direction, and it is crucial to the strength of that member. In the sill plates, the overturning moment of lateral force causes the bottom cord to go under tension stress parallel to the grain. On the other hand, the sheathing and studs due to uplift pressure or forces perpendicular to the sheathing cause cross-grain and perpendicular tension and bending in sill plates. Any form of stresses in cross-grain in not very welcomed by Codes. In the design of the sill plate, engineers should be cautious about this situation. This research focuses to determine the capacity of sill plate under cross-grain bending and tension stresses.

2.3.1 Connections in Sill Plate

Sill plates are a connected to various items e.g. Studs, Sheathing, and anchor bolts to the foundation.

2.3.1.1 Nailing

The nailing in Shear wall board requirement is a function of unit shear in the wall and the materials of construction. Appendix Table from IBC Section 2304.9.1 gives standard connection nailing and spacing to all wood structural members.

2.3.1.2 Anchorage in Sill Plate

Anchorages refer to tying down together the structural elements in a building to resist the design load. Mostly anchorage emphasizes on lateral loads. However, vertical loads, lateral parallel to the shear wall as well normal to the shear wall are the forces that are resisted by anchorages and transferred to the foundation.

The overturning moment and tension in sill plate is generally of primary concern, and the importance of large tension force depends on Shear wall segment height, width, and small resisting dead loads. If these conditions do not exist, there might not be concerned uplift pressure at the still plate. (Breyer, Fridley, & Cobeen, 2007)

The concrete foundation for sill plate should resist the uplift and bearing stresses for the anchorage and overturning moment caused by the wall.

Anchor bolt selection depends on the design capacity of the connection. The size and type of anchor bolt are determined by the capacity of wood parallel to grains or by the strength of anchor bolt in the concrete foundation. The smaller of these two values are used for selection of anchor bolt in the sill plate. The recommendation for the capacity of anchor bolt in concrete foundation is given in IBC.

2.3.2 Sill Plate Failure modes

Previously in this reporting failure of sill plate was discussed in brief. It was accepted that there were three mainly failure modes for the sill plate. 1) Failure in cross-wise bending 2) Brittle failure along edge of sill plate 3) failure in connection, it can be of sheathing nailing or anchor bolt.

(Caprolu, Evaluation of Splitting Capacity of Bottom Rails in Partially Anchored Timbr Frame Shear Walls, 2014) Used plastic design method to determine failure capacity of partially anchored shear walls. For this purpose, the ductile behavior of sheathing to sill plate connection is important to consider. Due to the absence of hold-downs in test specimens it is the nails that transfer the uplift force to the frame. The horizontal distance of vertical force on sheathing board and support connection of anchor bolts cause couple moment at half width of the sill plate. This couple moment causes crosswise bending along the bottom face of the frame. Figure 2-4 shows failure along the bottom face.

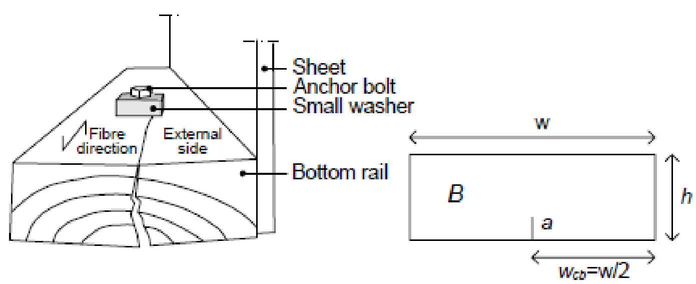


Figure 2-4 Sill Plate Failure along Bottom Face Due to Bending Stresses (Caprolu, Källsner, Girhammar, & Vessby, 2012)

Failure in cross-wise bending is not favorable as the aim is to determine the plastic design for the failure mode. The other common failure mode for sill plate can be failure along the edge, which is due to extreme normal stresses from the nails. Many factors affect this second type of failure. Any form of stress transfer from bending to shear cause brittle failure along the edge. (Caprolu, Evaluation of Splitting Capacity of Bottom Rails in Partially Anchored Timbr Frame Shear Walls, 2014) Tests indicate that washer size and shape is of vital importance in types of failures. Increasing washer size reduces the distance between sheeting and supports which reduce the moment arm, and this takes normal stress to control the failure. Rectangular washer size is more in favor of type two failure mode rather than round shape washers.

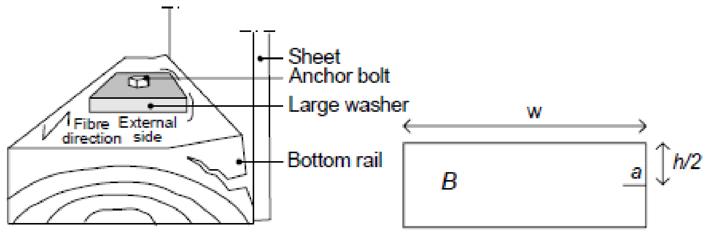


Figure 2-5 Sill Plate Failure along the Edge Due to Shear (Caprolu, Källsner, Girhammar, & Vessby, 2012)

Two failure mode has been studied by (Caprolu, Evaluation of Splitting Capacity of Bottom Rails in Partially Anchored Timbr Frame Shear Walls, 2014) and theoretical models were compared by experimental tests and two failure modes. The connection was designed to not fail under applied load for the purpose of finding the first two failure modes.

1) Fails along bottom due to bending stress that caused from eccentric loads. It tears the plate apart at mid-width as shown in figure 2-4.
2) Fails along the edge due to extreme normal force. It happens in a situation where normal stress is large than the bending moment and cause the plate to tear apart along the edge where the tensile capacity of the wood material is weak in perpendicular to grains.
3) Fail by the sheathing. It can only happen when a connection from sheathing to sill plate is incapable of uplift force. Main factors for this type of failure is nailing spacing.

By studying in depth these two failure mode, we can understand the fracture mechanics for sill plate and avoid brittle failure in practice and design.

2.3.2.1 Factors Affecting Failure Modes

There can be the various reason for each failure mode, but few are essential to take into account. Among them, anchor-bolt size, washer shape and size, pith orientation, nails spacing and material properties.

2.3.2.1.1 Sheathing to Frame Nail Spacing

The difference between the anchor bolt and hold-down bolts. Anchor bolts are mainly placed to resist shear forces while hold-down bolts are to resist any uplift forces. Hold-downs transfer vertical loads to the sill plate and then through anchor bolts to the foundation. If no hold-downs provided in the frame, it can cause the load to be transferred through nails to the sill plate and then to the foundation. In this case, nails undergo ultimate shear forces.

To reduce stress on nails, the best way is to increase the numbers of nails where on the another hand spacing is reduced. This makes connection safer to not fail in-between sheathing and sill plate.

2.3.2.1.2 Washer Size and Shape

Washer size and shape is the game-changing element in modes of failure for sill plates. In analyzing of plate fracture, we assume plate to act as cantilever beam as shown on figure 2-6. By increasing the size of the washer the free length is reduced and this cause for less moment at the fixed end and eventually lesser bending stress. The failure mode along the bottom is changed to along the edge. (Caprolu, Evaluation of Splitting Capacity of Bottom Rails in Partially

Anchored Timbr Frame Shear Walls, 2014) States square washers were used in failure along the edge while round shape washers are more likely toward failure along the bottom.

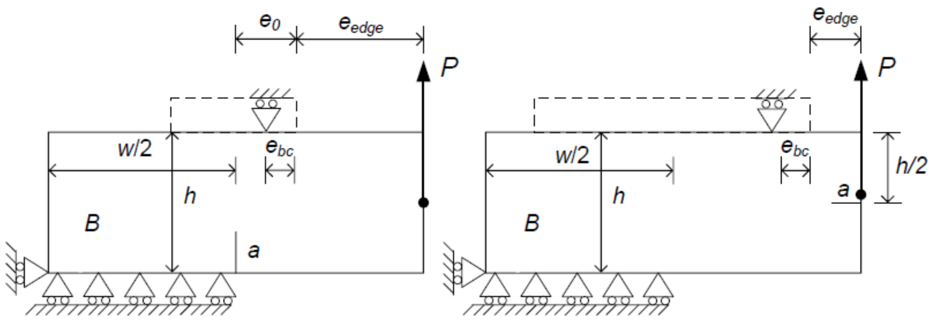


Figure 2-6 Sill Plate Stress Distribution Mechanism (Caprolu, Källsner, Girhammar, & Vessby, 2012)

2.3.2.1.3 Pith orientation

Pith orientation in many places during construction is not considered to be taking into account, but from test results it is imperative in failure modes of sill plate or at all any wood member.

When the anchor bolts are tightened on the sill plate, it creates bending stress on its edge and meantime tensile stress downward. On the other hand, the sheathing-to-sill plate also

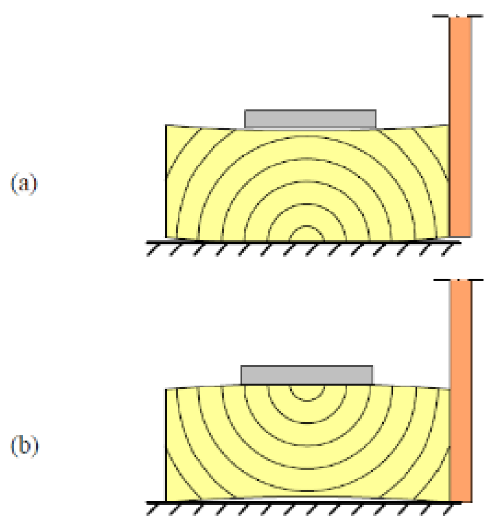


Figure 2-7 a) Pith downward (b) Pith Upward (Caprolu, Källsner, Girhammar, & Vessby, 2012) causes crosswise bending. In this time, it is paramount to place the piece of wood with piths oriented download.

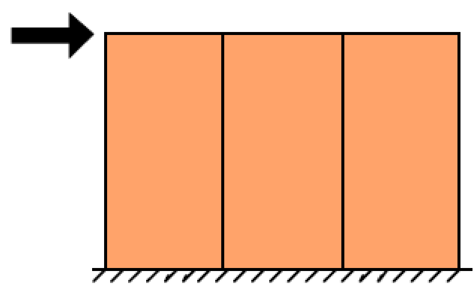
2.3.2.1.4 Materials Properties

It is very obvious that material properties are the final values decision-making element in the capacity of members. Since, wood is orthotropic and inhomogeneous materials its properties in any direction or location differs from the rest.

2.3.3 Sill Plate Test Programs

Previously it was discussed that that loads from Shear wall transferred to the sill plate and the foundation. The lateral loads or normal loads to Shear wall cause sill plate to experience uplift pressure only in partially anchored sill plate situation. The uplift pressure is happening mostly at either end of the wall as it is shown in figure 2-8.

(a)



(b)

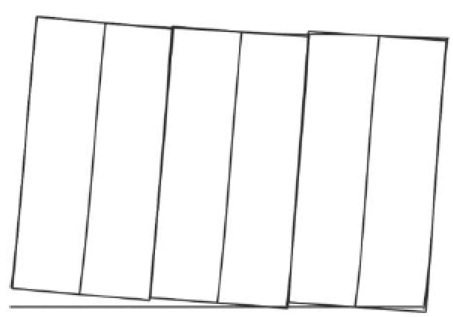


*Figure 2-8 (*a) partially anchored wall subjected to a horizontal load. (b) Displacement plot at the instance of maximum load obtained by a finite element calculation (10 times enlargement of displacements). *(Caprolu, Källsner, Girhammar, & Vessby, 2012)*

(Caprolu, Källsner, Girhammar, & Vessby, 2012) Has conducted a several test to determine the failure modes for the sill plate. The investigation parameters for the tests are as the size of washer, pith orientation, anchor bolt position. Eight series of tests have been grouped, modeled and tested as shown in Table 2-2 and 2-3.

Table 2-2 Test Configurations PD=Pith Downward, PU=Pith Upward

Series	**Set**	**Number of Tests**		**Anchor bolt position[mm]**	**Size of Washer [mm]**
		PD	**PU**		
1	1	8	2	60 (b2)	40X40X15
	2	8	2		60X60X15
	3	8	2		80X70X15
	4	8	2		100X70X15
2	1	8	2	45 (3b/8)	40X40X15
	2	8	2		60X60X15
	3	8	2		80X70X15
3	1	8	1	30 (b/4)	40X40X15
	2	8	1		60X60X15

Table 2-3 Test Configuration PD=Pith Downward, PU=Pith Upward

Series	**Set**	**Number of Tests**		**Anchor bolt position[mm]**	**Size of Washer [mm]**
		PD	**PU**		
4	1	8	8	60 (b2)	40X40X15
	2	8	8		60X60X15
	3	8	8		80X70X15
	4	8	8		100X70X15
5	1	7	7	45 (3b/8)	40X40X15
	2	8	8		60X60X15
	3	8	8		80X70X15
6	1	8	8	30 (b/4)	40X40X15
	2	8	8		60X60X15

In both testings, pith orientation is taking into account and study its influence in failure models. In the first study, the number of pith upward are lesser than downward where this give us clear idea of how influential pith orientation can be.

The model is designed in a way that uplift force should be centered with the structure as shown in figure 2-9. For the first test, the applied torque is 50Nm (442.5 lbs-in) with a constant displacement of 2mm/min (0.078in/min) and nail spacing of 25mm for the sheathing-to-sill connection. For the series 4, 5 and 6 a torque 50Nm with a constant displacement of 10mm/min (0.39in/min) and 50mm (1.96in) sheathing-to-sill connection. Details of models are as below:

- Bottom rail: C24 according to EN 338, 45X120 mm (1.78inX4.72in).
- Sheathing: Hardboard, 8 mm (0.31in)
- Sheathing-to-framing joints: Annular ringed shank nails, 50 2.1 mm.
- The joints were nailed manually, and the holes were pre-drilled (only in the sheet), 1.7 mm. Nail spacing was 25 mm or 50 mm (2 or 4 in). Edge distance was 22.5 mm (1.9in) along the bottom rail.

- Anchor bolt: Ø 12 (M12). The holes in the bottom rails were pre-drilled, 14 mm (0.55 in).

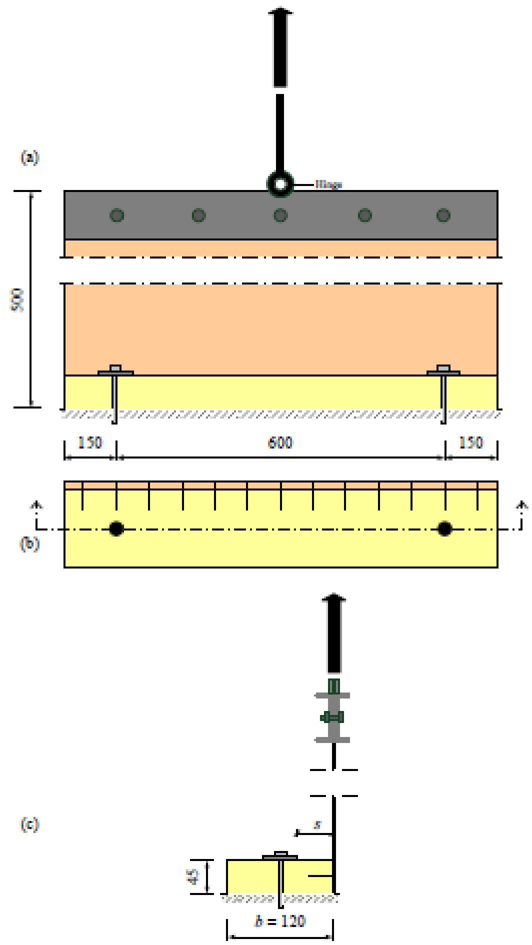


Figure 2-9 Test Model (Caprolu, Källsner, Girhammar, & Vessby, 2012)

2.3.3.1 Test Results

The tests outcome validates the three fundamental failure modes for sill plates as below:

1) A vertical crack develops from the bottom side of the rail
2) A horizontal crack develops from the edge side of the rail, in line with the fasteners. The crack changes gradually direction to an angle about 45 degrees
3) Yielding and withdrawal of the fasteners in the sheathing-to-framing joints.

Failure mode results for two tests are shown in Figures 2-10 and 2-11. For the first test, the results for plates with piths oriented upward are not displayed as the number of test samples were considerably less.

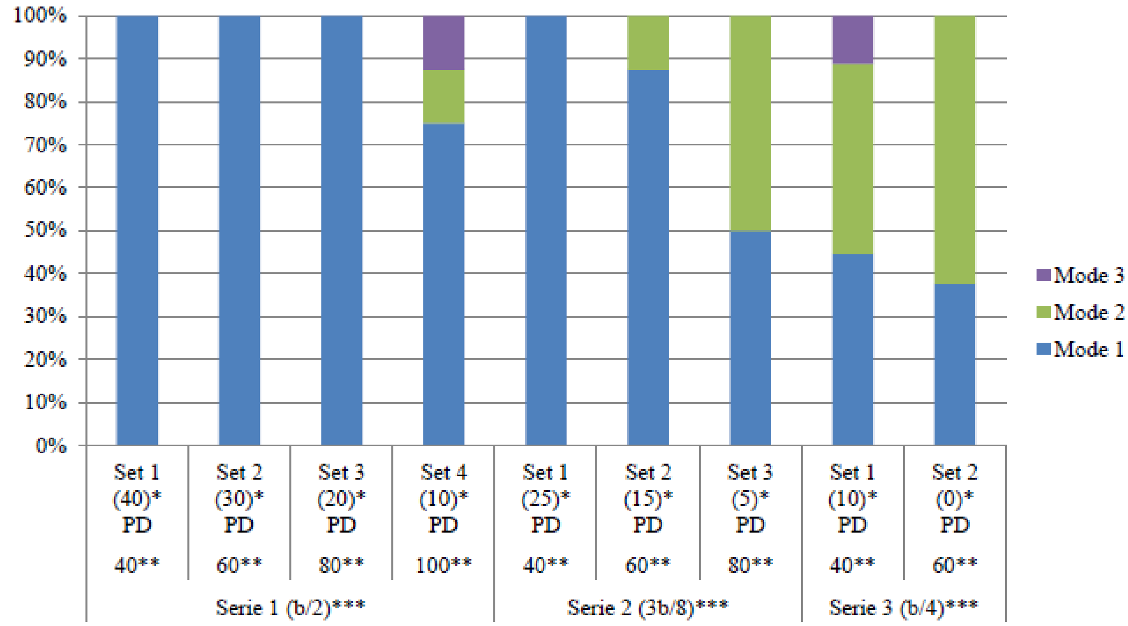


Figure 2-10 Test Results (Caprolu, Källsner, Girhammar, & Vessby, 2012)

In the first study, the failure of mode three is very few in compare to test two results. However, on the other hand, the failure loads in series 1-3 are higher than series 4-6. The reason behind this issue is nail spacing of sheathing to sill plate connection. It has been taken 25mm (2in) nail spacing for test one and 50mm (4in) for test two. The other significant factor here is the pith orientation. Tables 2-4 and 2-5 depicts that the failure loads for plates with piths oriented downwards are about 10% higher than those that are upwards.

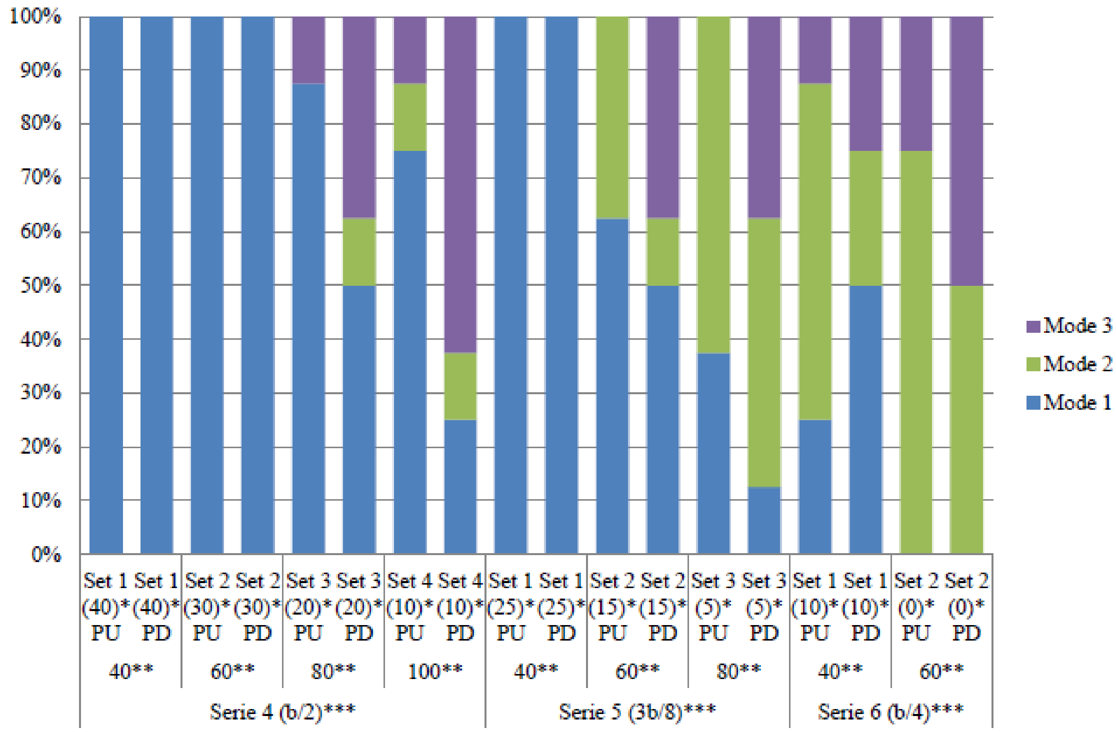


Figure 2-11 Test Results

Table 2-4 Results of testing of specimens with the pith oriented downwards (PD). $\rho_{0,\omega}$ = dry density, ω = moisture content *(Caprolu, Källsner, Girhammar, & Vessby, 2012)*

Series	Set	Number of tests	Failure load mean [kN]	Failure load stddev [kN]	$\rho_{0,\omega}$ [kg/m³]	ω [%]
1	1	8	12.0	1.8	418	13.2
	2	8	13.5	2.5	372	12.6
	3	8	17.4	1.8	380	12.9
	4	8	22.8	4.4	398	12.4
2	1	8	16.0	2.2	424	13.0
	2	8	20.7	2.6	389	12.5
	3	8	29.1	2.9	419	13.4
3	1	8	21.6	3.1	348	12.6
	2	8	29.2	1.9	420	13.1
4	1	8	10.2	1.8	392	12.2
	2	8	13.5	2.1	383	10.9
	3	8	18.2	1.5	426	10.9
	4	8	21.8	1.7	406	10.9
5	1	7	14.0	2.8	394	9.0
	2	8	17.9	4.5	381	12.6
	3	8	23.7	3.2	415	11.6
6	1	8	18.1	2.3	364	11.4
	2	8	23.8	2.5	414	11.9

Table 2-53 Results from testing of specimens with the pith oriented upwards (PU). $\rho_{0,\omega}$ = dry density, ω = moisture content

Series	Set	Number of tests	Failure load mean [kN]	Failure load stddev [kN]	$\rho_{0,\omega}$ [kg/m^3]	ω [%]
1	1	2	12.6	1.3	394	13.6
	2	2	11.3	0.5	368	12.2
	3	2	17.0	5.7	365	13.1
	4	2	24.1	0.4	397	13.1
2	1	2	21.5	0.5	426	13.5
	2	2	21.1	0.8	398	13.1
	3	2	28.9	2.5	427	13.4
3	1	1	19.9	-	312	12.1
	2	1	27.1	-	380	12.7
4	1	8	9.5	2.6	397	11.9
	2	8	10.6	2.0	390	10.9
	3	8	17.1	3.0	409	10.7
	4	8	20.0	1.6	422	11.1
5	1	7	12.2	2.4	405	9.6
	2	8	16.9	2.6	360	12.4
	3	8	22.6	4.1	416	11.6
6	1	8	18.6	2.3	379	11.6
	2	8	21.3	2.7	402	12.2

As tensile load to models were at the constant rate, the load-displacement curves are plotted that shows load vs. time for all three failure modes. Figure 2-12 (a) is mode one where the failure occurs along the bottom side of the plate due to the bending moment. In this graph, we see that the first drop in the load due the first crack near anchor bolt. Similarly the second decrees for a second crack along other side anchor bolt and final failure of sill plate in bending stresses. In (b) there is one significant distinct decrease, and it is clear that failure along the edge happened at once. (c) in this curve we see that load is dramatically decreasing, and the reason is that nails are pulling off the plate as the load is declining.

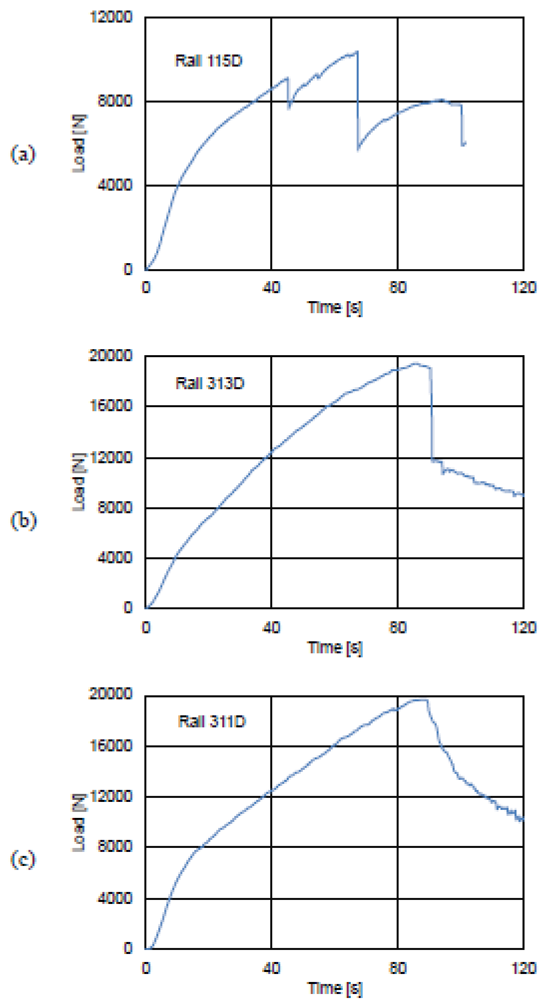


Figure 2-12 (Caprolu, Källsner, Girhammar, & Vessby, 2012)

Results for bolt position and washer size are shown in figures 2-13 & 2-14

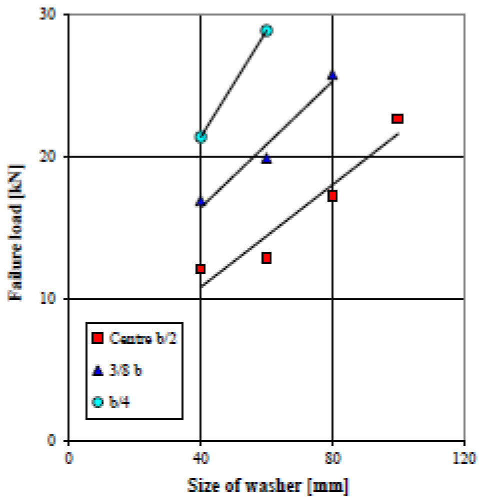


Figure 2-13 Mean failure load versus size of the washer for different bolt positions in the first study. *(Caprolu, Källsner, Girhammar, & Vessby, 2012)*

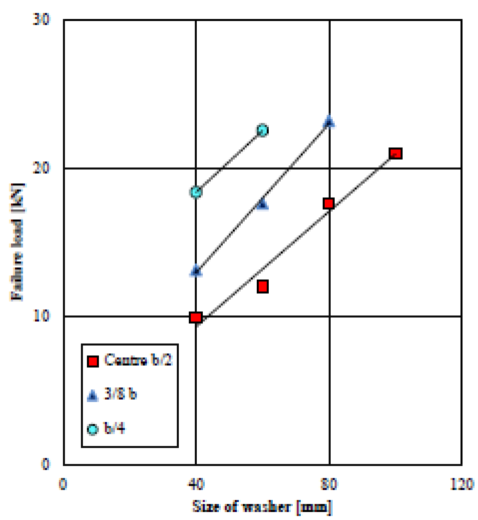


Figure 2-14 Mean failure load versus size of washer for different bolt positions in the second study. *(Caprolu, Källsner, Girhammar, & Vessby, 2012)*

(Caprolu, Källsner, Girhammar, & Vessby, 2012) Test results give this conclusion that all above mentioned factors are significantly affecting the sill plate failure modes due to uplift pressure. Among all, the bolt location and the edge distance from the washer to the edge of the plate is the key element in transforming failure mode one to failure mode two.

CHAPTER III

Theoretical Analytical Approach and Laboratory Testing Methodology

In this chapter, theoretical approach to failure capacity of the sill plate is determined. An average perpendicular to grain tensile strength of 350 psi for Douglas fir is taken in calculations. This value is not exactly the same as testing specimens, but it is a very typical value for the purpose of calculations. Later in the second section of this chapter testing procedure is discussed and described in details.

3.1 Theoretical Approach to Failure Mode Capacities

3.1.1 CASE I

We assume failure mode one where plate starts fail along the bottom face and cracks initiated and goes all the way along the face. Ideally we take all 36 inches length for capacity strength. Figure 3-1 shows this case. The hatched area is the stresses area for tensile stresses and taken the area of moment of intertie.

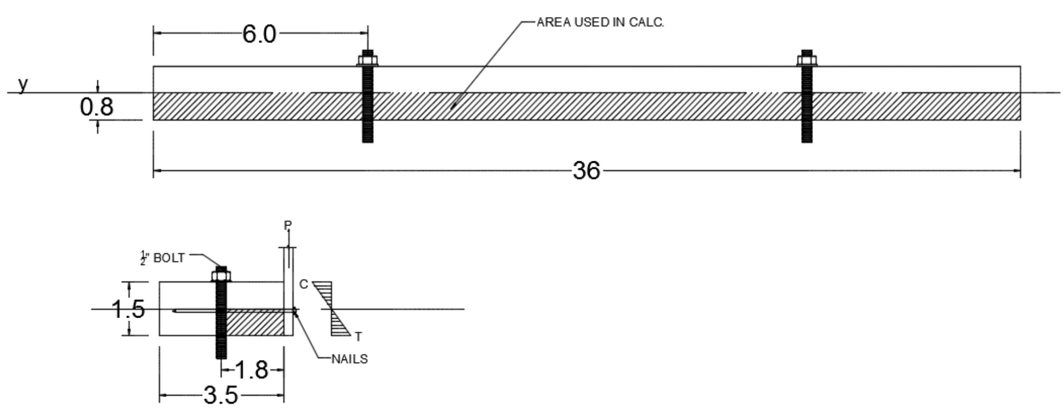


Figure 3-1 Cross Sectional Detail CASE I, Hatched Area Is Taken For Moment Of Inertia. Al Dimensions Are In Inches

Assumed values:

$\sigma(allowable\ tensile\ strength) = 350psi$

$d(distance\ from\ face\ of\ sheathing\ to\ bolts) = \frac{7}{4} inches$

$I = moment\ of\ inertia = 10.125in4$

$y = \frac{3}{4} inches$

$$\sigma = \frac{my}{I} = m = \frac{I\sigma}{y}$$

$$m = 4{,}725\ lb - in$$

$$p = \frac{m}{d} = 2{,}700\ lb$$

3.1.2 CASE II

It is assumed that plate starts fail along the bottom face, and cracks initiated and goes only a foot from both sides. Figure 3-2 shows this case. The hatched area is the stresses area for tensile stresses and taken for the area of moment of intertie.

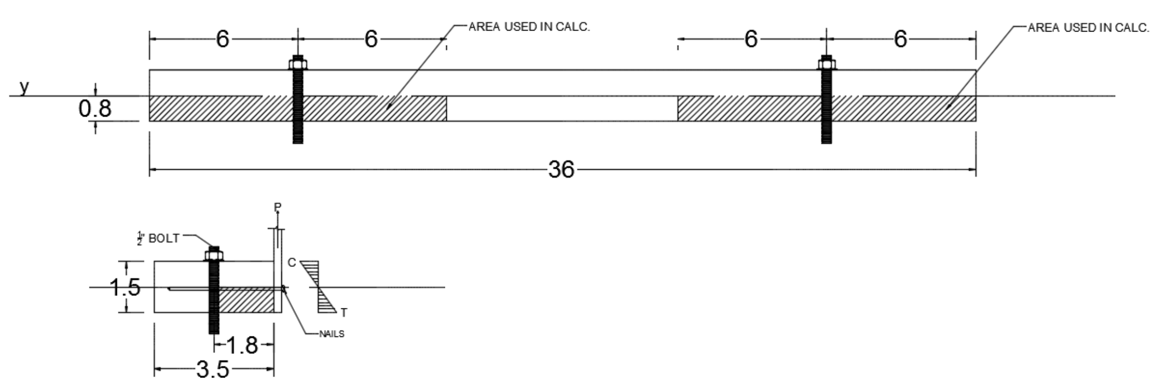


Figure 3-2 Cross Sectional Detail CASE Il, Hatched Area Is Taken For Moment Of Inertia. Al Dimensions Are In Inches

Assumed values:

$\sigma(allowable\ tensile\ strength) = 350psi$

$d(distance\ from\ face\ of\ sheathing\ to\ bolts) = \frac{7}{4}inches$

$I = moment\ of\ inertia = 6.75in4$

$y = \frac{3}{4}inches$

$$\sigma = \frac{my}{I} = \ m = \frac{I\sigma}{y}$$

$$m = 3{,}150\ lb - in$$

$$p = \frac{m}{d} = \ 1{,}800\ lb$$

3.1.3 CASE III

In this case failure capacity of mode 2 is determined. It is assumed that loads from sheathing to the plate is transferred via nails. Stress distribution to nails is assumed to be not uniform while it is considered unsymmetrical. The area used for stress calculations is taken a conservative value from the edge of the washer to the sheathing. Figure 3-3 shows how load is distributed from nails to the wood.

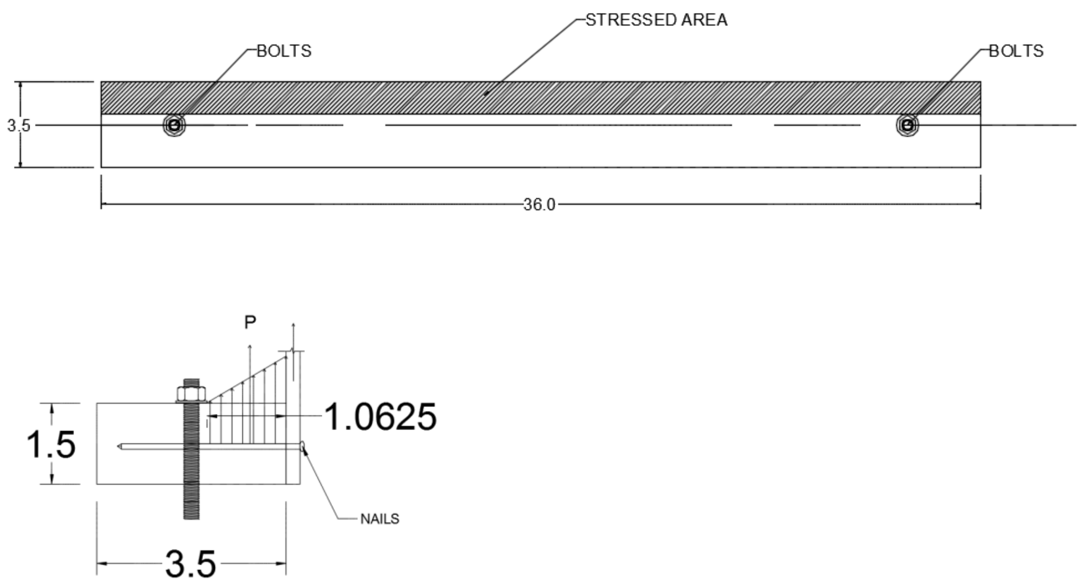


Figure 3-3 Cross Sectional Detail CASE III, Hatched Area Is Stressed. Al Dimensions Are In Inches

Assumed values:

$$\sigma(allowable\ tensile\ strength) = 350psi$$

$$\sigma = \frac{P}{A}$$

$$A = 38.25\ in2$$

$$P = (350psi)(38.25in2) = 13{,}387.5lb$$

3.1.4 CASE IV

In this case, the width of crack along the edge is reduced comparatively to CASE III by 0.5 inches. Still by a significant reduction in crack length it still has higher capacity than any of CASE I and CASE II.

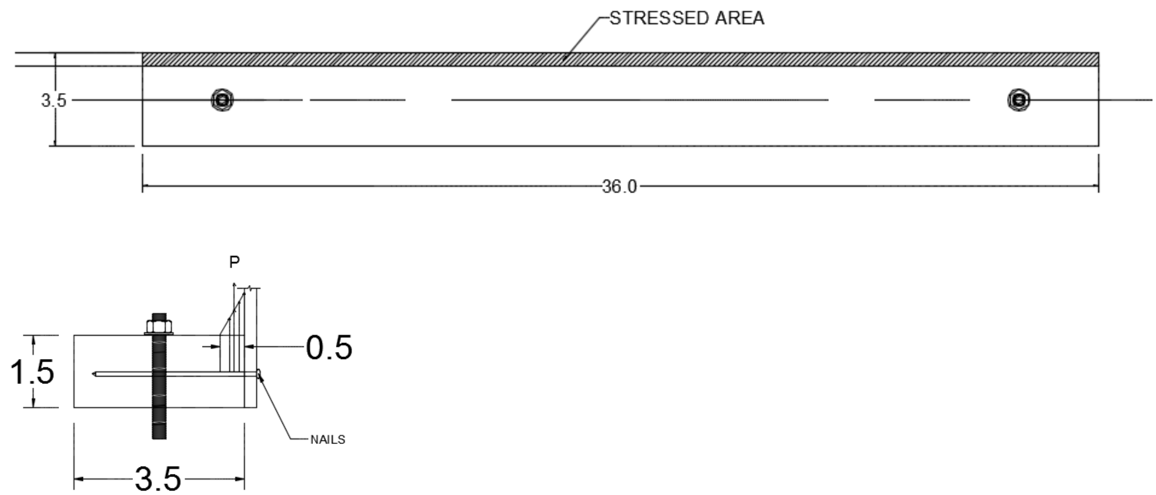


Figure 3-4 Cross Sectional Detail CASE IV, Hatched Area Is Stressed. Al Dimensions Are In Inches

Assumed values:

$$\sigma(allowable\ tensile\ strength) = 350psi$$

$$\sigma = \frac{P}{A}$$

$$A = 18\ in2$$

$$P = (350psi)(18in2) = 6{,}300\ lb$$

It looks like the failure mode 2, failure along the edge, has higher capacity in comparison to bending failure of mode 1. Later test results also validated this theory.

3.2 Introduction to Testing Procedure

In the previous chapters, failure modes for the sill plate have been explained. The major factors affecting failure modes has also been described. Among important and significant ones are nailing spacing, washer size, bolt offset from the sheathing and pith orientation in the cross section. In this research, additional scenarios are proposed for sill plate tests to find out how significant these additional scenarios improve the ultimate capacity of the sill plate. It is obvious that due to bending moment the bottom face of the sill plate undergoes tensile stresses. By having this idea, in this research one of the ways to enhance the ultimate capacity is the addition of metal plate connectors on the bottom face of the sill plate.

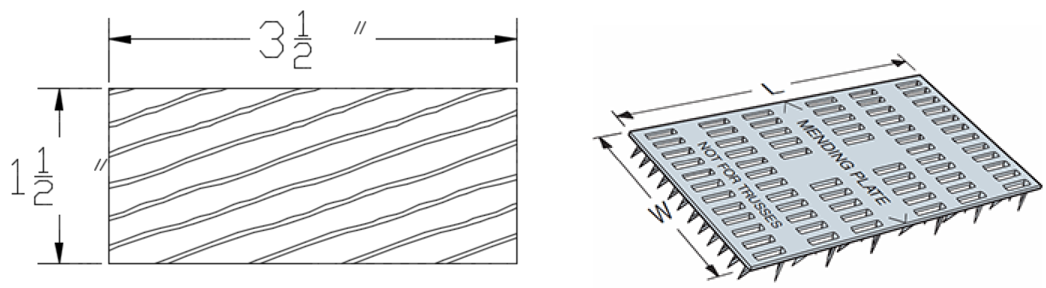


Figure 3.5 (a) 2X4 Douglas Fir No.2 2actual size. (b) Metal Plate Connecter 3inX3in. (Strong-Tie, n.d.)

In all tests round shape washers are used. To find the ultimate capacity of the sill plate with a metal plate, it is required to apply maximum bending while preventing other failure modes. One solution to this situation is repositioning the bolts at an offset distance from the center of the plate. This research focused on the failure of the sill plate. As such, to prevent sheathing failure, a ¼" steel plate is acting as a stiff sheathing in all tests. All wood pieces are 2X4-Douglasfir No. 2. The design value for Douglas fir is given by National Design Standard code in the appendix. All lumber pieces are the same size but with some distinct surface conditions and pith orientation. Later to compare the test results it was considered to designated by labels D, R and P were used to identify the wood as shown in figure 3.6. These designated labels are used for a similar type of wood pieces. All nails are the 16d type and are spaced 2 inches apart.

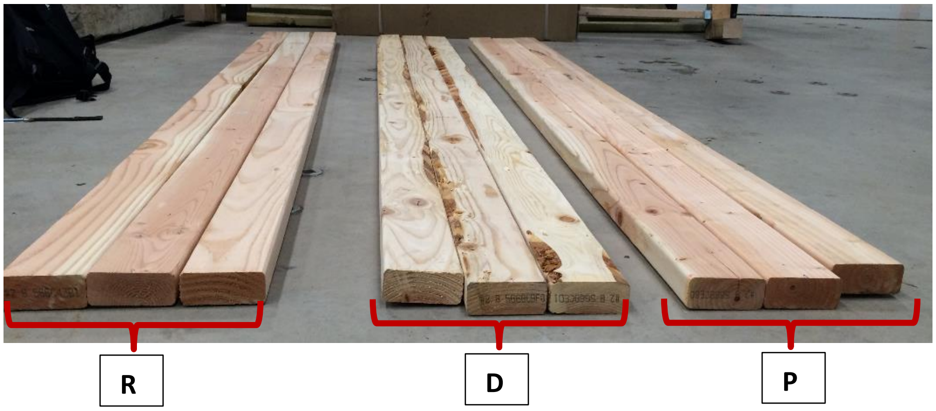


Figure 3-6 Wood Samples, R= ***R****andomly Picket Wood, D=Pieces with Some* ***D****eficiencies on Surface, P=***P***ith in Center*

3.3 Testing Setup

The base for sill plate is made of L4X4X1/4 angle and stiffened additionally by stiffeners in various locations, and it is assumed this piece acts as an infinite rigid foundation. A ½ inch whole slot is drilled 6 inches from both edges. The main purpose of long slots is that bolts can freely move from sheathing to a distance to increase the moment arm. Sheathing is made of 1/4in steel plate that is capped at the top for the reason of gripping by the testing machine. Nail spacing in sheathing metal is 2 inches. Another cap at the bottom leg of angles is welded for grips in the testing machine. Tests were done using an Instron Universal Testing Machine 1500 HDX, and data was collected by software linked to the computer. The uplift extension rate in the machine is set to be 0.1 inches per minute and readings in 0.1 seconds. Following figure 3.7 gives details of the testing model.

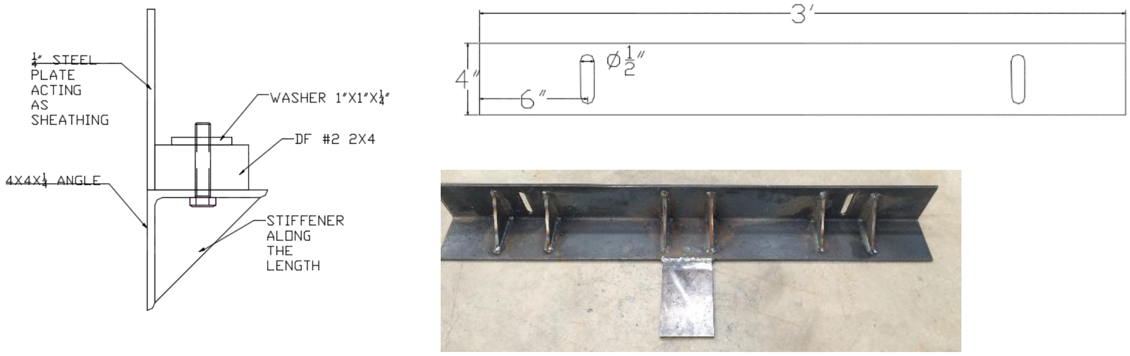


Figure 3-7 Testing Model

3.4 Test Configurations

All test are clustered in four samples where each sample consists of three specimens. Following Table 3.1 describes all samples configurations.

Figure 3-8 Sample Specimens

Figure 3-9 Sill Plate with Metal Plate and Offset Holes.

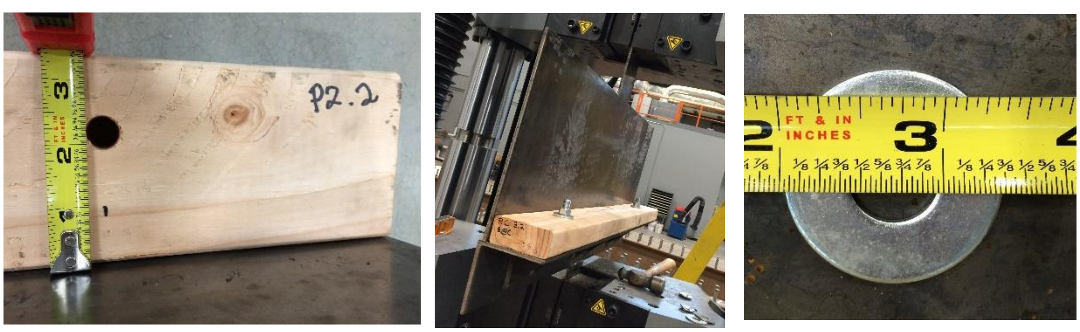


Figure 3-10 Offset Bolt, Testing Model and Washer Size

Table 3.1 Test Configuration. The designation ID is used to determine the specimen of same lumber that are cut in two pieces. E.g. D3.2 and D3.1 are from same lumber cut in two pieces.

	SPECIMEN[1]	DESIGNATION	PITH ORIENTATION	DESCRIPTION
SAMPLE.1	#1	D3.2	Downward	In this sample, a normal condition for sill plate is considered. Bolt are located at the center. ½” bolts are placed with a 1.375 outer diameter washer size.
	#2	R3.2	Downward	
	#3	P3.2	Upward	
SAMPLE.2	#1	D3.1	Downward	In this sample, a normal condition for sill plate is considered. Metal Plate connectors are provided on the bottom face of the plate. Bolt are located at the center. ½” bolts are placed with a 1.375 outer diameter washer size.
	#2	R3.1	Downward	
	#3	P3.1	Upward/Center	
SAMPLE.3	#1	D2.2	Downward	In this sample, a normal condition for sill plate is considered. Bolt are located at an offset distance of 2.25 inches from sheathing with a double 1.375 outer diameter washer size.
	#2	R2.2	Upward	
	#3	P2.2	Downward/Center	
SAMPLE.4	#1	D2.1	Downward	In this sample, a normal condition for sill plate is considered. Metal Plate connectors are provided on the bottom face of the plate. Bolt are located at an offset distance of 2.25 inches from sheathing with a double 1.375 outer diameter washer size.
	#2	R2.1	Downward	
	#3	P2.1	Downward/Center	

[1] Photos for all specimens are in Appendices under result of each specimen.

CHAPTER IV

TEST RESULT FINDINGS

In this chapter, the output results from tests are analyzed and details are described. Each sample is described separately while results for single specimens can be found in the appendix. Later in this chapter an overall comparison between all samples and specimens are tabulated in the Table 4-1. Moreover, individual samples are compared in pairs to see how much metal plate contributed and brought changes in final load capacity of the sill plate.

4.1 SAMPLE#1 Test Results

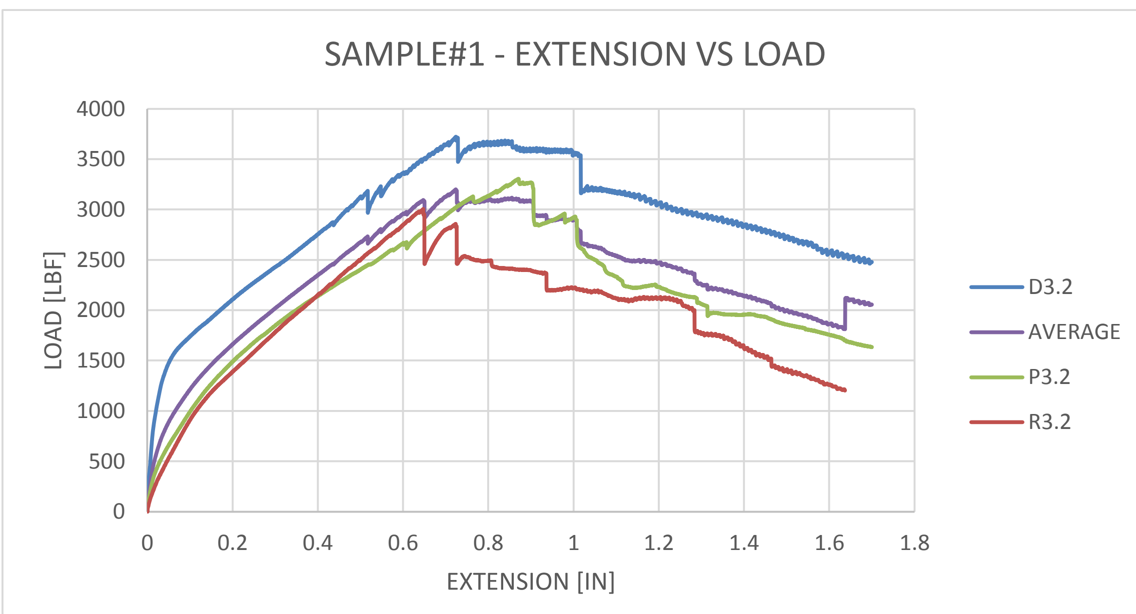


Figure 4-1 SAMPLE#1 Test Results

Table 4-1 SAMPLE#1 Test Results

SPECIMEN	EXTENSION AT MAX. LOAD (IN)	MAX. LOAD(LBF)
D3.2	0.72	3722
R3.2	0.64	3003
P3.2	0.87	3305
MEAN	**0.74**	**3343**
MEDIAN	0.72	3305

For all specimens in SAMPLE#1, the maximum test length is set to be 60% of the drop in peak load. This means that testing machine stops when the load is 60% less than the maximum load recorded previously. Each major drop is an indication of cracks along the bottom face. Moreover, then steady drops are micro cracks.

The maximum load for this sample is with specimen D3.2 and minimum specimen R3.2. The mean value for this sample is 3343 lbs. Failure modes for all specimens are due to cross-wise bending. Cracks initiated on the bottom face. The failures for all three specimens are shown in figure 4-2.

Figure 4-2 Fracture of SAMPLE#1 Specimens are all in bending.

4.2 SAMPLE#2 Test Results

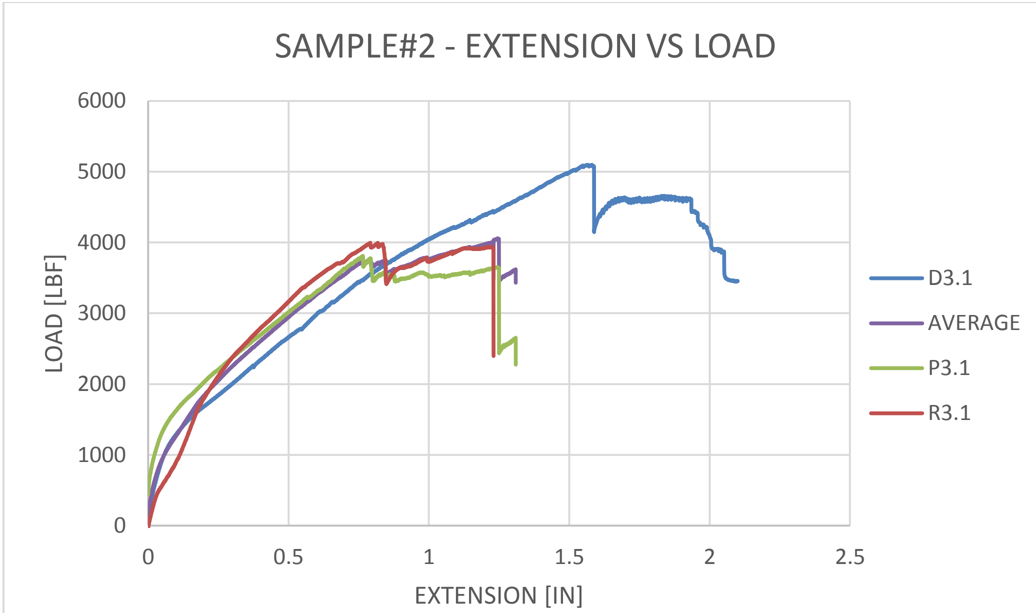


Figure 4-3 SAMPLE#2 Test Results

For specimen D3.1, the maximum test length is set for 60% drop of peak load while for the two other specimens it is assigned to be 40%.

Table 4-2 SAMPLE#2 Test Results

SPECIMEN	EXTENSION AT MAX. LOAD (IN)	MAX. LOAD(LBF)
D3.1	1.56	5095
R3.1	0.79	3994
P3.1	0.76	3807
MEAN	1.03	4299
MEDIAN	0.79	3994

The maximum load for this sample is with specimen D3.1 and minimum with specimen P3.2. The mean value is 4299 lb. We see the capacity for each specimen is significantly increased in compare to SAMPLE#1 specimens. The failure mode in this sample is along the edge, and the reason is that metal plate does not let the plate fail along the bottom face. It has been observed that the bottom face has not suffered any damages as shown in Figure 4-4 (b). The failure mode for all three samples is shown in the figure.

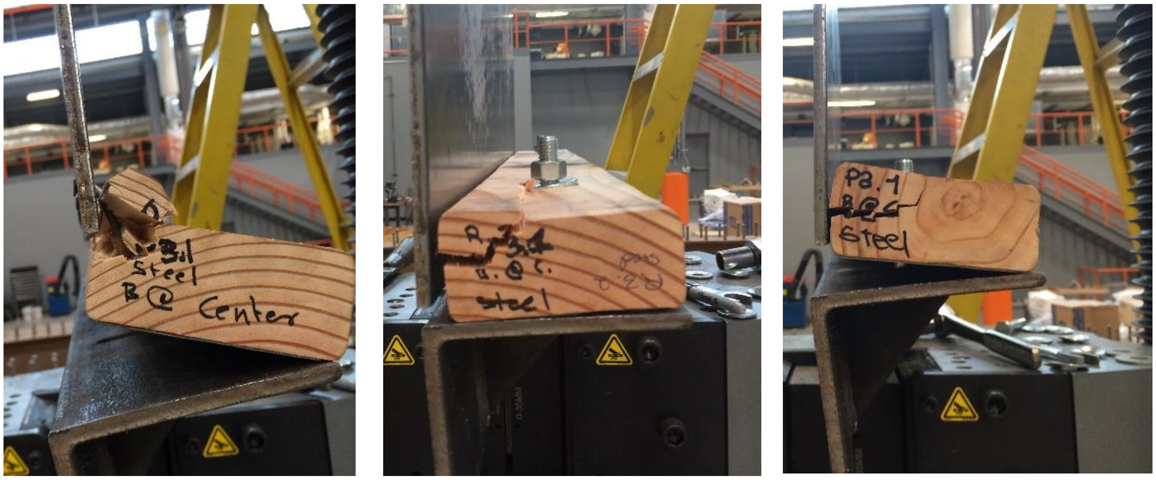


Figure 4-4 (a) Fracture of SAMPLE#2 Specimens along the Edge.

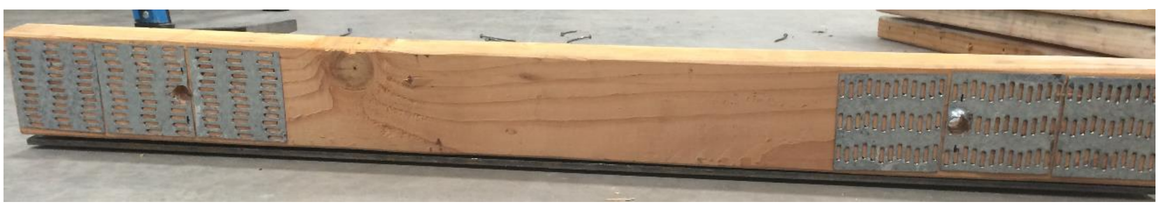


Figure 4-4 (b) Bottom Face of SAMPLE#2 Specimen. No Sign of Crack Or Any Damage Can Be Seen.

4.3 SAMPLE#3 Test Results

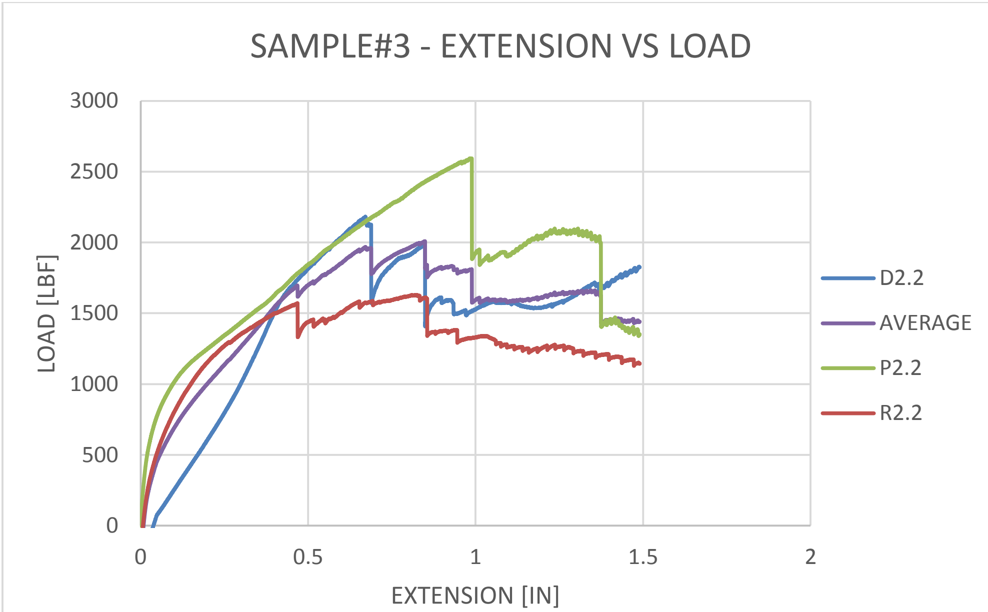


Figure 4-5 SAMPLE#3 Test Results

For all specimens, the maximum test length is assigned for 50% drop of peak load.

The highest load for this sample is with specimen P2.2 and minimum with specimen R2.2. The overall mean for all three specimens is 2135.7 lb. We see the capacity for each specimen is significantly reduced in compare of SAMPLE#1, and the reason is that the moment arm is increased and the bending moment is ultimate. The failure mode in this sample is along the bottom face. The failure for all three samples is shown in figure 4-6.

Table 4-3 SAMPLE#3 Test Results

SPECIMEN	EXTENSION AT MAX. LOAD (IN)	MAX. LOAD(LBF)
D2.2	0.67	2182
R2.2	0.82	1629
P2.2	0.98	2594
MEAN	**0.82**	**2135**
MEDIAN	**0.82**	**2182**

Figure 4-6 Fracture of SAMPLE#3 Specimens along bottom face.

4.4SAMPLE#4 Test Results

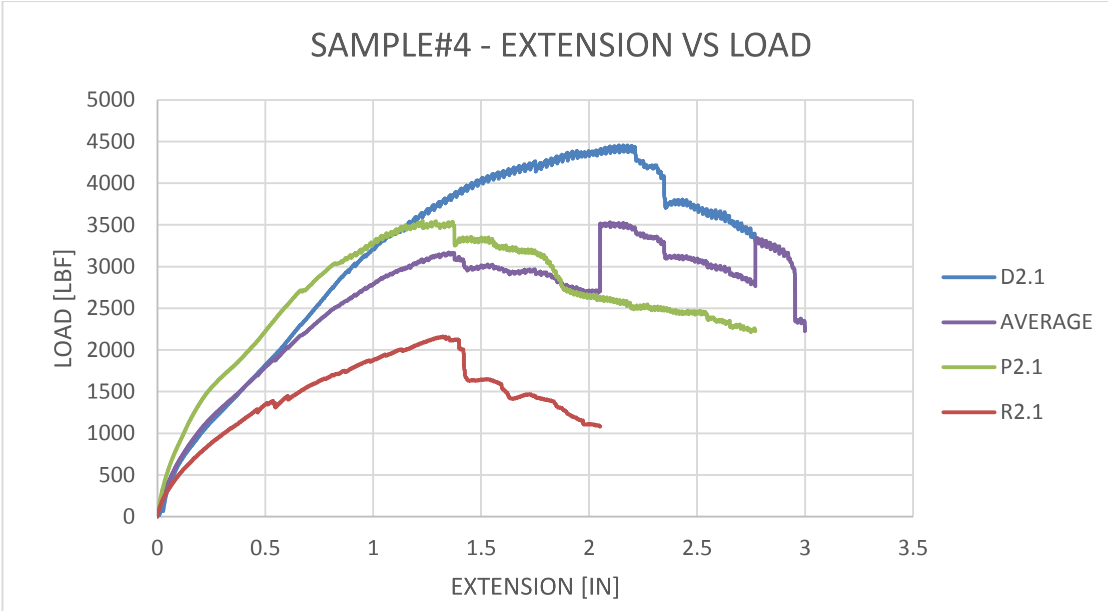


Figure 4-7 Test Results for SAMPLE#4

For all specimens, the maximum test length is assigned for 50% drop of peak load.

Table 4-4 SAMPLE#4 Test Results

SPECIMEN	EXTENSION AT MAX. LOAD (IN)	MAX. LOAD(LBF)
D2.1	2.15	4452
R2.1	1.32	2159
P2.1	1.22	3548
MEAN	**1.57**	**3386**
MEDIAN	**1.32**	**3548**

The maximum load for this sample is with specimen D2.1 and minimum with specimen R2.1. The mean value is 3386.8.7 lb. It can be seen that the capacity for each specimen is significantly high in compare to SAMPLE#3 specimens, and it is all due to the addition of metal plate on the bottom face. The failure mode in this sample is various. For the specimen D2.1, it is along the lower face far the end and incline at an approximately 45-degree angle. For specimen R2.1, it is along the edge, and for P2.1 it is again along the bottom face and a straight crack from bottom to the top face of the plate. Overall, the cracks are initiated at an offset from bolts to the left edge of the plate. The fracture mechanics for all three samples are shown in the figure 4-8.

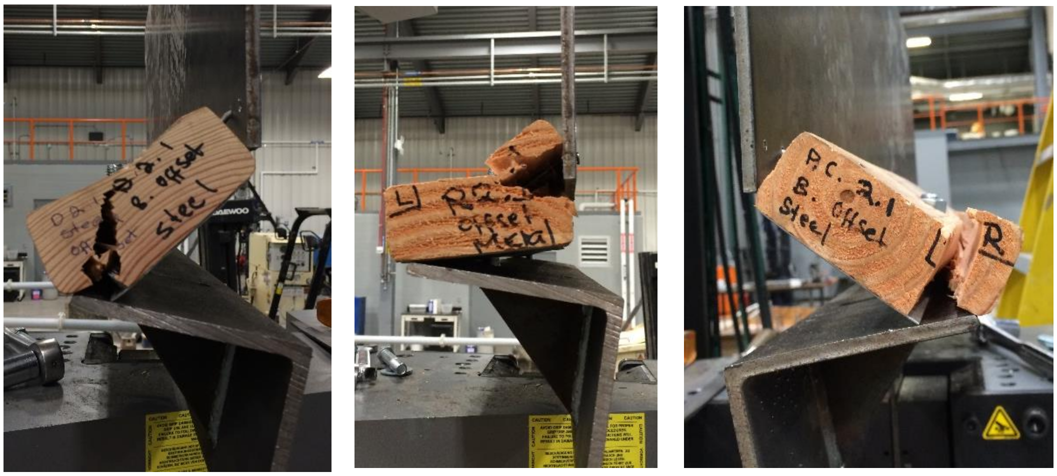


Figure 4-8 Fracture of SAMPLE#4 Specimens. D2.1 Crack goes from bottom in an angle to the line of bolts. R2.1 Fails along edge due to tensile perpendicular to grains. P2.1 splits apart.

4.5 Result Comparison Between Samples

Table 4-5 Result Comparison Between Samples

	SAMPLE#1			SAMPLE#2			ENHENCEMENT	
	SPECIMEN	EXTENSION[IN]	LOAD [LBF]	SPECIMEN	EXTENSION[IN]	LOAD [LBF]	EXT. %	LOADS %
BOLTS AT CENTER	D3.2	0.72404	3722.789	D3.2	1.56328	5095.1112	116%	37%
	R3.2	0.64703	3003.88	R3.2	0.79137	3994.6565	22%	33%
	P3.2	0.87102	3305.314	P3.2	0.76287	3807.4645	-12%	15%
	MEAN	**0.74736333**	**3343.994**	**MEAN**	**1.03917333**	**4299.0774**	39%	29%
	MEDIAN	**0.72404**	**3305.314**	**MEDIAN**	**0.79137**	**3994.6565**	9%	21%
	SAMPLE#3			SAMPLE#4			ENHENCEMENT	
	SPECIMEN	EXTENSION[IN]	LOAD [LBF]	SPECIMEN	EXTENSION[IN]	LOAD [LBF]	EXT. %	LOADS %
BOLTS OFFSET FROM CENTER	D3.2	0.67113	2182.483	D3.2	2.15873	4452.8268	222%	104%
	R3.2	0.82313	1629.994	R3.2	1.32428	2159.3404	61%	32%
	P3.2	0.98401	2594.706	P3.2	1.22897	3548.3362	25%	37%
	MEAN	**0.82609**	**2135.728**	**MEAN**	**1.57066**	**3386.8345**	90%	59%
	MEDIAN	**0.82313**	**2182.483**	**MEDIAN**	**1.32428**	**3548.3362**	61%	63%

4.6 Comparison Between SAMPLE#1 and SAMPLE#2

To visualize the differences between paired samples, they are merged into single tables and graph figures. The difference between loads, extensions are shown in numbers as well percentage wise The following table 4-2 shows load difference between sample 1 and 2 as well extension at maximum load.

Table 4-6 SAMPLE#1 & SAMPLE#2

SAMPLE	EXTENSION AT MAX. LOAD(IN)	MAX. LOAD(LBF)
SAMPLE#1	0.75	3344
SAMPLE#2	1.24	4299
Difference	**0.49**	**955**
		+29%

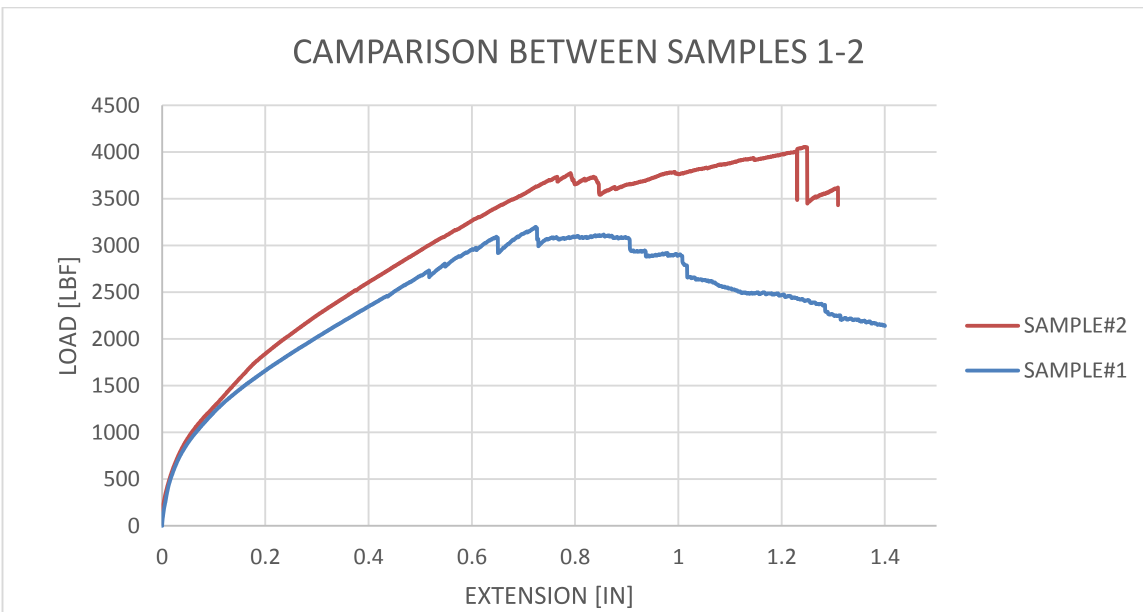


Figure 4-9 Comparison between SAMPLE#1 and SAMPLE#2

From the table and figure, it can be seen that SAMPLE#2 has taken the higher load. This enhancement in capacity is nearly 29% more than SAMPLE#1. Additionally, the major crack in SAMPLE#1 occurs in a large extension.

4.7 Comparison Between SAMPLE#3 and SAMPLE#4

Table 4-7 Comparison of SAMPLE#3 and SAMPLE#4

SAMPLE	EXTENSION AT MAX. LOAD	MAX. LOAD(LBF)
SAMPLE#3	0.83	2136
SAMPLE#4	2.07	3387
Difference	**1.24**	**1251**
		+59%

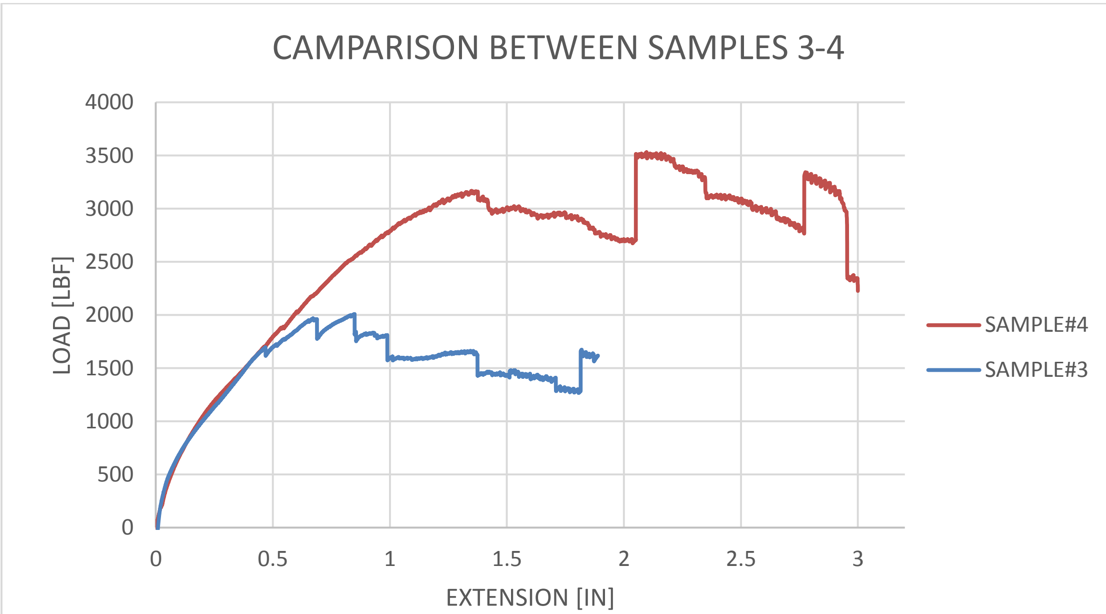


Figure 4-10 Comparison between SAMPLE#3 and SAMPLE#4

In this pair, the difference between samples is as high as 59%. These samples as explained Chapter III are in the extreme bending moment due the large moment arm. This moment arm is the distance of bolts from sheathing. Metal plate connector acts effectively when high tensile stress is present and in SAMPLE#4 it is clearly visible how magnificently changes the results. Additionally, the extension length for SAMPLE#4 is far large than SAMPLE#3, and the major cracks occur at a large extension.

From the comparison of all samples, it seems that SAMPLE#2 has the highest load capacity. The reason for having high capacity is because of Metal Plate Connector and failure along the edge. The theoretical failure approach in Chapter III indicated that failure along the edge has higher capacity than the cross-wise bending. So, this can also be seen in these test results that failure along the edge gives higher strength. Coming back to SAMPLE#2, the combined strength of failure mode and metal plate connector allows this sample to resist higher loads than all other samples.

CHAPTER V

CONCLUSION

5.1 Failure Modes of Sill Plate

The findings in (Caprolu, Källsner, Girhammar, & Vessby, 2012) explained the factors affecting sill plate failure modes. Among most important are nail spacing, washer size, bolt offset from sheathing and pith orientation. These factors are the primary consideration in the determination of failure modes. By taking these factors into account, it somehow explains the failures of sill plates that were discussed in Chapter I.

5.2 Metal Plate in Sill Plate Conclusion

The main focus of this research was study of sill plate behavior with the addition of metal plate connectors, and how much this configuration can enhance the ultimate capacity of the sill plate. Test results between samples with and without metal plate explained very well that addition of metal plate significantly contributes to the tensile strength of plate in crosswise bending. It was also found that metal plate connector acts more efficiently when bolts were placed at the center of the sill plate width. Metal plate connectors change the failure mode from the bottom face to along the edge. In Chapter III theoretical approach to failure modes also validated the outcome that failure mode 2 (failure along the edge) has higher load strength.

5.3 Overall Conclusion and Recommendations

It has been determined that sill plate is a crucial element in wood structures. Failure of a sill plate causes the wall to collapse. The factors affecting failure modes should be always considered in design and construction. Mostly, pith orientation is not focused during construction or design, but results indicated how this can contribute to the overall capacity of the sill plate. Findings in this research recommend orienting the member pith downward during the construction phase of sill plate to take more capacity in bending stresses. In this report, it was discussed that bending stress is a function of moment arm (distance from sheathing to bolts) and washer size. One of the solution to enhance the bending capacity of sill plate in this report was providing a metal connector on the bottom face of the sill plate. By taking this into account, if metal plate connectors are present in sill plate then small washer size can be used. Findings indicated that bolts at center configuration has higher capacity than those further far. Metal plate contribution is considered to be little but crucial changes that increased the capacity nearly by 60% in maximum bending failure and almost 30% of failures along the edge. This research, recommends usage of the metal plate connector on sill plate for buildings with partially anchored shear wall subjected to uplift pressure. To get the ultimate capacity for the shear wall in its foundation, this research recommends placing the bolts at the center of sill plates, metal plate at the bottom face and small but thick washers size. At the present time cost of metal plate connectors is less than a dollar, and constructability is extremely easy but its enhancement in severe loading can be lifesaving.

5.4 Future Research Works

In this research, nail spacing was adequate so that failure in nailing to sheathing was avoided in all test. As well the sheathing was made of steel sheet. Future work could be tests of sill plate with long plywood sheathing with nail spacing exceeding 2 inches more close to actual site construction. It will be necessary to see what will failure mode controls after the addition of metal plate connector.

REFERENCES

Blue Ridge Timberwright. (n.d.). Retrieved from http://www.brtw.com/historyoftimberframing.php

Breyer, D., Fridley, K., & Cobeen, K. (2007). *Design of Wood Structures.* New York: McGraw-Hill.

Caprolu, G. (2014). *Evaluation of Splitting Capacity of Bottom Rails in Partially Anchored Timbr Frame Shear Walls.* Luleå: Luleå University of Technology.

Caprolu, G., Källsner, B., Girhammar, U. A., & Vessby, J. (2012). ANALYTICAL AND EXPERIMENTAL EVALUATION OF THE CAPACITY OF THE BOTTOM RAIL IN PARTIALLY ANCHORED SHEAR WALLS. *World Engineering on Timber Engineering.* Auckland: WCTE.

Code, I. B. (2012). Wood. In I. B. Code, *International Building Code.* International Building Code.

FEMA. (2015, March 19). *Earthquake Hazard Maps*. Retrieved from Federal Emergency Management Agency: https://www.fema.gov/earthquake-hazard-maps

Foundation Bolts in Houses. (n.d.). Retrieved from Bay Area Retrofit: http://bayarearetrofit.com/bolting/

HOME BUILDER’S GUIDE TO COASTAL CONSTRUCTION. (2010, December). Retrieved from Federal Emergency Management Agency: http://www.fema.gov/media-library-data/20130726-1537-20490-6326/fema499_4_1.pdf

Pidwirny, M. (2008, May 20). *The Encyclopedia of Earth*. Retrieved from Atmospheric Science: http://www.eoearth.org/view/article/156662/

Rainer, J., & Karacabeyli, E. (2000, December). *nsuring good seismic performance with platform-frame wood housing*. Retrieved from National Research Council Canada: http://www.nrc-cnrc.gc.ca/ctu-sc/ctu_sc_n45

Retrofit, B. A. (n.d.). *Foundation Bolts in Houses*. Retrieved from Bay Area Retrofit: http://bayarearetrofit.com/bolting/

Safety, I. I. (2012). *Making the Right Connections*. Retrieved 7 8, 2015, from Insurance Institute for Business & Home Safety: https://www.disastersafety.org/high_winds/making-the-right-connections/attachment/continuous-load-path-illustration-2/

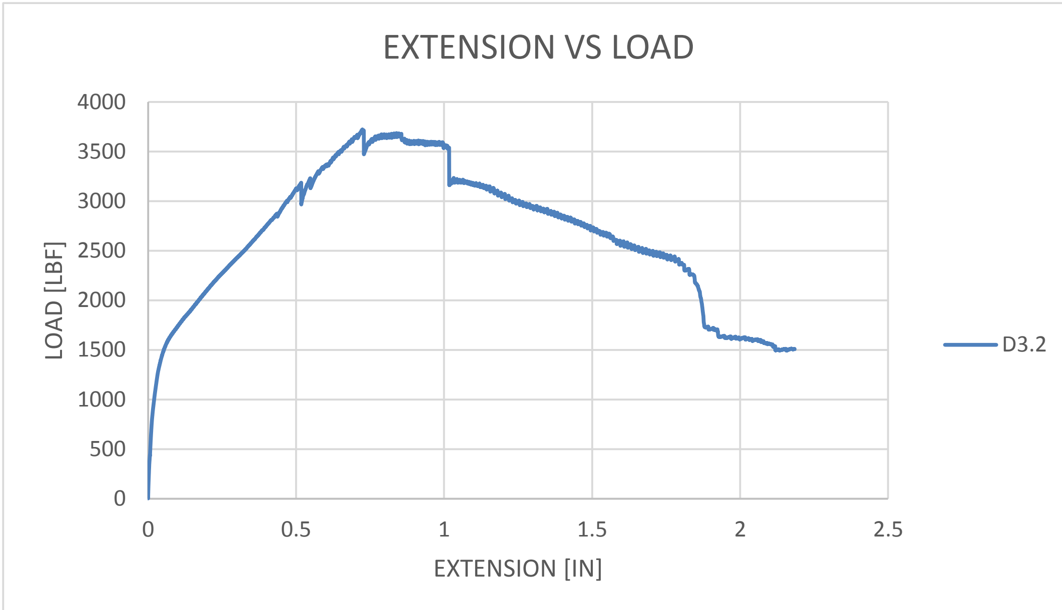
Strong-Tie, S. (n.d.). *Stud to Sill Plate*. Retrieved from Simpson Strong-Tie: http://www.strongtie.com/products/highwind/Stud-Sillplate.html?source=highwindnav

(2014). *Tornado Damage Assessment in the aftermath of the May 20th 2013 Moore Oklahoma Tornado.*

Webmaster, O. (2015, May 18). *Monthly and Annual Tornado Statistics*. Retrieved from National Oceanic And Atmospheric Administration: http://www.srh.noaa.gov/oun/?n=tornadodata-ok-monthlyannual

APPENDICES

D3.2



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Extension : Extension	18.39066	mm
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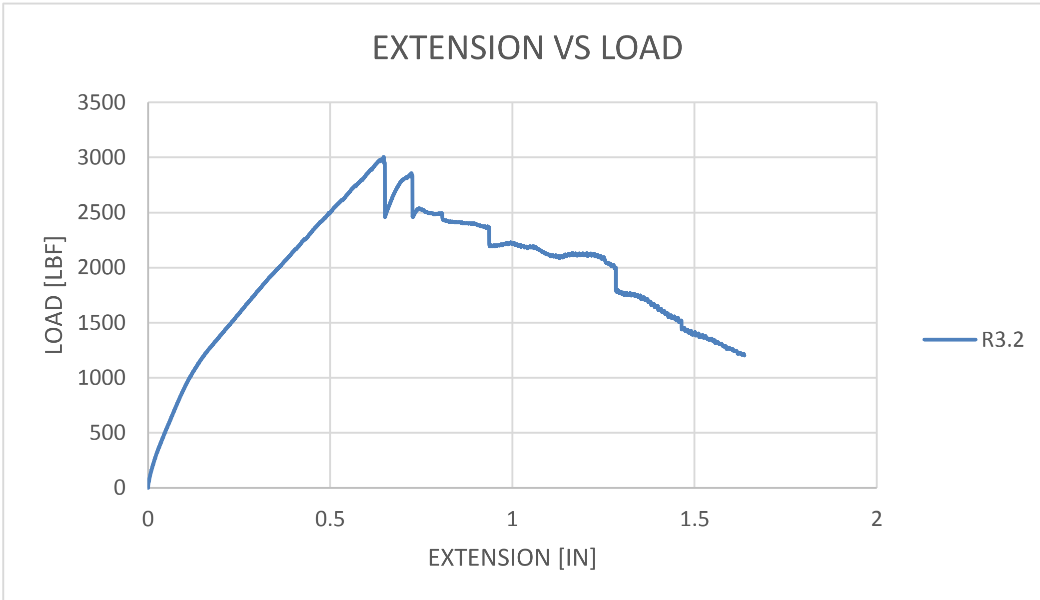
Maximum Load : Moment at Maximum Load	736.0827	N-m
Maximum Load : Tensile stress at Maximum Load	0.47533	MPa
Moment : Moment	736.0827	N-m

R3.2



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Extension : Extension	0.64703	in
Maximum Load : Maximum Load	3003.8799	lbf
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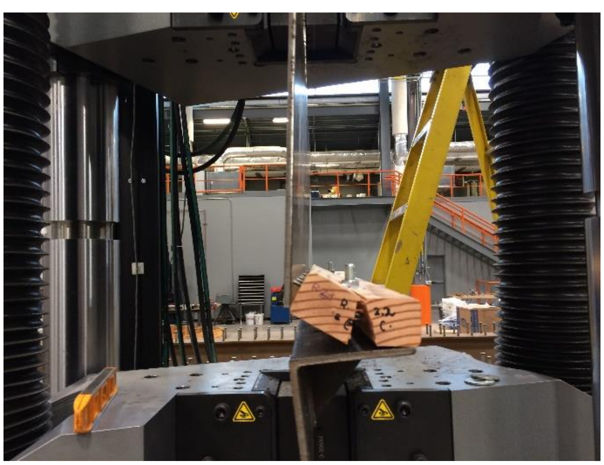
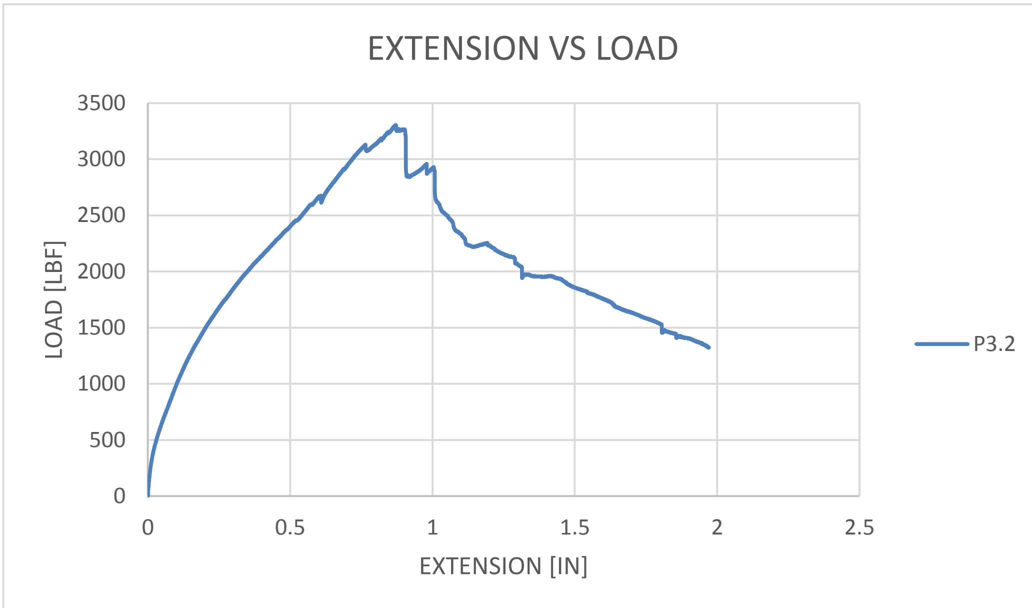
Maximum Load : Tensile stress at Maximum Load	55.6274	psi
Moment : Moment	5256.7904	lbf-in



P3.2



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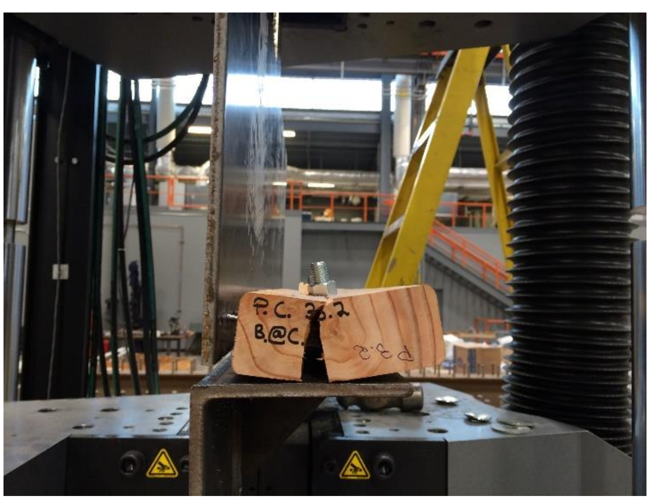
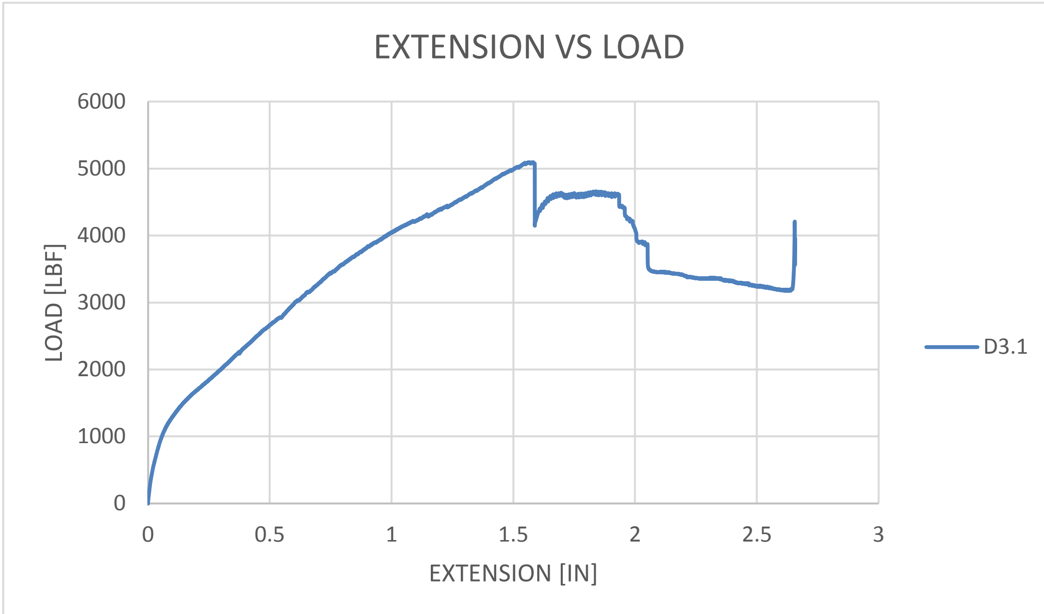
Strain : Tensile strain (Extension) gauge length	1.75	in
Test : Rate 1	0.1	in/min
Extension : Extension	0.87102	in
Maximum Load : Maximum Load	3305.3136	lbf
Maximum Load : Moment at Maximum Load	5784.29946	lbf-in
Maximum Load : Tensile stress at Maximum Load	61.2095	psi
Moment : Moment	5784.2995	lbf-in



D3.1



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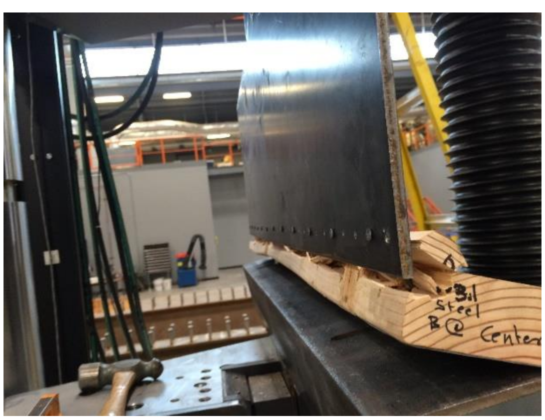
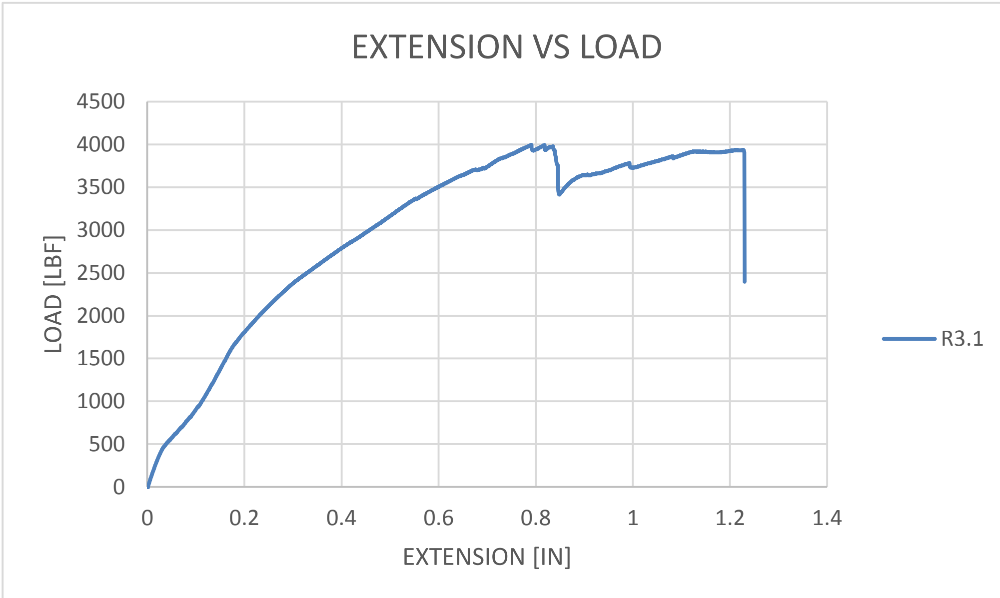
Strain : Tensile strain (Extension) gauge length	1.75	in
Test : Rate 1	0.1	in/min
Extension : Extension	2.73943	in
Maximum Load : Maximum Load	349887.7473	lbf
Maximum Load : Moment at Maximum Load	612303.5829	lbf-in
Maximum Load : Tensile stress at Maximum Load	6479.4026	psi
Moment : Moment	612303.5829	lbf-in



R3.1



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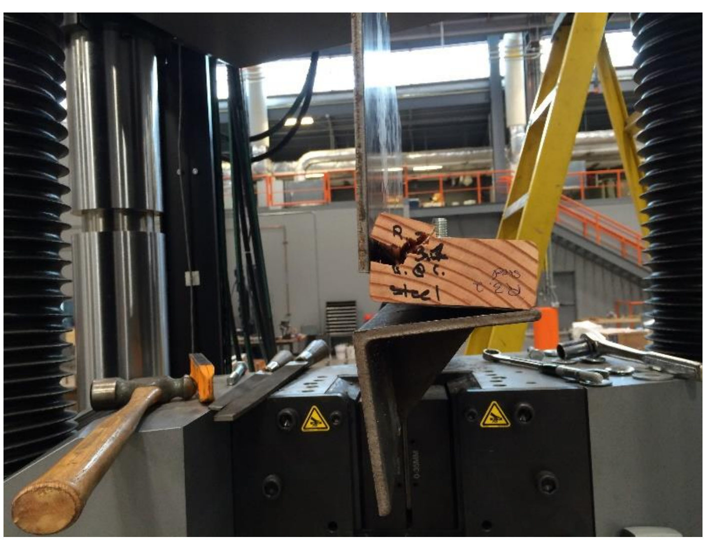
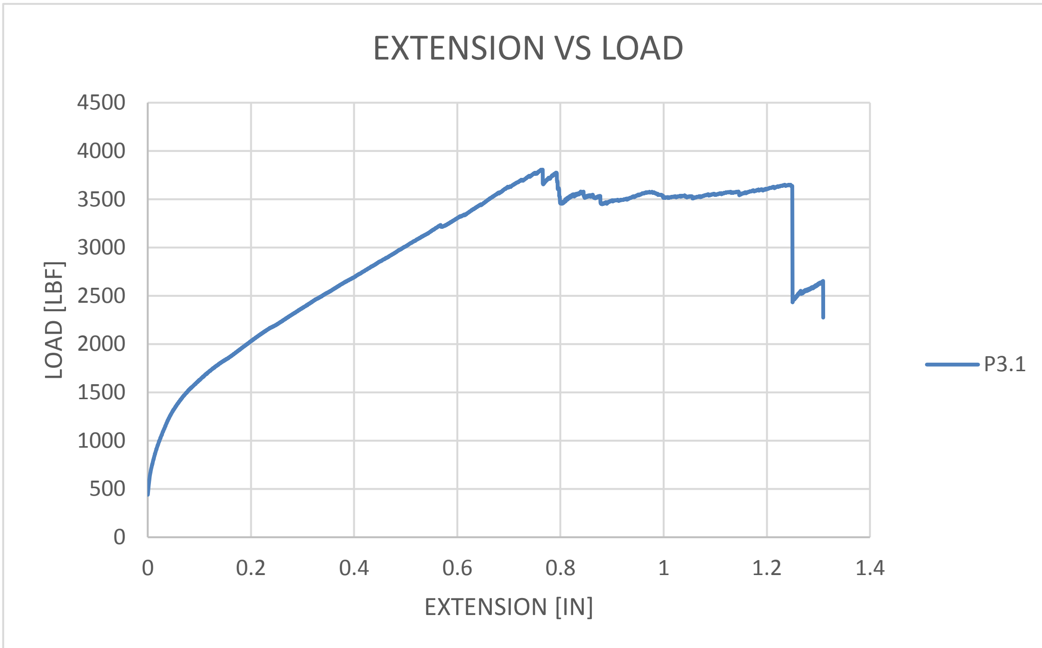
Strain : Tensile strain (Extension) gauge length	1.75	in
Test : Rate 1	0.1	in/min
Extension : Extension	0.79137	in
Maximum Load : Maximum Load	3994.6565	lbf
Maximum Load : Moment at Maximum Load	6990.64952	lbf-in
Maximum Load : Tensile stress at Maximum Load	73.9751	psi
Moment : Moment	6990.6495	lbf-in

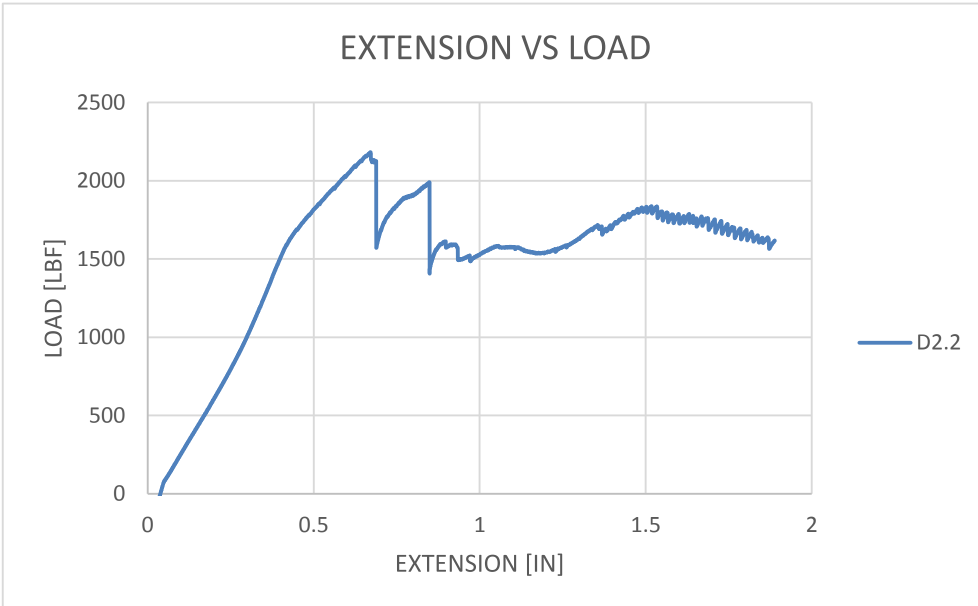


P3.1



General : Specimen number (included)	2	
Strain : Tensile strain (Extension) gauge length	1.75	in
Test : Rate 1	0.1	in/min
Extension : Extension	0.76287	in
Maximum Load : Maximum Load	3807.4645	lbf
Maximum Load : Moment at Maximum Load	6663.06335	lbf-in
Maximum Load : Tensile stress at Maximum Load	70.5086	psi
Moment : Moment	6663.0633	lbf-in

D2.2



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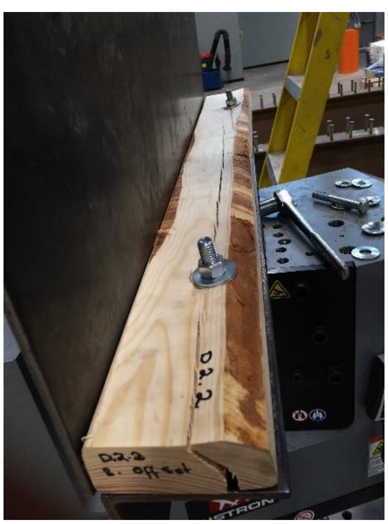
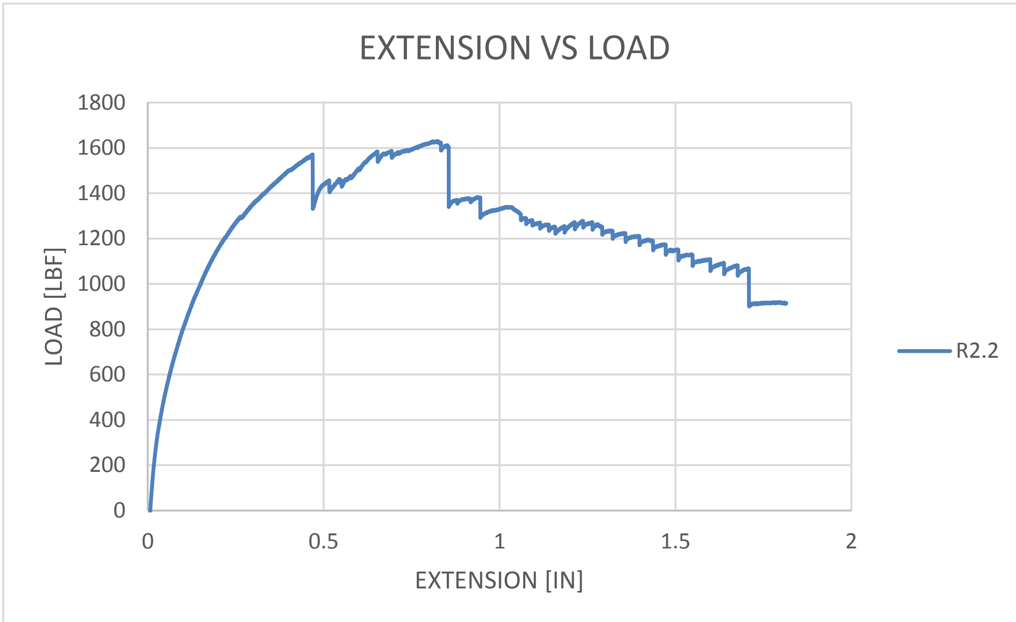
Extension : Extension	0.67113	in
Maximum Load : Maximum Load	2182.4828	lbf
Maximum Load : Moment at Maximum Load	3819.3453	lbf-in
Maximum Load : Tensile stress at Maximum Load	40.4163	psi
Moment : Moment	3819.3453	lbf-in

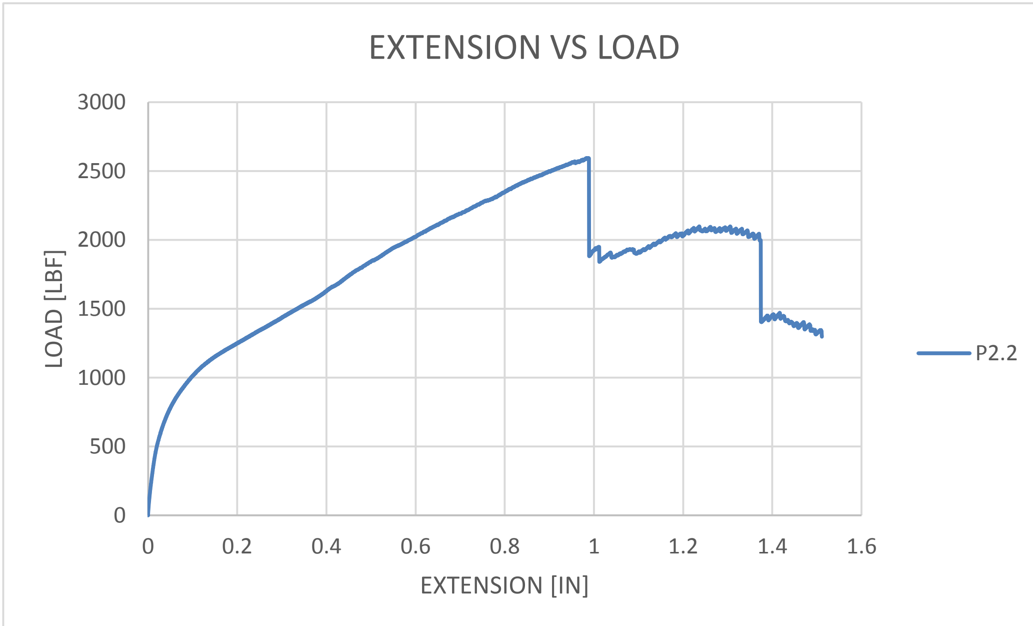


R2.2



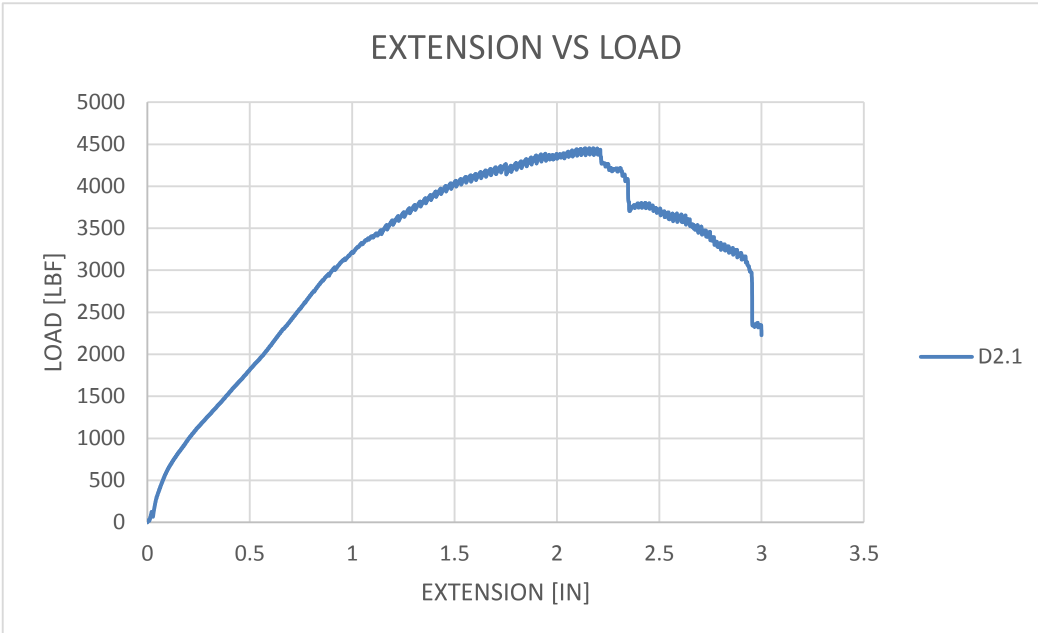
General : Specimen number (included)	2	
Strain : Tensile strain (Extension) gauge length	2.25	in
Test : Rate 1	0.1	in/min
Extension : Extension	0.82313	in
Maximum Load : Maximum Load	1629.9939	lbf
Maximum Load : Moment at Maximum Load	2852.48965	lbf-in
Maximum Load : Tensile stress at Maximum Load	30.1851	psi
Moment : Moment	2852.4897	lbf-in

P2.2



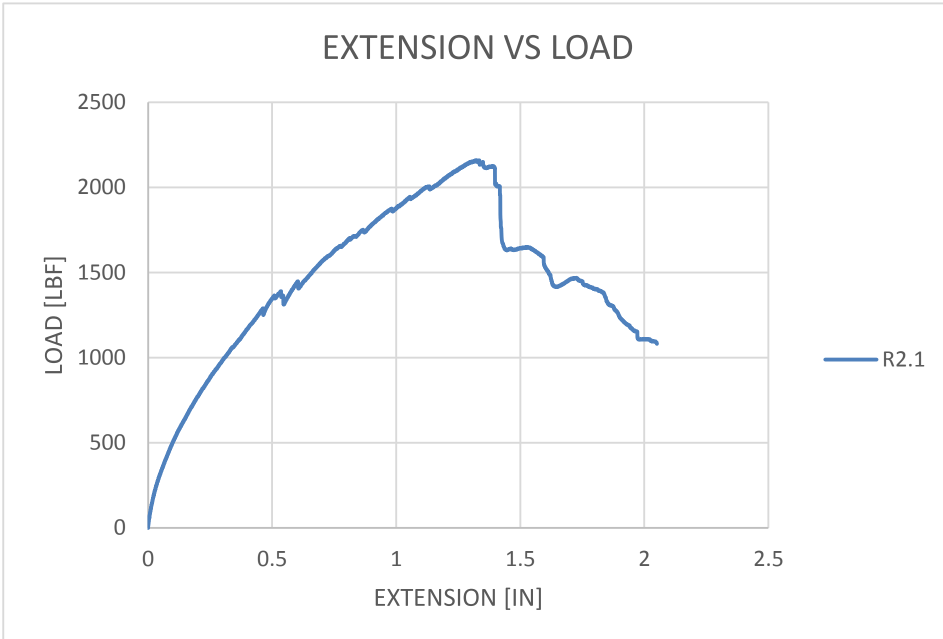
General : Specimen number (included)	3	
Strain : Tensile strain (Extension) gauge length	2.25	in
Test : Rate 1	0.1	in/min
Extension : Extension	0.98401	in
Maximum Load : Maximum Load	2594.7064	lbf
Maximum Load : Moment at Maximum Load	4540.73673	lbf-in
Maximum Load : Tensile stress at Maximum Load	48.0501	psi
Moment : Moment	4540.7367	lbf-in

D2.1



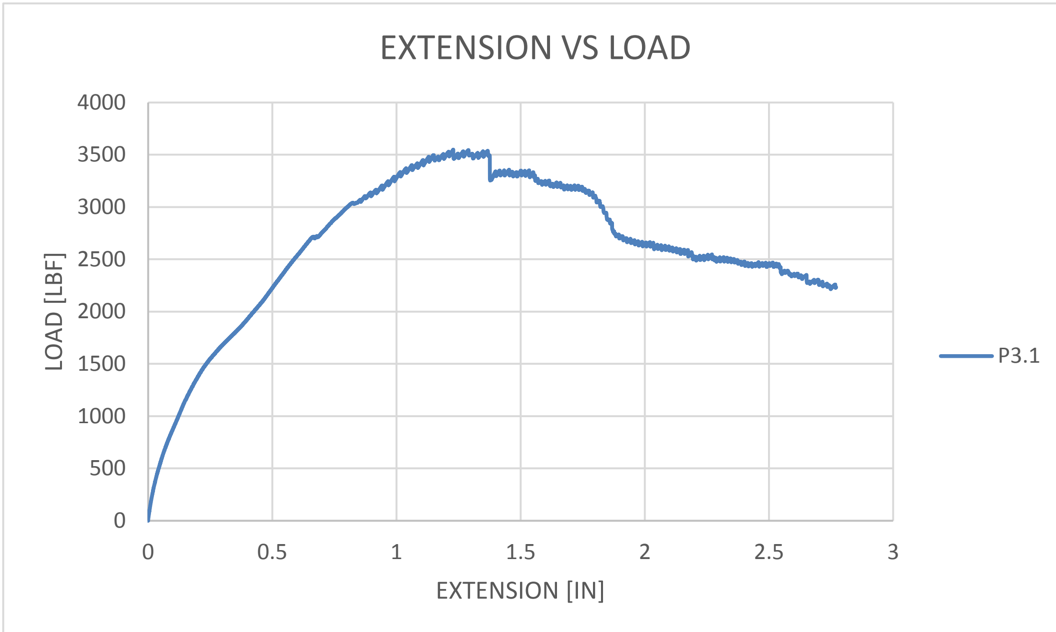
General : Specimen number (included)	1	
Strain : Tensile strain (Extension) gauge length	2.25	in
Test : Rate 1	0.1	in/min
Extension : Extension	2.15873	in
Maximum Load : Maximum Load	4452.8268	lbf
Maximum Load : Moment at Maximum Load	7792.44747	lbf-in
Maximum Load : Tensile stress at Maximum Load	82.4598	psi
Moment : Moment	7792.4475	lbf-in

R2.1



General : Specimen number (included)	2	
Strain : Tensile strain (Extension) gauge length	2.25	in
Test : Rate 1	0.1	in/min
Extension : Extension	1.32428	in
Maximum Load : Maximum Load	2159.3404	lbf
Maximum Load : Moment at Maximum Load	3778.84601	lbf-in
Maximum Load : Tensile stress at Maximum Load	39.9878	psi
Moment : Moment	3778.846	lbf-in

P2.1



General : Specimen number (included)	3	
Strain : Tensile strain (Extension) gauge length	2.25	in
Test : Rate 1	0.1	in/min
Extension : Extension	1.22897	in
Maximum Load : Maximum Load	3548.3362	lbf
Maximum Load : Moment at Maximum Load	6209.58904	lbf-in
Maximum Load : Tensile stress at Maximum Load	65.7099	psi
Moment : Moment	6209.589	lbf-in

IBC CHAPTER 23

2304.9.1 Fastener requirements.
Connections for wood members shall be designed in accordance with the appropriate methodology in Section 2301.2. The number and size of fasteners connecting wood members shall not be less than that set forth in Table 2304.9.1.

TABLE 2304.9.1 FASTENING SCHEDULE

CONNECTION	FASTENING[a, m]	LOCATION
1. Joist to sill or girder	3 - 8d common ($2^1/_2$″ × 0.131″) 3 - 3″ × 0.131″ nails 3 - 3″ 14 gage staples	toenail
2. Bridging to joist	2 - 8d common ($2^1/_2$″ × 0.131″) 2 - 3″ × 0.131″ nails 2 - 3″ 14 gage staples	toenail each end
3. 1″ × 6″ subfloor or less to each joist	2 - 8d common ($2^1/_2$″ × 0.131″)	face nail
4. Wider than 1″ × 6″ subfloor to each joist	3 - 8d common ($2^1/_2$″ × 0.131″)	face nail
5. 2″ subfloor to joist or girder	2 - 16d common ($3^1/_2$″ × 0.162″)	blind and face nail

6. Sole plate to joist or blocking	16d ($3^1/_2$″ × 0.135 ″) at 16″ o.c. 3″ × 0.131″ nails at 8″ o.c. 3″ 14 gage staples at 12″ o.c.	typical face nail
Sole plate to joist or blocking at braced wall panel	3- 16d ($3^1/_2$″ × 0.135″) at 16″ o.c. 4 - 3″ × 0.131″ nails at 16″ o.c. 4 - 3″ 14 gage staples at 16″ o.c.	braced wall panels
7. Top plate to stud	2 - 16d common ($3^1/_2$″ × 0.162″) 3 - 3″ × 0.131″ nails 3 - 3″ 14 gage staples	end nail
8. Stud to sole plate	4 - 8d common ($2^1/_2$″ × 0.131″) 4 - 3″ × 0.131″ nails 3 - 3″ 14 gage staples	toenail
	2 - 16d common ($3^1/_2$″ × 0.162″) 3 - 3″ × 0.131″ nails 3 - 3″ 14 gage staples	end nail

9. Double studs	16d ($3^1/_2$″ × 0.135″) at 24″ o.c. 3″ × 0.131″ nail at 8″ o.c. 3″ 14 gage staple at 8″ o.c.	face nail
10. Double top plates	16d ($3^1/_2$″ × 0.135″) at 16″ o.c. 3″ × 0.131″ nail at 12″ o.c. 3″ 14 gage staple at 12″ o.c.	typical face nail
Double top plates	8 - 16d common ($3^1/_2$″ × 0.162″) 12 - 3″ × 0.131″ nails 12 - 3″ 14 gage staples	lap splice
11. Blocking between joists or rafters to top plate	3 - 8d common ($2^1/_2$″ × 0.131″) 3 - 3″ × 0.131″ nails 3 - 3″ 14 gage staples	toenail
12. Rim joist to top plate	8d ($2^1/_2$″ × 0.131″) at 6″ o.c. 3″ × 0.131″ nail at 6″ o.c. 3″ 14 gage staple at 6″ o.c.	toenail

13. Top plates, laps and intersections	2 - 16d common ($3^1/_2$″ × 0.162″) 3 - 3″ × 0.131″ nails 3 - 3″ 14 gage staples	face nail
14. Continuous header, two pieces	16d common ($3^1/_2$″ × 0.162″)	16″ o.c. along edge
15. Ceiling joists to plate	3 - 8d common ($2^1/_2$″ × 0.131″) 5 - 3″ × 0.131″ nails 5 - 3″ 14 gage staples	toenail
16. Continuous header to stud	4 - 8d common ($2^1/_2$″ × 0.131″)	toenail

(continued)

TABLE 2304.9.1—continued FASTENING SCHEDULE

CONNECTION	FASTENING[a, m]	LOCATION
17. Ceiling joists, laps over partitions (see Section 2308.10.4.1, Table 2308.10.4.1)	3 - 16d common ($3^1/_2$″ × 0.162″) minimum, Table 2308.10.4.1 4 - 3″ × 0.131″ nails 4 - 3″ 14 gage staples	face nail
18. Ceiling joists to parallel rafters (see Section 2308.10.4.1,	3 - 16d common ($3^1/_2$″ × 0.162″) minimum, Table 2308.10.4.1 4 - 3″ × 0.131″ nails 4 - 3″ 14 gage staples	face nail

Table 2308.10.4.1)		
19. Rafter to plate (see <u>Section 2308.10.1</u>, Table 2308.10.1)	3 - 8d common (2$^{1}/_{2}$″ × 0.131″) 3 - 3″ × 0.131″ nails 3 - 3″ 14 gage staples	toenail
20. 1″ diagonal brace to each stud and plate	2 - 8d common (2$^{1}/_{2}$″ × 0.131″) 2 - 3″ × 0.131″ nails 3 - 3″ 14 gage staples	face nail
21. 1″ × 8″ sheathing to each bearing	3 - 8d common (2$^{1}/_{2}$″ × 0.131″)	face nail
22. Wider than 1″ × 8″ sheathing to each bearing	3 - 8d common (2$^{1}/_{2}$″ × 0.131″)	face nail
23. Built-up corner studs	16d common (3$^{1}/_{2}$″ × 0.162″) 3″ × 0.131″ nails 3″ 14 gage staples	24″ o.c. 16″ o.c. 16″ o.c.
24. Built-up girder and beams	20d common (4″ × 0.192″) 32″ o.c. 3″ × 0.131″ nail at 24″ o.c. 3″ 14 gage staple at 24″ o.c.	face nail at top and bottom staggered on opposite sides
	2 - 20d common (4″ × 0.192″) 3 - 3″ × 0.131″ nails 3 - 3″ 14 gage staples	face nail at ends and at each splice
25. 2″ planks	16d common (3$^{1}/_{2}$″ × 0.162″)	at each bearing
26. Collar tie to rafter	3 - 10d common (3″ × 0.148″) 4 - 3″ × 0.131″ nails 4 - 3″ 14 gage staples	face nail
27. Jack rafter to hip	3 - 10d common (3″ × 0.148″) 4 - 3″ × 0.131″ nails 4 - 3″ 14 gage staples	toenail

	2 - 16d common ($3^1/_2$″ × 0.162″) 3 - 3″ × 0.131″ nails 3 - 3″ 14 gage staples	face nail
28. Roof rafter to 2-by ridge beam	2 - 16d common ($3^1/_2$″ × 0.162″) 3 - 3″ × 0.131″ nails 3 - 3″ 14 gage staples	toenail
	2 -16d common ($3^1/_2$″ × 0.162″) 3 - 3″ × 0.131″ nails 3 - 3″ 14 gage staples	face nail
29. Joist to band joist	3 - 16d common ($3^1/_2$″ × 0.162″) 4 - 3″ × 0.131″ nails 4 - 3″ 14 gage staples	face nail

(continued)

TABLE 2304.9.1—continued FASTENING SCHEDULE

CONNECTION	FASTENING[a, m]		LOCATION
30. Ledger strip	3 - 16d common ($3^1/_2$″ × 0.162″) 4 - 3″ × 0.131″ nails 4 - 3″ 14 gage staples		face nail at each joist
31. Wood structural panels and particleboard[b] Subfloor, roof and wall sheathing (to framing)	$^1/_2$″ and less	6d[c, 1] $2^3/_8$″ × 0.113″ nail[n]	
		$1^3/_4$″ 16 gage[o]	
	$^{19}/_{32}$″ to $^3/_4$″	8d[d] or 6d[e]	
		$2^3/_8$″ × 0.113″ nail[p]	
		2″ 16 gage staple[p]	

	7/8″ to 1″	8d[c]	
	1 1/8″ to 1 1/4″	10d[d] or 8d[e]	
Single floor (combination subfloor-underlayment to framing)	3/4″ and less	6d[e]	
	7/8″ to 1″	8d[e]	
	1 1/8″ to 1 1/4″	10d[d] or 8d[e]	
32. Panel siding (to framing)	1/2″ or less	6d[f]	
	5/8″	8d[f]	
33. Fiberboard sheathing[g]	1/2″	No. 11 gage roofing nail[h]	
		6d common nail (2″ × 0.113″)	
		No. 16 gage staple[i]	
	25/32″	No. 11 gage roofing nail[h]	
		8d common nail (2 1/2″ × 0.131″)	
		No. 16 gage staple[i]	
34. Interior paneling	1/4″	4d[j]	
	3/8″	6d[k]	

DOUGLAS FIR PROPERTIES

Table 4A (Cont.) **Reference Design Values for Visually Graded Dimension Lumber (2" - 4" thick)[1,2,3]**

(All species except Southern Pine — see Table 4B) (Tabulated design values are for normal load duration and dry service conditions. See NDS 4.3 for a comprehensive description of design value adjustment factors.)

USE WITH TABLE 4A ADJUSTMENT FACTORS

Species and commercial grade	Size classification	Design values in pounds per square inch (psi)							Grading Rules Agency
		Bending F_b	Tension parallel to grain F_t	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of Elasticity E		
							E	E_{min}	
BEECH-BIRCH-HICKORY									
Select Structural	2" & wider	1,450	850	195	715	1,200	1,700,000	620,000	NELMA
No.1	2" & wider	1,050	600	195	715	950	1,600,000	580,000	NELMA
No.2	2" & wider	1,000	600	195	715	750	1,500,000	550,000	NELMA
No.3	2" & wider	575	350	195	715	425	1,300,000	470,000	NELMA
Stud	2" & wider	775	450	195	715	475	1,300,000	470,000	NELMA
Construction	2" - 4" wide	1,150	675	195	715	1,000	1,400,000	510,000	NELMA
Standard	2" - 4" wide	650	375	195	715	775	1,300,000	470,000	NELMA
Utility	2" - 4" wide	300	175	195	715	500	1,200,000	440,000	NELMA
COTTONWOOD									
Select Structural	2" & wider	875	525	125	320	775	1,200,000	440,000	NSLB
No.1	2" & wider	625	375	125	320	625	1,200,000	440,000	NSLB
No.2	2" & wider	625	350	125	320	475	1,100,000	400,000	NSLB
No.3	2" & wider	350	200	125	320	275	1,000,000	370,000	NSLB
Stud	2" & wider	475	275	125	320	300	1,000,000	370,000	NSLB
Construction	2" - 4" wide	700	400	125	320	650	1,000,000	370,000	NSLB
Standard	2" - 4" wide	400	225	125	320	500	900,000	330,000	NSLB
Utility	2" - 4" wide	175	100	125	320	325	900,000	330,000	NSLB
DOUGLAS FIR-LARCH									
Select Structural	2" & wider	1,500	1,000	180	625	1,700	1,900,000	690,000	WCLIB WWPA
No.1 & Btr	2" & wider	1,200	800	180	625	1,550	1,800,000	660,000	WCLIB WWPA
No.1	2" & wider	1,000	675	180	625	1,500	1,700,000	620,000	WCLIB WWPA
No.2	2" & wider	900	575	180	625	1,350	1,600,000	580,000	WCLIB WWPA
No.3	2" & wider	525	325	180	625	775	1,400,000	510,000	WCLIB WWPA
Stud	2" & wider	700	450	180	625	850	1,400,000	510,000	WCLIB WWPA
Construction	2" - 4" wide	1,000	650	180	625	1,650	1,500,000	550,000	WCLIB WWPA
Standard	2" - 4" wide	575	375	180	625	1,400	1,400,000	510,000	WCLIB WWPA
Utility	2" - 4" wide	275	175	180	625	900	1,300,000	470,000	WCLIB WWPA
DOUGLAS FIR-LARCH (NORTH)									
Select Structural	2" & wider	1,350	825	180	625	1,900	1,900,000	690,000	NLGA
No.1 & Btr	2" & wider	1,150	750	180	625	1,800	1,800,000	660,000	NLGA
No.1/No.2	2" & wider	850	500	180	625	1,400	1,600,000	580,000	NLGA
No.3	2" & wider	475	300	180	625	825	1,400,000	510,000	NLGA
Stud	2" & wider	650	400	180	625	900	1,400,000	510,000	NLGA
Construction	2" - 4" wide	950	575	180	625	1,800	1,500,000	550,000	NLGA
Standard	2" - 4" wide	525	325	180	625	1,450	1,400,000	510,000	NLGA
Utility	2" - 4" wide	250	150	180	625	950	1,300,000	470,000	NLGA
DOUGLAS FIR-SOUTH									
Select Structural	2" & wider	1,350	900	180	520	1,600	1,400,000	510,000	WWPA
No.1	2" & wider	925	600	180	520	1,450	1,300,000	470,000	WWPA
No.2	2" & wider	850	525	180	520	1,350	1,200,000	440,000	WWPA
No.3	2" & wider	500	300	180	520	775	1,100,000	400,000	WWPA
Stud	2" & wider	675	425	180	520	850	1,100,000	400,000	WWPA
Construction	2" - 4" wide	975	600	180	520	1,650	1,200,000	440,000	WWPA
Standard	2" - 4" wide	550	350	180	520	1,400	1,100,000	400,000	WWPA
Utility	2" - 4" wide	250	150	180	520	900	1,000,000	370,000	WWPA
EASTERN HEMLOCK-BALSAM FIR									
Select Structural	2" & wider	1,250	575	140	335	1,200	1,200,000	440,000	NELMA NSLB
No.1	2" & wider	775	350	140	335	1,000	1,100,000	400,000	NELMA NSLB
No.2	2" & wider	575	275	140	335	825	1,100,000	400,000	NELMA NSLB
No.3	2" & wider	350	150	140	335	475	900,000	330,000	NELMA NSLB
Stud	2" & wider	450	200	140	335	525	900,000	330,000	NELMA NSLB
Construction	2" - 4" wide	675	300	140	335	1,050	1,000,000	370,000	NELMA NSLB
Standard	2" - 4" wide	375	175	140	335	850	900,000	330,000	NELMA NSLB
Utility	2" - 4" wide	175	75	140	335	550	800,000	290,000	NELMA NSLB

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