8-2014

HOLLOW MASSIVE TIMBER PANELS: A HIGH-PERFORMANCE, LONG-SPAN ALTERNATIVE TO CROSS LAMINATED TIMBER

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HOLLOW MASSIVE TIMBER PANELS: A HIGH-PERFORMANCE, LONG-SPAN ALTERNATIVE TO CROSS LAMINATED TIMBER

A Thesis
Presented to
the Graduate School of
Clemson University

In Partial Fulfillment
of the Requirements for the Degree
Master of Science
Civil Engineering

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by
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August 2014

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ABSTRACT

Since the development of Cross Laminated Timber (CLT), there has been a surge in interest in massive timber buildings. Furthermore, recent conceptual and feasibility designs of massive timber towers of 30 or more stories indicate that performance of mass timber structural elements can compete with other building materials in the commercial industry (MGB Architecture and Design et al.). However, in order for massive timber to penetrate the commercial market even further, a solution is needed for long-span massive timber floor systems. Unfortunately, CLT falls short in this area and is unable to span long distances. The hollow massive timber (HMT) panel presented in this thesis offers one potential long-span solution.

Optimal panel geometries and wood properties were investigated through several numerical parametric studies in which each variable related to the structural properties was analyzed. It was established that two 3-ply flanges be used with an unbalanced orientation - double outside layer of Grade #2 Southern Pine lumber and an inner crosswise layer of Grade #3 Southern Pine lumber. Additionally, it was recommended that 2½-inch wide glulam beams be used as web members spaced at 32 inches on-center. It was also determined that the connector stiffness used for the flange-to-web connection has the greatest impact on panel performance. Therefore, experimental shear tests were performed to evaluate a range of different connection configurations. These tests resulted in two possible recommended configurations. The first configuration utilized Emulsion Polymer Isocyanate adhesive to join the upper surface of the glulam to the lower surface of the top flange and the lower surface of the glulam to the upper surface of the bottom

flange. The second configuration consisted of screwed connection with large screws passing through the top flange (side member) and embedded into the glulam web (main member) with or without an acoustical membrane at between the top flange and the web. The bottom flange connection was assumed to be the same connection as in the first configuration. Since the screwed connection is more expensive, screws should only be considered when the either field removal of the top flange is desired to have complete access into the voids or the addition of the membrane is needed to improve the acoustics or control the vibration of the HMT panel system.

By varying the depth of the final recommended panel, it is possible to achieve spans between 30 to 50 feet, which is more than adequate as a long-span solution. In addition to span, other important design requirements were considered, such as fire and acoustical performance, as well as constructability. In the end, the recommended HMT panel retained many benefits of CLT, while improving upon many of its shortcomings.

DEDICATION

I would like to dedicate this work to the most important things in my life, my awesome God, my wonderful wife Anna, and my loving family.

ACKNOWLEDGMENTS

I would first like to thank God for all the blessing, opportunities, grace and mercy He has given me. Also, I would like to thank Him for his magnificent creation and the ability to investigate and subdue it. It has been an extraordinary opportunity to be able to do this research.

I would like to thank my advisor Dr. Scott Schiff who went above and beyond his duties by putting in countless hours to help to complete this research project. Also, I would like to thank my other advisors, Dr. Weichiang Pang and Professor Dustin Albright, for advice, support and all the resources that I needed to complete this project. It was a pleasure working with all of you. I would like to thank the USDA for funding this project and making it possible to look for further solutions that can promote timber use. Also, I would like to thank Dr. Pat Layton for allowing me to use the wood shop in Lehotsky.

I wish to thank the following for donation of materials used in this project: Southern Pine Inspection Bureau, Collum's Lumber Products, Simpson Strong-Tie, My-Ti-Con Timber Connectors, TiConTec, SFS Intec, AkzoNobel, and Franklin Adhesives.

I would like to acknowledge others that were of tremendous support and help during this time. My wife, Anna, was always understanding, helpful and an encouraging to me though out many of the long work days and nights. I would also like to thank the Creative Inquiry team that put too many hours of volunteered help to make and break my massive timber panels. Thank you all for your help and influence in my life and I am truly grateful for your self-sacrifice and service.

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CHAPTER ONE

INTRODUCTION

Recent advances in engineered wood products, such as the development of Cross Laminated Timber (CLT), have sparked new ideas and innovation around the world, which has challenged engineers to go back to the drawing board and rethink the possibilities of this unique building material. One specific topic receiving much attention is tall wood buildings. Multiple different studies have been conducted in order to see what heights these wood buildings can achieve, and have shown that it is possible to reach 30 or more stories with wood (MGB Architecture and Design et al.). This is a remarkable revelation, which no one would have thought about 50 years ago. Discovering this potential proves that performance of wood can compete with other building materials in the commercial industry, and there is much innovation in wood construction left to be explored.

Many new products have been developed in the last decade to overcome traditional performance limitations of wood while advancing new and expanded uses. The invention of CLT in the 1990s helped spark these new developments due to its capabilities as a structural panel element. This addition has greatly increased the potential of the family of massive timber products. The growth of CLT has also opened many doors for further research and development. The research described in this thesis document, considers one such development: hollow massive timber (HMT) panels.

Along with searching for the potential of wood skyscrapers, there is another trend in research that can be identified. This trend is the search for a long-span massive timber floor system. Unfortunately, CLT falls short and is unreliable for spanning long distances. While CLT can be made as thick as desired, and can technically obtain whatever span length is required, it is economically infeasible to span more than 25 feet. With most architects desiring more than 30-foot span lengths, especially in commercial buildings, this leaves a gap for a much needed long-span solution. An economical long-span solution could greatly aid in the ability to penetrate the commercial building market with massive timber including the push for wood skyscrapers.

Just as with wood skyscrapers, surprising performance can be achieved in a massive timber floor system if wood is put together to its advantages. The performance and cost competitiveness of massive timber buildings are being examined, and it is believed that HMT panels may provide adequate answers to both.

Project Description

This project set out to examine an all-wood, long-span system. Hollow massive timber panels were determined to have a high potential and were therefore chosen to be investigated as a solution. First, the general feasibility of this concept was studied. Next, specific elements and geometries were researched to make up an efficient and effective system. Subsequently, attention was placed on finding solutions for a connection needed between the flange and the web in order to gain composite action. Furthermore, other topics pertaining to constructability, acoustics, and fire performance of the panel were considered when addressing which elements, geometries and flange to web connection should be chosen. Lastly, panel performances were predicted using analytical models of full-scale panels.

Objectives and Scope of Research

The project objectives were:

- Perform a literature review pertaining to massive timber products, connections and long-span systems,
- Conduct a parametric study to find geometry optimization,
- Conduct a parametric study to find which properties most affect overall bending performance,
- Perform small-scale experimental testing to gain knowledge of strength and stiffness properties for the connectors used for the flange-to-web connection,
- Identify concerns with panel performance and seek practical solutions,
- Model and predict performance of optimized panel, and
- Recommend the best options for manufacturing, design and construction of optimized hollow massive timber panels.

The scope of the research does not include full-scale panel tests. Future testing on panels will be useful for validating predictions of the refined model presented in this document. The scope of research also does not include in-depth testing of non-structural properties. That being said, these properties were acknowledged throughout the project as the HMT panels were developed.

Outline of Thesis

The subsequent chapters lay out concepts of massive timber panels, issues to be addressed, and the studies undertaken to solve these issues in order to present recommendations for a high-performance floor panel. Chapter 2 discusses concepts of basic timber properties, massive timber products and many recent innovations in the wood industry that make high-performance timber buildings possible. Furthermore, it will discuss examples of long-spanning systems and properties needed to obtain a successful floor panel. Initial performance issues and code issues will also be addressed in this chapter. Chapter 3 describes parametric studies performed in order to gain knowledge about structural properties of the panel. From this, geometric and material properties are recommended. Chapter 4 describes shear tests performed on different connectors in order to compare performances and give recommendations for the flange-to-web connection. Chapter 5 discusses the modeling of a full-scale panel which has been refined by adding stiffness's and strength capacities of the recommended connectors tested in the previous chapter. Chapter 6 discusses and gives recommendations for the many non-structural topics, which are very important to the success of a hollow massive timber panel. To close, the final conclusions and recommendations for a long-spanning hollow massive timber panel are reported in Chapter 7.

CHAPTER TWO

BACKGROUND

Introduction

The idea that massive timber could be a regular choice for commercial buildings is relatively new. For the past few decades, wood has commonly been viewed as unsuitable for non-residential construction. This is due to negative perceptions stemming from certain inherent properties of wood that make it seem suboptimal when compared to other materials. Some of these undesirable properties of wood include the inherent variation in natural strength, volumetric instability, low strength-to-volume ratio, durability, creep, perceived vibrational performance and combustibility. With the help of new technologies and by utilizing wood's strengths, engineers have introduced products that have addressed all of these issues.

By implementing various techniques to grade lumber and by using appropriate grade combinations to strengthen massive timber products, variation in structural elements can be lessened so that overall strength can be increased. An example of this is the use of higher grade wood in the top and bottom laminates of glulam beams while lower quality wood is used in the center. Another way massive timber reduces variations is by using significant quantities of dimensional lumber. Using more wood reduces the statistical probability of how much one defect can affect the overall performance of a structural element. Both CLT and glulams have randomly distributed knots throughout the overall element, which reduces the isolated impact of individual knots on the strength of the member. The strength of visually graded lumber is mostly affected by size of knots and knot location. Having a knot can cause a weak point in a structural member and, to make things worse, there is not enough other wood to compensate and share the load with. This causes a much lower value to be assigned to a single board than if many boards were used with randomly distributed knots, as in massive timber elements. Likewise, this also contributes to glulams having higher design values for shear.

Cross Laminated Timber is a good example of how dimensional stability can be achieved. Wood does not shrink or swell appreciably in its longitudinal wood grain direction. When laminated together, this serves to restrict perpendicular shrinkage and swelling in the boards of opposing layers. This is similar to the tendency of concrete slabs to crack because of shrinkage or expansion. In these cases cracks can be controlled with steel reinforcement acting to restrict movement.

While wood has a lower strength-to-volume ratio, it has a very high strength-to-weight ratio. Even so, more volume tends to help in some areas, especially by adding stability in the case of buckling for columns or the case of lateral torsional buckling for beams. Since technology has allowed for large pieces of wood to be pieced together, very high strengths can be achieved. These strengths are substantial enough that these systems can even be designed to support high-rise buildings. By its nature, CLT uses a lot of wood. Therefore high grades of wood are not required because of the resulting excess strength and stability in all directions. Also, this excess strength makes a CLT building redundant and can be very useful if a member ever failed when subjected to extreme events like an earthquake, explosion or fire.

Wood is susceptible to various other challenges also seen in concrete and steel. Much like concrete, wood exhibits a creep behavior. This can be controlled in bending elements by choosing to camber beams so that they offset the long-term deflections. Also, even though they are classified as non-combustible, fire still adversely affects steel, concrete, masonry, and light-gauge structures and can cause structural failure, just as it can with wood. Moreover, these other systems tend to be less redundant than a CLT building which could allow for partial collapses to occur sooner. The topic of "concealed spaces," which is described in the International Building Code, will be addressed in detail in Chapter 6. That being said, all buildings have some form of concealed spaces, and these can allow for unconstrained fire spread no matter the construction type.

The issue of fire performance is much different for massive timber than when compared to light-framed wood construction. Both burn at the surface and char, but massive timber has a much larger ratio of volume to surface area. Because of this, the char layer of the wood insulates the rest of the beam/panel and significantly slows the burning of the rest of the wood, allowing for a sizeable untouched cross section to remain and carry the structural loads. Wood chars at a very predictable rate and this allows for engineers to design for specific fire ratings just like with other materials.

As can be seen, massive timber is very different than customary wood structures that most people are familiar with. Massive timber can perform very well and meet the requirements that are required for use in commercial construction. Every part of the massive timber system from the panel elements to the connections is important for the behavior of the whole building. Therefore, a literature review was performed outlining the basic properties of wood, massive timber elements, various connections, and the building systems that employ massive timber. Examples of traditional and innovative ideas will be presented in the following sections in order to lay a foundation for the development of a successful hollow massive timber system. As with every other structural material, designers are welcoming innovations that improve the overall performance. This research project is aimed at doing the same thing for wood. Also, since HMT panels seem to have characteristics that lend themselves to long-span situations, a review of other massive timber spanning systems will be included, as well as the code-related issues faced by any wood-based spanning solution.

Wood Properties

Wood is a very complex material that requires an in-depth knowledge in order to ensure proper structural performance. Also, wood is naturally grown, making it a uniquely renewable building material. Various unique properties of wood stem from it being a natural material. These properties include that it is non-homogeneous and anisotropic, meaning that its physical properties change according to location within its volume as well as in different grain directions. Wood has the greatest strength in its longitudinal direction, which is parallel to the wood grain. This property can be explained by looking at the cell structure of wood, which is shown below in Figure 2.1.

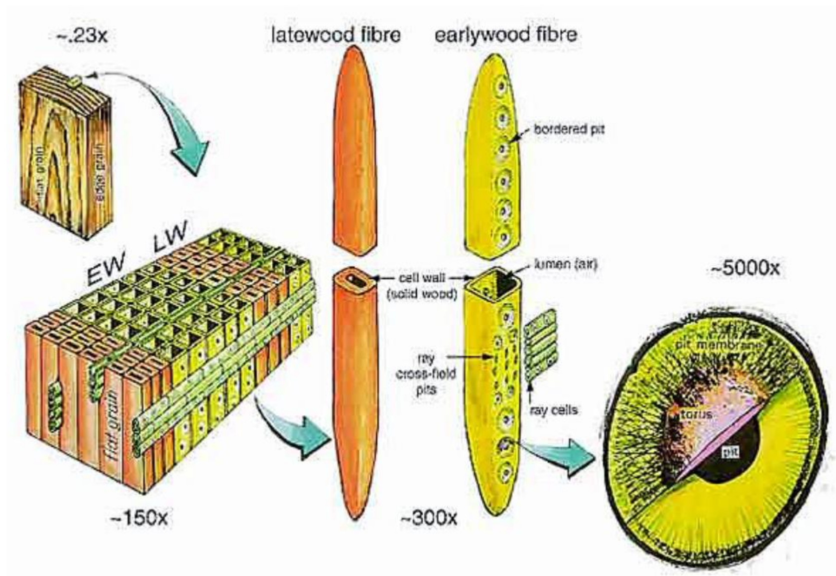

Figure 2.1: Cellular Makeup of Wood Fibers (Pang)

The tensile strength of wood is lower than the compressive strength largely because the greater effect of knots and other defects on tensile properties. Defects that can affect the strength of wood include knots, checking, slope of grain, and wane, each of which are illustrated in Figure 2.2. Because of the high variability in wood, grading and statistical analysis have been implemented in order to identify reliable strength values.

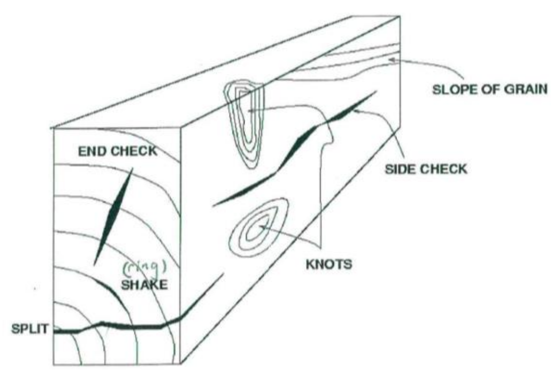

Figure 2.2: Growth Characteristics and Drying Defects (Pang)

The use of wood as a structural material is not new. Wood has been used for many centuries, and not solely for low-rise detached housing. One particular structure that showcases the strength and durability of wood is the Horyu-ji Temple in Japan, which is illustrated in Figure 2.3. It has stood at 122 feet tall for 1400 years even though it is located in a high seismic zone and a wet climate (MGB Architecture and Design et al.). Furthermore, throughout more recent history, many heavy timber structures have been built around the US and Canada.

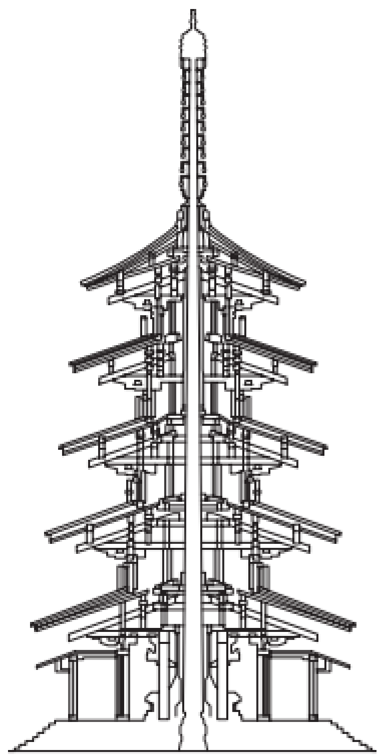

Figure 2.3: Horyu-ji Temple in Japan (MGB Architecture and Design et al.)

The strength of trees is a great example of wood's innate potential. Trees have small cross sections and are akin to a very tall cantilevering structure with sprawling branches that catch any wind. Nonetheless their strength is so great that they are still able to resist high lateral forces. Wood has been shown to be durable and strong in nature and in the built environment over the centuries, but the full potential naturally bound up in wood has not yet been fully realized because of insufficient technology.

The use of stick framing and even large solid-sawn timbers is unacceptable for accomplishing the higher heights, heavier loads, and higher performance demanded for today's larger buildings. By using modern massive timber products and engineered lumber, larger and more uniform building products can be manufactured, allowing for wood construction to meet these demands.

Advances in manufacturing and adhesives have played a large role in the development of massive timber products. Manufactures are now able to remove the defects that tend to reduce strength and add variability in strength data. Also, product sizes are no longer limited to the girth of available trees, as manufactures are able to make large beams, columns and panels from small pieces of wood glued together. Overall dimensions are now only limited by the capabilities of laminating presses and other factors such as shipping restrictions.

Massive Timber Structural Elements

Massive timber elements are large, prefabricated wood members comprised of smaller dimensional lumber or lumber that would otherwise be unsuitable for structural purposes ("Mass Timber Products"). They can be used for any part of a building including wall, floor and roof elements. Examples of massive timber products are Cross Laminated Timber (CLT), Glue Laminated Timber (Glulam), and Structural Composite Lumber (SCL). Massive timber is classified as engineered wood because the elements are engineered to have higher performances. This is usually achieved by reducing variability in the product by processing of the wood or by increasing the mass to the point that these variations do not matter.

CLT panels are made by gluing together dimensional lumber into layers of perpendicular grain orientation. Usually there are an odd number of layers with a symmetrical arrangement of grain orientations in the cross section. Because of this, CLT panels are often likened to large plywood. The outer layers of a floor panel run parallel to the span to create a strong axis for out-of-plane bending. Strength is also contributed by the weak axis, which runs perpendicular to span. This is because of the cross-ply boards running that direction. Normally, CLT floor panels are controlled by vibration/deflection because they lack enough out-of-plane stiffness compared to their strength. Because of the shear mass and crosswise orientation, these panels have extremely high in-plane shear strength and compression strength. CLT is very stiff and strong in the plane of the panel and typically can be viewed as a rigid body. Because of these properties CLT panels can be used anywhere, either for floors, roofs or walls. An example of a 5-ply CLT panel is shown in Figure 2.4.

Figure 2.4: Cross Laminated Timber Panel (TimberFirst)

CLT panels are cut using computer numerically controlled machinery (CNC) which provides a very high precision to obtain low tolerances. This allows for every panel to be prefabricated and cut to exact dimensions with any openings, holes or unique shapes cut into it during manufacture. CLT is also highly dimensionally stable because wood tends to be very stable parallel to grain and, therefore, each of the different layers work to restrain expansion or shrinkage in alternating layers. All of this makes CLT an extremely durable product with much potential for a long service life. An example of a CLT building is shown in Figure 2.5.

Figure 2.5: Structure of CLT Building (Waugh Thistleton)

CLT is also known as a sustainable building material compared to steel and concrete because of the renewability and low embodied energy of wood. As buildings become more energy efficient, the energy embodied in the materials of construction constitute a larger percentage of a building’s overall carbon footprint. In addition to the comparatively low energy required to harvest and process wood building products, wood also sequesters carbon in the form of the carbon dioxide absorbed and stored by living trees. Thus, wood can actually achieve a negative net carbon footprint. Furthermore, wood is also highly reusable and recyclable. Additionally, CLT can utilize wood that would otherwise be undesirable for building applications such as the pine beetle infested wood being utilized in Canada (Atkinson). This is wood that would otherwise be left to rot and relinquish all of its carbon sequestered during the life of the tree.

The precise dimensions of these panels and their solid construction allow for tight joints and minimal air leakage, which can play a large role in the performance of the building envelope. Also, the wood has some inherent insulating properties, which adds slightly to the overall R-value of wall and roof assemblies.

CLT panels complement the use of glulam beams and create a cohesive massive timber construction system with many possibilities. Glulams are formed from pieces of dimensional lumber that are glued together all in the same direction to build up any cross section. This is preferred because glulams can be optimized by using higher strength boards where the structural demands are the greatest. This is typically at the top and bottom a beam cross section. Also, like CLT, glulams are ideal because they use small pieces of lumber that can be obtained from trees of smaller diameter compared to solid-sawn timbers. Besides beams, glulams can be used for columns or truss elements.

Some glulam beams are now being reinforced with fiber reinforced polymers (FRP) to gain extra bending capacity. Tensile stresses in glulam beams commonly govern the design; therefore by adding reinforcement in tensile portions of the beam, strength can be gained. This is similar to the design concepts of reinforce concrete. By employing a second material of greater consistency and higher strength, the beam's overall strength performance dramatically improves. Additionally, wood of lower quality can often be substituted for expensive high-quality wood in the middle of the cross section without significantly lowering beam stiffness.

SCL is a term used to describe multiple engineered wood products. The most widely used of these is laminated veneer lumber (LVL). LVL is produced from bonding together thin sheets of wood veneer with the grain direction running parallel to span orientation. Parallel strand lumber (PSL) is similar except the wood used is from the very outer layers of the tree which are the strongest. These layers of the tree cannot be used for veneers in LVL because the tree is not perfectly round at its outer circumference. Therefore, this portion of the tree can be cut down into 2-ft to 8-ft long strands and glued together to form a PSL. Laminated strand lumber (LSL) utilizes small diameter trees by cutting the wood into small strands and then gluing them together to form beams or panels. This is much like oriented strand board (OSB) but with the strands all oriented in the parallel direction instead of randomly oriented. These products minimize the impact of defects by either eliminating them or distributing them to create a stronger product. Some added benefits are dimensional stability and a wide range of product sizes. All SCL products improve the resource efficacy of the manufacture process by using a higher percentage of wood from each tree ("Structural Composite Lumber (SCL)").

Most massive timber products and engineered wood products are relatively new if looking at the long history of building structures. The first glulam patent was issued in

1900 in Switzerland and Germany and the first glulam structure to be built in the U.S. was the USDA Forest Products Laboratory in Madison, Wisconsin ("A Profile Of The Glulam Industry"). The first SCL was developed in the 1969 by Trus Joist Corporation who also developed laminated veneer lumber. Later, parallel strand lumber was developed and then SCL was expanded with laminated strand lumber in 1990. CLT was introduced in the early 1990's in Austria and Germany and lesser-known FRP glulams were developed in Oregon in the 1990s. Thus, the twentieth century and, in particular, the 1990's mark a historic point for creative developments in wood building. Alex Rijke from dRMM Architects states that, "I personally think, in my potted history of the last three hundred years, that the eighteenth century is defined by brick, the nineteenth century is the steel frame era, and the twentieth century was concrete. In the 21st century, timber is the new concrete" (Alex de Rijke).

Massive Timber Connectors

Since the introduction of CLT, there has been an explosion in the innovation of massive timber connectors. All of these products have considered the inherent properties of wood and discovered how to supplement them with other materials like steel. Examples of different connection strategies will be outlined throughout this section. Since there are innumerable varieties of connections, the most relevant ones to this project will be addressed.

Self-tapping screws (STS) are one of the connections that became better known after the development of CLT. These screws utilize a bit at the tip of the screw that allows the screw to drill through the wood. There are two main advantages to this. First, because the screw is drilling a hole as it goes, the stresses that could cause splitting of the wood are lessened. Second, STS are normally very long screws. Without be self-tapping this length could lead to high torsional resistance preventing these screws from being driven into the wood to their full depth.

Putting self-tapping screws at an angle is a very common use and is normally referred to as inclined screws. When screws are placed at an angle, they are allowed to carry a portion of their load axially in tension or compression instead of traditionally in shear when placed at a 90^o angle to the wood surface. In contrast to shear connections, this produces a very strong and stiff connection. One disadvantage, however, is that the failure mode tends not to be ductile. That said, failures are still very predictable and consistent. A requirement of this set up is to not have a reversing load or if there is a possible reversal of load then screws must be placed opposing one another so that loads can be carried in both directions. There are many situations that meet this requirement, which can take advantage of this type of connection.

Much research has been performed on this topic outside of the United States. Blass at KIT in Germany has performed work with analytical models, experimental testing, and allowable spacing of STS. Also, research has been performed at the University of British Columbia on the performance of STS in shear and moment connections under dynamic loads.

The mechanics behind placing a screw at an angle are clearly illustrated in the publication titled *Joints with Inclined Screws* (Bejtka and Blass). Nails and screws placed at 90^o have several failure modes, but when they are placed with large embedment into both sides of the joint, they bend and bear into the wood until they act like a screw in tension and either rip out or rupture in the screw. This results in a highly pliable connection that is very useful for many situations. Also, it is interesting to point out that if pushed far enough, it may result in a stiffness increase because it begins to act like a screw in tension. As shown in Figure 2.6, an inclined screw simply skips over most of the bending and bearing segment of a 90^{o} screw failure and ends up acting in tension. Therefore, the important factor is the embedment of the screw and the strength of the screw in tension.

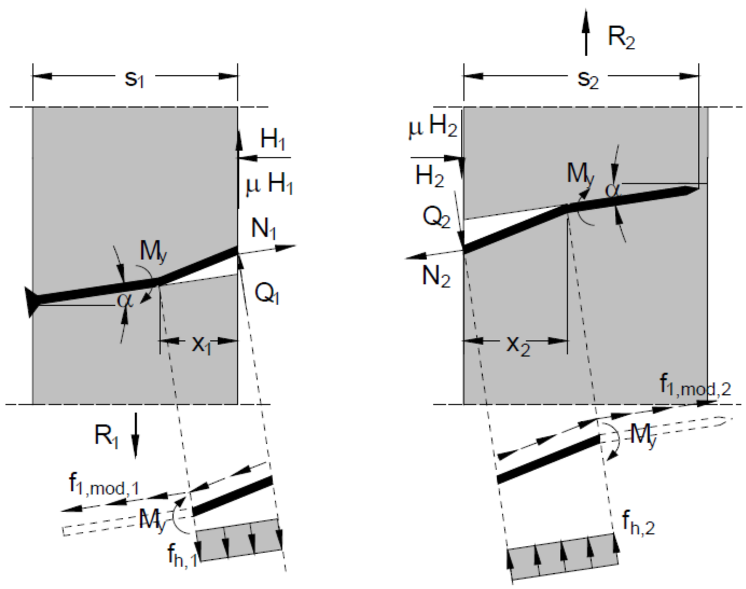

Figure 2.6: Joints with Inclined Screws (Bejtka and Blass)

Much research has been performed investigating the required embedment depth of a screw in order to obtain design values for inclined screws. It has been shown that there are several factors that affect the withdrawal resistance of a screw (Closen). These factors are:

- Length of screw penetration, l_{eff}
- Screw diameter, d
- Wood density, ρ
- Screw in angle, Θ
- Characteristic withdrawal resistance parameter, $f_{1,k}$

One of the most recent and significant design equations was accepted by a Canadian Construction Materials Centre (CCMC) evaluation report for SWG ASSY screws (*SWG ASSY® VG Plus and SWG ASSY 3.0 Self-Tapping Wood Screws*). The design equation approved for SWG ASSY inclined screws is:

$$P_{rw,\alpha} = \varphi \frac{0.8 * \delta (b * 0.84 * \rho)^2 * d * l_{ef} * 10^{-6}}{sin^2\alpha + \frac{4}{3} * cos^2\alpha} * K_D * K_{SF} \tag{2.1}$$

where:

$P_{rw,\alpha}$ = Factored Withdrawal Resistance of SWG ASSY Screw at an Angle (N)

φ = 0.9

0.8 = adjustment to standard term loading

δ = material adjustment factor: 82 for $\rho \geq 440$ kg/m^3; 85 for $\rho < 440$ kg/m^3

b = 1 for D-Fir-L, SPF, SYP, WRC, Hem-Fir or 0.75 for Parallam (PSL)

ρ = mean oven-dry relative density (kg/m^3)

0.84 = adjustment of mean oven-dry relative density to fifth percentile value

d = outside screw diameter (mm)

l_{ef} = embedment depth into member (threaded length – tip length (= d)) (mm)

α = screw angle

K_D = load duration factor = 1.0

K_{SF} = service condition factor = 1.0

Though there has been significant research performed on this topic, there has been no consensus on the stiffness of these joints. This is an issue that needs further investigation since inclined screws are sometimes used to create composite action between two wood members. Without reliable stiffness values for the screws it is hard to tell how much composite action will be produced.

Because of the structural efficiency of inclined screws, many products have incorporated them for use in diverse applications other than simple shear joints. One of these applications is using large STS as reinforcement to wood where extra strength is needed. They can be used for shear reinforcement in beams, as shown in Figure 2.7, or in panels, or they can be used to resist punching shear produced by point loads acting on panels. They can also be used to increase the bearing resistance perpendicular-to-grain. This could be useful for increasing the load-carrying-capacity of walls or columns with high loads that are resting on floor panels. Furthermore, they can be used for reinforcement in cases of grain splitting perpendicular-to-grain, which could be seen at stress concentrations where notches or holes are placed near the end of a beam or panel. This can also become a problem when connections are applied to a side of the wood that causes that side to pull away in tension from the rest of the structural element.

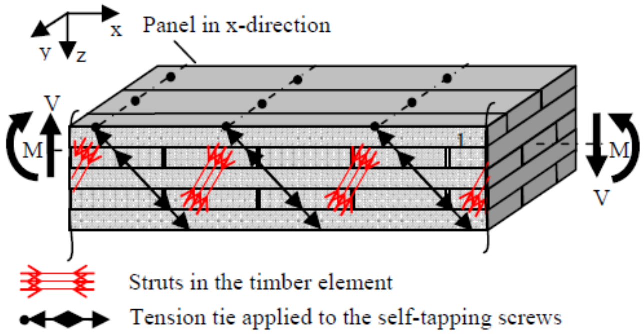

Figure 2.7: Strut and Tie Model of Panel Reinforced with Self-Tapping Screws (Mestek and Winter)

Inclined screws can also be used for gaining composite action between two members. They are a good choice for this because of how stiff the connection is and the inclined angle would result in more composite action gained than if screws were placed at 90^{o}. A curve for partial composite action typically looks like the example shown in Figure 2.8. Both strength and stiffness follow an S-shaped curve when connector stiffness vs. relative strength and stiffness is plotted in a semi-log scale. With increasing connector stiffness between two members, it can be seen that strength and stiffness of the combination of both elements approaches a certain value. This value equates to the section properties computed with full composite action. With decreasing connector stiffness, it can be seen that the combined strength and stiffness of both elements in the combination again approach a certain value. This value is computed by adding the properties of the two individual elements together and therefore represents the overall properties with no composite action. As can be seen, there is a transition area where combined stiffness and strength is very sensitive to connector stiffness. It is most desirable to be in the upper part of this curve past this transition, but at a certain point there is a diminishing return for having a stiffer connector.

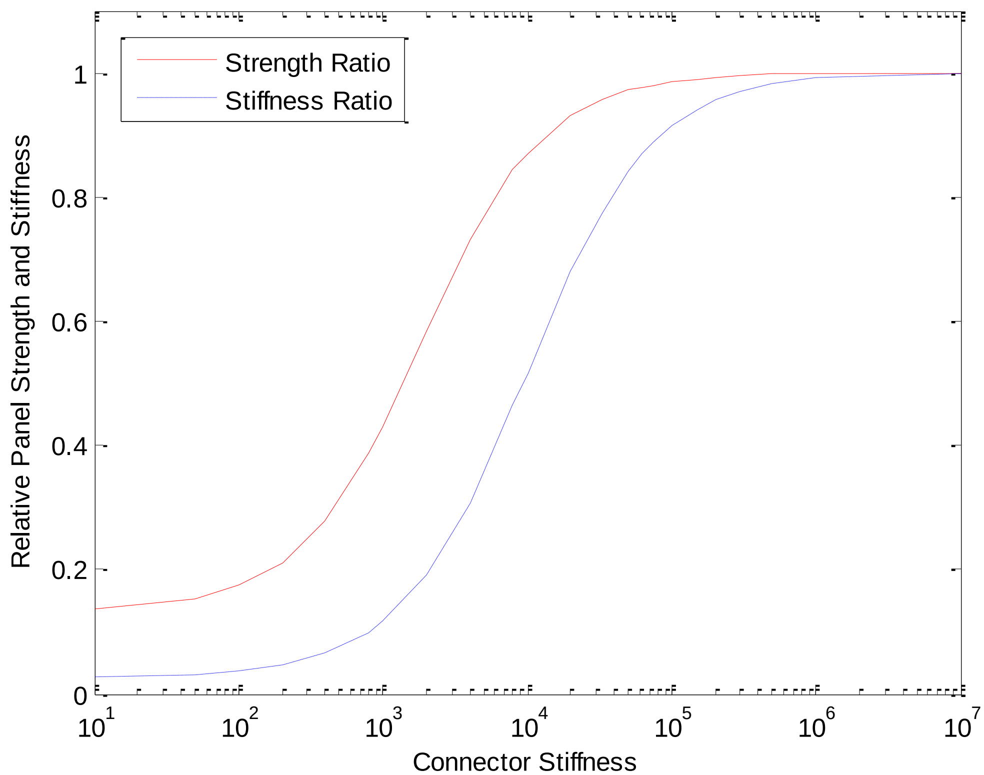

Figure 2.8: Influence of Composite Action on Built-up Beam Performance

Inclined screws have enough stiffness to gain larger percentages of the composite action of a beam, but it is relatively unknown how much is gained because the stiffnesses of these joints have not been researched as much as their strength properties.

Inclined screws have been used for composite action in many different emerging structural systems. One example from Europe is a joist floor system in which CLT is placed over wood beams spaced at approximately 2 to 3 feet on-center and then connected with screws in order to obtain a more efficient composite cross section. A larger version of this could a T-beam, in which the stiffness and strength of a glulam beam is increased by connecting it to a CLT panel above it.

Very similarly, a wood-concrete composite slab has been made that uses ASSY plus VG screws. In this system a plastic connector embedded into a precast concrete slab is used to hold the top of the screw while the bottom of the screw penetrates into the wood. This system has obtained a European Technical Approval (ETA-12/0196) and can be seen in Figure 2.9.

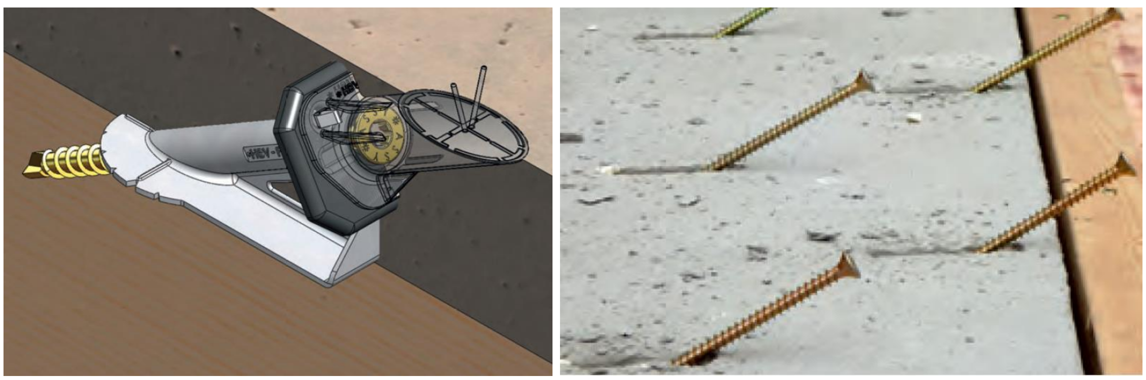

Figure 2.9: Wood-Concrete Composite Slab with Inclined Screws (Schraubenwerk Gaisbach GmbH)

Another application for screws placed at an angle is in connections at the end of members. Large screws can be placed through the end of a beam and into a supporting wall, column, or girder to transfer the beam reaction. An example of this is shown in Figure 2.10. These screws can be placed in tension, where the screws are angled down into the end of the beam, or they can have a crossing pattern.

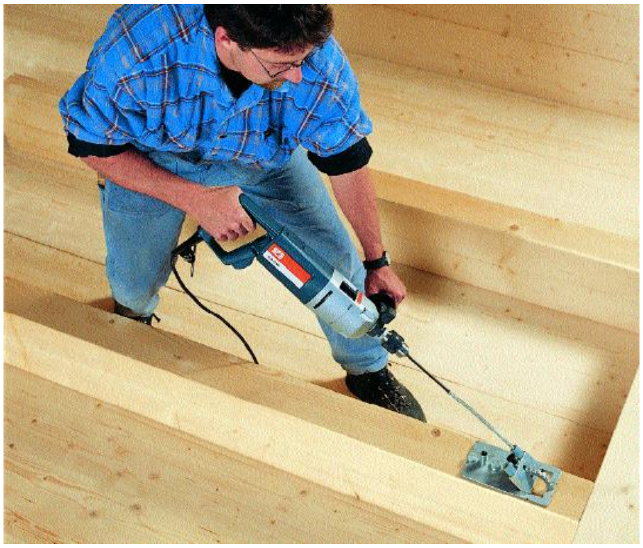

Figure 2.10: Self-Tapping Screw Used as End of Beam Connection (SFS Intec, Inc.)

While inclined angles are often beneficial, self-tapping screws are also frequently used at 90^{o} in simple connections like floor to wall panel connections and floor panel to floor panel connections.

The Sherpa connector, shown in Figure 2.11 is an innovative product that provides a step up from simple STS joints. This connector has two dove-tailed metal slots that are separately imbedded into each structural member in the connection. Instead of typical long STS being used, many smaller screws are used at various angles. They are mostly placed so that they will act in tension because of gravity loads acting downward, but some screws are placed in other directions to provide stability and the capability for the connection to resist forces in every direction. The dovetails are made so that the plate on the beam slides down into the plate on the wall or column and is locked in place as long as forces are pushing downward. To prevent the dovetail from sliding upward, a locking screw is placed at the top of the connector. Some benefits of the Sherpa connector, or similar models from competitors like Knapp, include that they are fairly simple and quick to connect in the field and that they can be routed and concealed into the wood. More importantly, because they are concealed they will not be exposed to high heats in a fire and therefore do not run the risk of losing capacity.

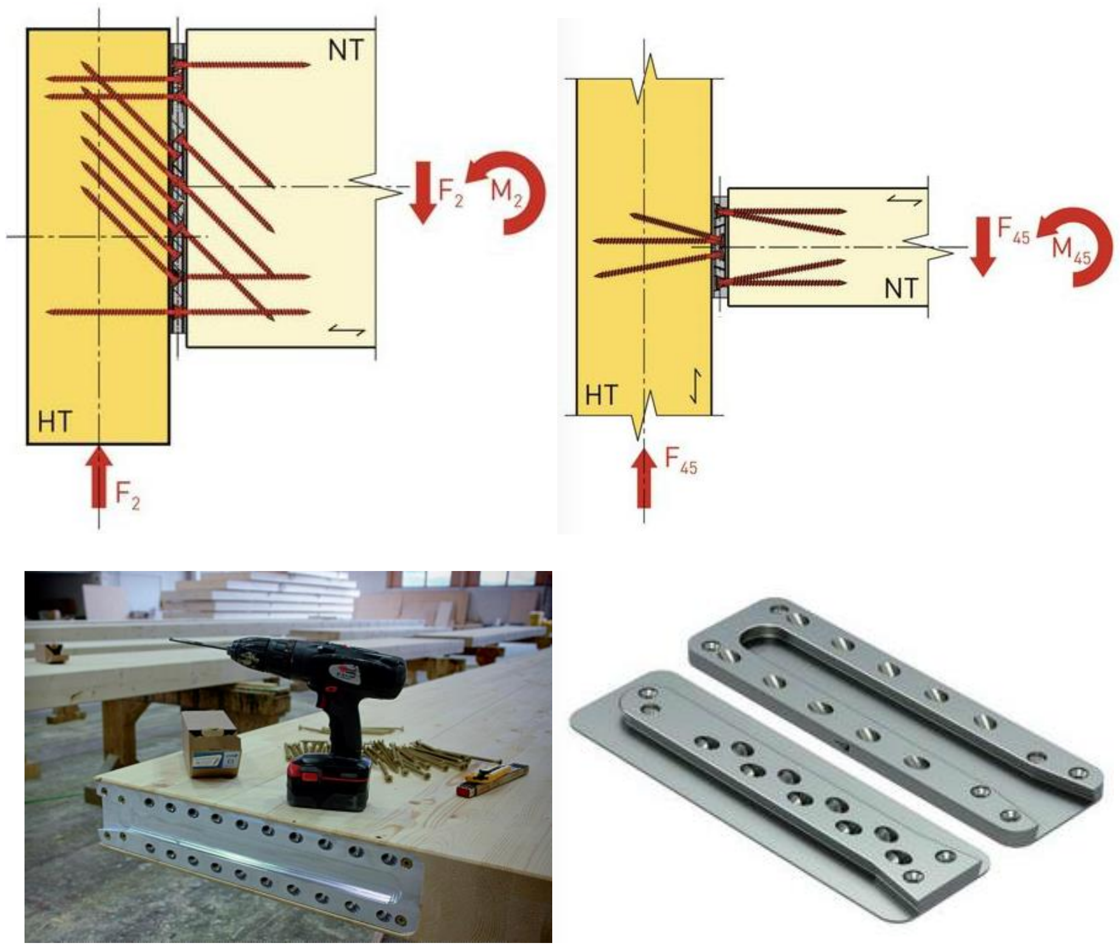

Figure 2.11: Sherpa Connector (Vinzenz Harrer GmbH)

Angle bracket connectors also use smaller screws and can resist force in all directions. These connectors are common to CLT construction, and are used to attach wall and floor panels together. Likewise, smaller screws can be used when connecting steel members to wood. Figure 2.12 shows a steel plate with screws used to attach the steel beam to the timber wall.

Figure 2.12: Steel Beam-to-Wood Connection (Wallwork)

Besides the host of mechanical connections mentioned, adhesive connections are used as well. Glue bonds can be used between two wood elements or adhesives can be used to embed steel parts. Adhesives are known to produce very strong and stiff connections. This can be seen by the fact that CLT is produced now with glued joints rather than being doweled together with nails as it was in early versions. Like welded connections are to bolted connections in steel construction, glue is to mechanical connectors in wood. Glue bonds are more of an art because glue bond strengths can vary greatly depending on many conditions including clamping pressure, temperature, adhesive mixture, moisture content, surface smoothness, surface flatness, if the surface cleanliness. For these reasons, glue bonds tend to be used in controlled environments like a factory setting.

Two examples of adhesives used to embed steel connectors into wood include glued-in rods and glued-in plates. Glued-in rods tend to be threaded and are inserted into pre-drilled holes in the CLT or other massive timber products. An adhesive is then used to fill the hole in order to make a strong and stiff connection with the wood. Because of the properties of the circular section of the threaded rod, these are mostly used when they are axially stressed. Therefore, potential applications include hold-downs at the end of shear walls and moment connections in which high tension and compression forces are produced. HBV and HSK connectors produced in Germany feature embedded steel plates. HBV, which stands for wood concrete composite, consists of expanded steel embedded into wood elements on one side with an adhesive. On the other side, it is cast into concrete and used to transfer shear along its axis in a wood-concrete composite system. HSK connectors utilize a steel plate embedded in wood-to-wood connections. The plate has holes in it to increase the adhesive bond strength much like the expanded metal. Because of combining the ductility of steel with the stiffness and strength of a structural adhesive, these connectors can be very effective.

There are many other products that have been produced for specific wood connections, but the ones described above are the most relevant to this research project. These connectors are combined with structural wood elements to produces massive timber building systems. These systems are described in the next section.

Massive Timber Structural Systems

There have not always been many options when putting together a large timber building. Historically, heavy timber buildings were limited to large-section timbers or glulams as structural members. Now there are many other options for wood structural systems because of the variety of massive timber products described above and the connectors that complement them.

With the development of CLT, in particular, there has been a change in the nature of massive timber construction. Since CLT panels are prefabricated off site, cut to exact dimensions, and maintain stable dimensions, they allow for more rapid on-site construction. For these reasons, CLT massive timber systems are akin to precast concrete construction. Remarkable construction times have been reported and more time could potentially be saved when construction crews become more familiar with this type of construction.

There are many other benefits to these massive timber systems with CLT. In addition to fast construction times, construction crews tend to be small. Four workers with one crane were used to complete the Stadthaus apartment building in London in 2008 (Crespell and Gagnon). In this case, since simple angle brackets and STS were used for connections, construction did not require highly skilled labor or heavy equipment but rather just drills for driving in the screws. Also, the utilization of wood instead of concrete or steel, meant that smaller cranes could be used. With lighter equipment, less complexity and fewer people on the worksite, safety is also dramatically increased. Another benefit of massive timber construction is that it fits well within urban environments. Not as much space is needed on the site because of the smaller cranes, and the surrounding populations are less disturbed because the construction process is quicker and quieter.

Another large benefit of massive timber systems is their sustainability. The carbon footprint of a building is reduced in two ways by using wood. The first is in the construction of the building and the second is by helping to reduce energy used over the life of the building. Wood takes less energy than steel or concrete to harvest and process into a finished construction material. Also, trees act like a sponge taking in and storing carbon dioxide. This carbon sequestration can actually result in a negative net carbon footprint. Besides being more energy efficient in its processing, wood is the only building material that is a truly renewable resource, and as more trees are grown more carbon is taken from the atmosphere. The importance of the carbon footprint in building construction is becoming more and more emphasized.

Additionally, long-term energy efficiency is promoted by massive timber in various ways. Because of the precision of CNC machines, the resulting wood elements fit together tightly. This allows for minimal air leakage, greatly improving the efficiency of heating and cooling systems. Also, wood is a natural insulator, with an R-value of $1^1/_4$ per inch of thickness. This is not much compared to materials made for insulation, but it is considerably more than other structural materials and benefits the overall R-value for exterior wall assemblies. Additionally, just like concrete, wood can act as a thermal mass in a building. If designed to do so, the massive amounts of wood in a building can absorb extra heat during the day and release it slowly at night when heating is desired. Thus, a thermal lag effect can be produced, allowing for a building's temperature to be better regulated and for less energy to be used for heating and cooling.

Another benefit of a massive timber system is its structural performance and resiliency. Wood has a very high strength-to-weight ratio, which results in a strong, stiff and lightweight building. This has been proven by full-scale shake table tests in Tsukuba, Japan (Quenneville and Morris). With the correct connections, massive timber buildings can withstand some of the most severe earthquakes.

As a result of the lighter construction material, there is an approximate $^{1}/_{3}$ reduction in building weight when compared to concrete. Therefore, the required foundations can be much smaller. This can equate to considerable savings in a large building. Additionally, seismic loads are reduced in proportion to the decrease in weight of the building.

Today's massive timber has greater structural capabilities and opens up possibilities in building design and construction. With the increase of strength in FRP-reinforced glulams, longer spans with shallower beams are possible. Additionally, glulams can form any shape or configuration that can support complex structural geometries. CLT is flexibile because of its strength in all directions. It can cantilever easily and can be used double cantilever situations as well. Panels can be used as flat plates structurally by creating a moment connection between panels and allowing the resulting plate to act as a two-way slab resting only on columns. Long, solid walls essentially act as deep beams. This can allow for long open spans on the floors above or below. Likewise, walls can act as transfer girders or to support longer cantilevers projecting from them building. Thus, massive timber offers tremendous flexibility in design, from traditional forms to striking new architecture.

Another important measure is the quality of the product itself. Because of CNC machines and prefabrication, the quality of construction is very high and the resulting structures are much more durable than traditional stick frame construction. Also, the cross lamination of CLT restricts swelling, shrinkage and warp and creates a high dimensional stability.

Increased fire performance is what makes the expansion of wood construction possible. The mass and volume of massive timber elements are its natural defenses against fire. As seen with large logs burning in a fire, it takes time to burn through wood of considerable mass. This is because of a charring effect that insulates the wood and protects it from rapid fire consumption. For example, a 5-ply CLT wall with one layer of type X gypsum on each side was tested and achieved close to a 3-hour rating (Rizo). These and other tests have demonstrated that massive timber elements can perform very effectively in a fire. In these cases, connections and other related details need to be carefully designed for adequate fire performance as well.

Massive timber is increasingly viable for commercial construction, and it is often the case that wood and other structural materials are being mixed. Steel and concrete are being used in combination with wood to optimize performance where needed in a massive timber building. Conversely wood is sometimes incorporated to enhance performance in otherwise non-wood systems. Hybrid steel systems result from structural steel frames used in combination with CLT floor slabs and CLT shear walls. This eliminates the poured concrete floors in traditional steel-framed buildings, saving time and weight. Another example of a hybrid system results from replacing glulam beams with steel beams in a massive timber building in order to reach longer spans with shallower beams and reasonable floor-to-floor heights. In the case of concrete, it is sometimes used as a composite with wood beams or panels.

Advances in massive timber have opened the gate to larger and taller wood buildings. At least two studies have looked into the possibility of timber skyscrapers and the type of massive timber structural system that is needed to make this viable. The first is called “Tall Wood” and was written by Andrew Waugh. In this study, massive timber is used as the predominant load-resisting system. For lateral forces, a massive timber core is used with steel beams to create a strong column weak beam effect. The beams link massive timber shear walls together and are allowed to yield and dissipate energy in a seismic event. The risk of a weak story collapsing from the racking of shear walls is eliminated because the massive timber panels are so strong. The second study is called “Timber Tower Research Project” and was written by Skidmore, Owings & Merrill. The unique design of this system utilizes reinforced concrete beams linking all the wood elements together (Skidmore, Owings and Merrill, LLP).

Development of Long-Span Massive Timber System

The research presented in this thesis is not the first to examine a long-span solution for a massive timber system. There have been other proposed designs and tests. Some originated from research and others have been designed by practicing engineers. All are at different stages of development with some having been researched extensively, some already in use in buildings, and others still just on the drawing boards. That said, it has not been easy to design a successful all-wood long-span system, and this provided the motivation for the HMT panels proposed in this document. In this section, the design requirements and desired properties of a long-span solution will be discussed and many of the previously-proposed designs will be described.

As with all structural elements, the design for long-span massive timber includes strength limit states and serviceability limit states. The strength limit states for a floor panel typically include bending and shear checks. Strength can be augmented with either larger or more efficient geometries, or by a higher strength material. Usually, a long-span floor system is limited by serviceability, and this is certainly the case for wood. Serviceability limit states normally include a maximum deflection limit for a total load case, a live load case, and a long-term load case. But, it has also been shown that typical serviceability limit states with a maximum deflection limit might not be enough to design for vibrations in a floor system. This is not just the case for wood, but rather vibration design is becoming a larger issue with most structural materials, including steel and concrete. This is generally due to the increasing design efficiency of these systems, which, while able to limit deflections, do not provide enough out-of-plane stiffness to minimize vibrations. Long-span systems, in particular, are prone to have problems with vibrations. This is true for CLT floor panels, which are controlled by vibration in most cases. Therefore, vibrations should be regarded not only as a design consideration, but as a limiting design factor.

In general, there are three properties that affect the dynamic performance of any structural element. These are 1) stiffness 2) mass and 3) damping. The desire to have large open spaces in buildings, has led to long-span floor systems and fewer partition walls in buildings. This greatly reduces the damping effects in floors so that more stringent requirements have to be met in order to control vibrations. These measures involve adjusting the stiffness and mass of a system. Generally, the mass is established by the type of floor system and its materials. Therefore, stiffness is left for adjustment by the designer in order to achieve acceptable vibrational performance. Equation 2 combines stiffness and mass to find the natural frequency

$$f_n = \frac{1}{2\pi}\sqrt{\frac{k}{m}} \tag{2.2}$$

Human disturbance from vibration is a very complicated topic, but it is commonly held that humans are sensitive to vibrations between four and eight Hz. Therefore, it is ideal to have a natural frequency above or below this range. Typically, stiffness is increased in order to gain a higher natural frequency above this range. An increase in stiffness requires a larger moment of inertia (MOI) or a higher modulus of elasticity of the material (MOE).

To obtain a larger MOI, one might use a deeper, more efficient section or a less efficient section with more material. For an effective and efficient long span panel, these considerations should be optimized so that the floor is not too deep, which adds floor-to-floor height in a building, or too inefficient, which drives up cost. Though designers often desire 30-foot minimum span lengths, a long-span solution with even more design flexibility and greater span range is desirable.

There are many other requirements when considering the success of general floor systems. Considering solid CLT floors at a benchmark, there are many areas for improvement yet there are many other advantages that would be great to maintain.

One important area for improved performance is acoustics. Acoustical performance in a structure are placed into two categories. These categories are airborne and structure-borne impact sound. The airborne sound insulation of a wall or floor system is measured by transmission loss, in other words, how much sound is depleted through the wall or floor assembly. The greater the transmission loss, then the less sound is transferred through the system. Airborne transmission loss is quantified into a single rating system called Sound Transmission Class (STC) so that the sound insulation properties of building element can be compared. Likewise, Impact Insulation Class (IIC) is used for impact sounds. Acoustical performance is very important for meeting occupant expectations and building code regulations. Unlike wall system, floor panels have to be checked for both an adequate STC and IIC (Hu and Adams). Solid CLT floor systems often require additional layers and materials to satisfy STC and IIC regulations. Drop ceilings or raised floors with acoustical insulation are common measures.

Another key aspect of CLT with room for improvement is its cost-competitiveness. Massive timber buildings can be cost-competitive, depending on building size and geometry, however, a more economical floor system is needed. In a study at the Vienna University of Technology, Wolfgang Winter states that the main challenge for cost-effectiveness in CLT buildings is the floor system, which tends to suffer from limited span lengths (Winter et al.). This study shows that efficient, floor systems have the potential to significantly reduce costs in a massive timber building.

Additional improvement to CLT might involve better coordination and integration with mechanical, electrical, and plumbing (MEP) systems. Currently, these systems are either exposed on the surface of walls or ceilings or concealed in raised floors, drop ceilings, or superficial chases. Ideally, there would be integrated spaces to run MEP without these additional measures.

There are various advantages of CLT construction that should be retained by alternative massive timber systems. Fire performance, in particular, is of utmost importance because CLT has set a high standard which needs to be maintained in order to gain acceptance for commercial buildings. The speed of construction offered by CLT and massive timber, in general, is critical to its appeal and its economic viability. Another advantage to maintain is low building weight. The addition of concrete in a hybrid system would add weight, undermine the advantage of smaller gravity and earthquake loads, and, in turn, affect the sizing of framing members and foundations. Tactics for economical transportation should also be maintained. Maximizing the number of floor panels that can be placed on a truck will help to reduce costs and lower carbon footprint. Finally, CLT is an inherently sustainable product and compromises on this front would disadvantage any alternative system.

All designs for long-span massive timber systems fall under one of two classifications. They are either a concrete and wood composite system or an all-wood system. When concrete is used, it is usually introduced in to gain higher stiffness and strength. It can also contribute to improved acoustical performance in floor systems. Additionally, concrete gives a good, flat finished surface.

One system that has been used in many built structures is the HBV-System mentioned previously. This system uses expanded metal embedded into the wood on one side with adhesive and cast into concrete on the other side in order to transfer shear and create a composite system. This system is advertised to span up to 15 meters (49.2 ft), but considerably deep sections are required to achieved these lengths. This system was utilized with LSL panels in the Earth Sciences building at the University of British Columbia (UBC) and is shown in Figure 2.13. Design spans for the composite panels in this building were only 21 ft, meaning that relatively thin panels could be used (Equilibrium Consulting Inc.).

Figure 2.13: HBV Shear Connectors Used in UBC Earth Sciences Building (Foit)

Another example of concrete wood composite system is the Life Cycle Tower in Dornbirn, Austria by CREE. Short, wide glulam beams were used at a relatively close spacing and then linked together at the end by a concrete beam. This system, illustrated in Figure 2.14, can span up to 9.45 meters (31 feet) (Rhomberg Group).

Figure 2.14: Life Cycle Tower Floor Panels (Rhomberg Group)

Theoretically, a thickened slab of CLT could serve as an all-wood solution for a long-span floor system. A 9-ply, 12.375” thick solid CLT panel with double parallel layers on top and bottom and the V3 wood combination from the PRG-320 (ANSI/APA) can span up to 26 feet. However, this exceeds the economical span range for CLT and shows that CLT panels spanning over 30 feet would need to be even larger and more expensive.

Thinking about the geometry of an ideal cross section provides a direction for improvement over thick, monolithic CLT panels. It is understood that the outer-most layers in a CLT panel are the most effective for resisting out-of-plane bending because they are the furthest away from the natural axis. By contrast, the wood near the center of the panel has minimal effect in resisting out-of-plane bending but is critical for resisting shear. Therefore, subtractions can be made from the center of a thick CLT panel as long as enough material is left to still perform adequately in shear. This would result in a more efficient cross section.

Thus, a CLT floor system with supporting ribs is a good alternative, and is sometimes used in Europe. This approach is supported by Winter's research, in which he suggests CLT with ribs for improved cost-competitiveness (Winter et al.). CLT with glulam ribs creates a more effective cross section and allows for shallow CLT panels to be used. One issue faced by this system is the transmission of shear between the two elements and how to ensure close to full composite action. Typically, this is achieved with an adhesive or inclined screws. Both of these options are reasonable to construct and offer sufficient stiffness and near full composite action.

The City Academy project in the United Kingdom used CLT as its main structural element. Throughout the design of this building, engineers at Ramboll came across several matters that required innovative solutions. One of these was the need for a longer span. The solution explored was a hollow CLT box element. This was chosen because of its more efficient cross section. The element consisted of two 3-ply CLT flanges glued to glulam web members at a consistent spacing. With respect to fire safety, the design philosophy held that if a fire burned through the bottom flange, then the remaining section of ribbed floor would be sufficient to handle reduced loads. In the end, Ramboll expressed reservation over taking this approach again because of the increased shipping costs of large box elements that took up too much space on a truck (G. White).

Another attempt to study hollow CLT panels was included in research conducted at the University of British Columbia under Dr. Frank Lam (Chen). This research, titled "Bending Performance of Box Based Cross Laminated Timber Systems," mainly examined the performance of hollow CLT with different panel configurations. The panel configurations are shown below in Figure 2.15.

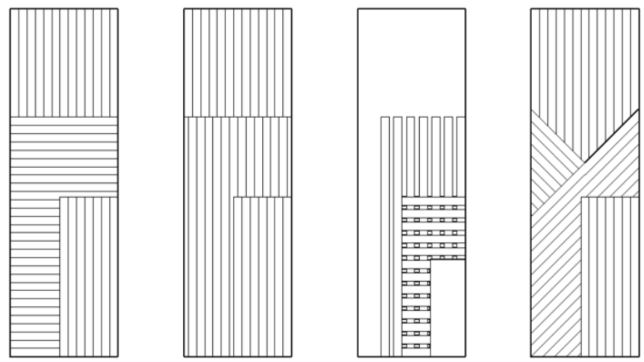

Figure 2.15: Flange Configurations for UBC Hollow Box Research

Since screws were placed at 90^{o} in every test specimen to connect the flange and web elements together, there was insufficient stiffness in the joints and this resulted in a panel with compromised composite action. Failures in web members before failures in the flanges indicated that there was sufficient slippage at the joints to cause the elements to tend to act individually, and this resulted in higher stresses in the web member. This failure is illustrated in Figure 2.16.

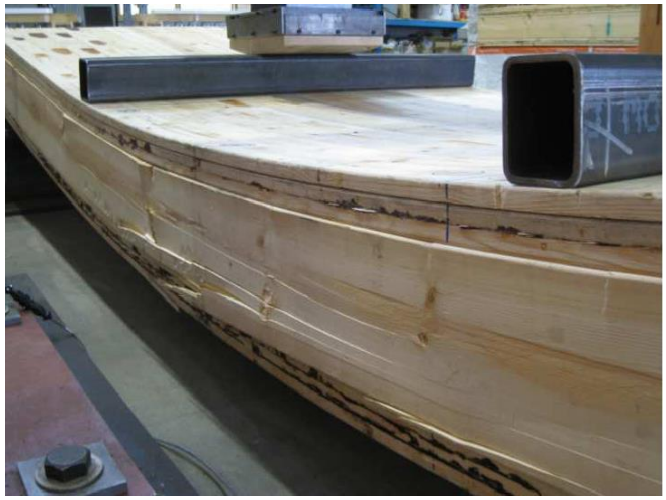

Figure 2.16: Web Bending Stress Failure Caused by Lack of Composite Action (Chen)

The research team at Clemson University has pursued an all-wood solution for a long-spanning system with the hope of retaining the advantages of CLT construction described above. That said, this research sets out to improve upon some of the limitations of CLT and other proposed long-span solutions. These areas for improvement include the efficiency of material use, vibrational properties, acoustical properties, cost-competitiveness, and potential MEP integration.

Code Related Issues

In this newly evolving field of study, it is understandable that code-related issues arise. This section will discuss new code provisions as well as existing code documents that are relevant to this research. Furthermore, it will discuss topics not considered in or in conflict with the 2012 International Building Code ("2012 IBC"). The three main code-related topics that will be discussed are fire design, inclined screw design, and composite action modeling.

In 2012, the Engineered Wood Association (APA) approved an American National Standards Institute (ANSI) standard for CLT called the PRG-320 (ANSI/APA). This document was used as a guide for this project in many ways. CLT is not currently recognized by the 2012 IBC but is slated to be included in the forthcoming 2015 edition. Even in its eventual treatment there will likely still be code-related hurdles and omissions to contend with. Therefore, many other sources including the CLT Handbook, which it's specific chapters are referenced throughout this thesis, were used for this research. Additionally, the National Design Specification for Wood Construction (NDS) was used as a reference regarding general wood design and wood connections (AF&PA).

One question that arises when talking about HMT panels is whether they can effectively be treated as CLT with respect to relevant building codes. CLT is defined in the PRG-320 as "a prefabricated solid engineered wood panel made of at least three orthogonally bonded layers of solid-sawn lumber or structural composite lumber (SCL) that are laminated by gluing of longitudinal and transverse layers with structural adhesives to form a solid rectangular-shaped, straight, and plane timber intended for roof, floor and wall applications" (ANSI/APA). Thus, a hollow massive timber panel might utilize CLT components, but it is distinct from CLT itself, which is defined as "solid." Therefore, it HMT panels will be dealt with as a separate and unique massive timber product.

CLT will be allowed for Type III, IV and V construction in the proposed 2015 IBC. Usually, it is most favorable to use CLT in Type IV heavy timber construction in order to decrease the hour ratings for fire design and allow for greater building heights and areas. That said, Type IV construction also comes with a stipulation that there be no concealed spaces in the structure. This prohibition stems from two principle concerns. First, is the danger of fire in concealed spaces spreading rapidly and undetected throughout the building. Second, is the concern that fires in concealed spaces would be challenging to locate and extinguish. This presents a challenge for HMT panels whose voids, might be considered concealed spaces and therefore disallowed in Type IV heavy timber construction. During discussions with Dr. Robert White of the Forest Products Laboratory (FPL), it was suggested that void areas in the panels be filled completely with a non-combustible material. In this case one could argue that the voids are no longer spaces and, therefore, not concealed spaces per the code definition. Additionally, it was suggested that Type III construction might be a better fit for HMT panels and maintain similar height and area regulations. Because of the expansive fire performance of massive timber along with increased interest in tall wood structures, there will likely need to be additional changes in the allowable heights and other building codes in order to keep up with demand and performance capacity. Since Type IV heavy timber is based on historical rather than engineered fire performance, it will not be emphasized as heavily. Instead, engineered fire performance will be the focus throughout the research presented here. Therefore, a specific hour rating will be considered in conjunction with charring of the bottom flange. The possibility of fire within the void spaces will continue to be considered as well (R. White).

Inclined screws will be used in this research to create composite action in a panel. Inclined screws are a fairly new concept and are not covered in the NDS. There is no major distinction between nails and screws found in the NDS when calculating the reference lateral design value. Both use the yield limit equations, which do not consider a screw in tension at an angle. Also, if considering the screw in withdrawal, then there is not an applicable equation to adjust for the reduced withdrawal value when not perpendicular to the grain. Equations recently accepted by a Canadian Construction Materials Centre (CCMC) evaluation report for SWG ASSY screws (*SWG ASSY® VG Plus and SWG ASSY 3.0 Self-Tapping Wood Screws*) were based on limit state design and are not as conservative as other simplified methods of obtaining a design resistance for inclined screws. It is expected that U.S. code will move more in this direction and, therefore, the equation from this evaluation report will be used predominately, even though it is not accepted under the 2012 IBC.

Another complexity not addressed in the 2012 IBC is how to deal with general composite action. For example, composite and partially composite steel beams are designed prescriptively through the Steel Construction Manual ("Specification for Structural Steel Buildings"), but the 2012 IBC or any of the standards that it references do not address partially composite action of any other materials. This is unlike what is found in Eurocode 5 EN1995-1-1, Annex B (Comite Europeen de Normalisation), which addresses composite action in wood with the γ-method. As described earlier and shown in Figure 2.8, the stiffness and strength can vary greatly depending on the stiffness of the connection between the structural elements. Therefore, neither an analytical model nor approximate solution was used in this research, but rather a full modeling approach was implemented. This model was set so that stiffness of connectors could be included and their effect could be considered.

CHAPTER THREE

PARAMETRIC STUDY: OPTIMIZATION OF PANEL PERFORMANCE

Introduction

In order to learn about how different properties affect the performance of a hollow massive timber panel, both analytical and computer models were used. An analytical model was first investigated using MATLAB due to simplicity. With these calculations it was assumed that 1) there was full-composite action and 2) the web and cross layer members did not have significant contribution to the stiffness and strength of out-of-plane bending. These assumptions produced trivial error, but this was deemed acceptable for the comparison of different properties. A computer model was then created using SAP2000 and used to evaluate the effects of different parameters on panel performance while including 1) the shear deformations of web, flange cross layers, flange outer layers and flange to web connections, and 2) the bending stiffness of the web. Only two dimensions were considered for all calculations and models. Although this floor panel might have strength in the perpendicular to span direction and should be able to have some load sharing capabilities, it is thought that this floor panel will primarily be used as a one-way system and therefore it was modeled just as a beam.

The preliminary section that was chosen for the cross section of a HMT panel consisted of a top and bottom 3-ply flange. Along with this, dimensional lumber placed on their edge was used for the web members. In regards to the flanges, it was decided that an unbalanced layup would be used where a double layer of Grade #2 Southern Pine lumber was used for the outside layer and an inner crosswise layer of Southern Pine

Grade #3 lumber was used. This particular flange configuration was chosen because of two reasons. The first reason is because it produces a higher moment of inertia by placing the parallel to span boards farther away from the neutral axis and therefore this results in a more efficient cross section. The second reason is because a greater fire rating can be achieved in comparison to a normal CLT panel configuration that has a crosswise layer in the middle of the panel. In contrast to this, an unbalanced orientation places the crosswise board on top of the bottom flange, which is in a position that is protected by two other layers of wood. If a normal CLT configuration was used, the crosswise layer would loses its strength earlier because of being less protected by other wood. This is important to consider because this layer is what holds the whole panel together. This would result in the rest of the bottom flange to not be adequately attached and therefore unable to help resist the load applied. The consequence of this would be a much smaller fire rating for a normal CLT panel layup; therefore, an unbalanced layup was chosen to be used throughout this research (R. White).

Design Equations and Assumptions

Standard wood design equations used for floor bending members were used throughout this research. As always in design, there are strength and serviceability requirements. Therefore, design equations for bending and shear strength were used along with applicable deflection limit equations and also a vibration limit equation. These will be discussed more thoroughly in the following paragraphs.

Bending strength was obtained by using an adjusted tensile stress value multiplied by the section modulus. The adjusted tensile stress value was obtained by using the NDS reference design value multiplied by appropriate adjustment factors. Being conservative, the tensile reference stress, f_t, was chosen over a bending reference stress, f_b, because all of the boards in the bottom flange of the hollow panel are in tension, whereas in a typical sawn lumber beam some of the cross section is in tension and some is in compression from bending. It might be possible to use the bending reference stress because this value is used when designing glulam beams and CLT solid panels, even though the outer laminates may only by resisting either tensile of compressive stresses.

For shear strength, the NDS uses an equation assuming the cross section is rectangular. The hollow panel that was investigated clearly does not have this shape. Therefore, shear strength was obtained using Equation 3.1 presented below. This Equation is founded on the mechanics of shear flow and assumes full-composite action of the elements. Since a HMT panel will act partially composite, this Equation will not accurately predict the shear strength, but is conservative when checking the two most critical shear planes. The first of these horizontal shear planes is located at the neutral axis. The second horizontal plane is located at the web-flange interface, because there needs to be a connection to transfer shear flow between the elements. Shear strength was not investigated in combination with bending stiffness and strength in the parametric studies, but rather investigated independently as an important secondary issue.

$$\tau = \frac{VQ}{It} \tag{3.1}$$

Comparison of predicted deflections to deflection limits were used to insure serviceability of the structure. Live load deflection limits along with a long-term deflection limit were checked. These limits were taken as $\Delta_{LL} \leq L/360$ and $\Delta_{Long} \leq L/240$. The long-term deflection was calculated by multiplying sustained load deflection by a 1.5 factor as suggested in the CLT Handbook in Section 2.2 of Chapter 6 (Pirvu, Douglas, and Yeh). The sustained loads were considered to be the dead load plus 30 percent of the live load. The total load deflection limit was not checked because it would have not controlled over the long-term deflection.

Vibration was considered in design calculations by using the equations published in the CLT Handbook for vibration design. The vibrational properties for a HMT panel are unknown, but these equations were chosen due to close relations between this HMT floor panel and a solid CLT floor panel (Hu and Chui). Ultimately, vibrational testing will have to be conducted on full-scale panels. Testing could include more detailed computer modeling, a vibrational analysis using accelerometers, and a human study to see what design lengths meet occupant vibrational expectations. For this research, the vibrational design limit was typically converted into an equivalent live load deflection in order to make it easy for comparison. The live load deflection limit is given by Equation 3.2 for a uniformly distributed load.

$$Live\ Load\ Deflection\ Limit = \frac{EI_{app}*384}{5*w_{LL}*L^3} \tag{3.2}$$

Load and Resistance Factored Design (LRFD), which takes into consideration the probability of the loading on a structural member and the variation in resistance, was used throughout this research. The wet service factor, temperature factor, incising factor and repetitive member factor were all taken as unity. The top flange was assumed to be of sufficient width and thickness to provide lateral stability to all parts of the cross section in compression. Therefore, no reduction was taken on the panels strength do to stability issues. All other factors were adjusted according to NDS. The load combination of 1.2D + 1.6L controlled the design for out-of-plane bending of a floor panel.

It was desired to reduce the number of variables so that a fewer number of combinations of variables needed to be investigated. Therefore, most of the variables had set values that were used while one variable was being varied. The magnitudes of the set values were chosen based on what were the expected values in the actual panel. Throughout this research some of these set values were changed as more was learned about HMT panel and what was the best value to use in the calculations.

The loads on the panel are a variable in the design, but in order to condense some of the variables, two representative loads were chosen so that all analysis could be compared. These total loadings were made up of a 40-psf service dead load and a 50- or 100-psf service live load added to a 15-psf partition load. Typical dead loads that might be seen in a building with this type system were calculated and then rounded up to 40 psf. The live load was predominantly set to 50 psf but a 100 psf load case was investigated to see what spans could be obtained for assembly occupancy area. The HMT panel might be used for live loads higher than 100 psf, but typically the spans would be shorter.

The species of lumber that was considered in this research was Southern Pine. There are many beneficial aspects of using Southern Pine with one being that it is an abundant local resource in the Southeast region. Other than Southern Pine’s abundance, there are many material properties of it that can be advantageous. Southern Pine is generally a stronger, stiffer and denser wood than many other softwood species used for structural purposes. These material properties can help in many areas including shear strength, bending stiffness and connector strength. Shear strength is increased which allows for either smaller webs to be used or webs to be used at a wider spacing. The rolling shear resistance of Southern Pine is also increased since it is based off of shear parallel-to-grain values. Since serviceability typically controls the design of HMT panels, the use of Southern Pine lumber can produce a more efficient design. Since the strength of the HMT panel doesn't control the design, the reduction in the reference design values for Southern Pine in 2013 may not impact the design. The high density of Southern Pine could also be a benefit, because connector design strengths are closely dependent on the density of the wood. Besides the lumber species, the size and normal grade of lumber that will be considered in the calculations are Grade #2 2x6 for the web and flange laminations parallel to span, and Grade #3 2x6 for the crosswise board layer. Using these two lumber grades produces a more economical member. The adjusted reference tensile design values of Southern Pine Grade #2 2x6 lumber is 1036.8 psi and for Grade #3 was 604.8 psi.

Also, all calculations and models were set to be 30-foot simple span panels, which were analyzed in as a single beam, which included only one web of a panel and the flange area tributary to one web. Properties of the panel were analytically investigated so that many of the variables could be set for the final design and further experimental testing can take place by considering only a few remaining parameters. An overarching objective for this research was to reduce the governing of serviceability requirements over the strength requirements. Also, another objective of this research was to understand the improved structural efficiency of HMT over a solid CLT panel.

MATLAB Analytical Model

A preliminary calculation was done in order to see the potential strength gain of using a hollow massive timber panel. With this study, the web was included and overall area of the cross section was held constant. This was done by plotting a ratio of the void height to the total height of the HMT panel and finding the ratios of stiffness and strength gained in comparison to an equivalent solid panel.

As can be seen from Figure 3.1, it is possible to gain up to 13.15 times the stiffness and 3.51 times the strength. It can be seen that there is a large difference in stiffness to strength gain. The ratio of stiffness gain to strength gain is about 3.75 times greater. This shows that greater stiffness than strength can be gained from making a HMT panel and this can be of great benefit because stiffness is what controls the final design.

In the range of 0.8 – 0.85 for the ratio of void height to total height, the maximum potential is reached. It is important to understand that a larger ratio produces a deeper panel with a thinner flange because the total area is held constant. After the maximum stiffness and strength gain is reached, the predicted gains go down because of making too thin of a flange.

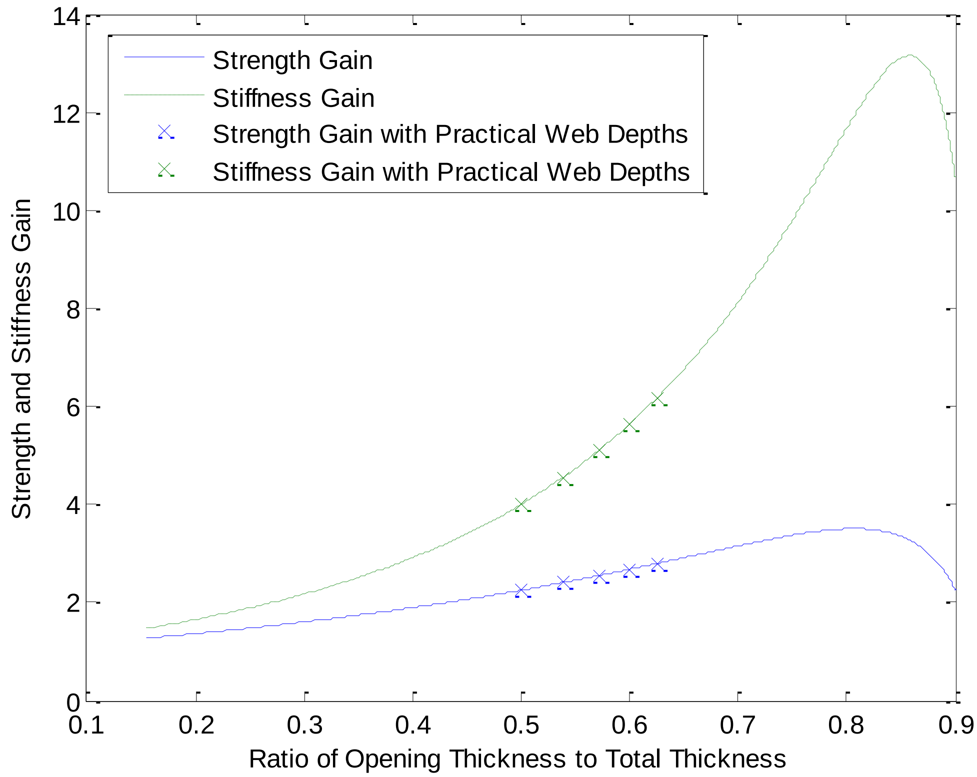

Figure 3.1: Potential Stiffness and Strength Gain

Another aspect that can be observed from Figure 3.1 is that at practical ranges of the ratio of void height to total height, much lower strength and stiffness gains can be achieved. These practical ranges were shown because in terms of constructability a flange can only be so thin and a void can only be so tall. For example, the thinnest flange that would be practical to make is by having two layer oriented perpendicular to each other so that one layer would provide stiffness to the panel and the other layer would hold the panel together. Therefore, this limits the flange to being at least 2.75 inches in thickness if planed 2x lumber is used. Also, an architect would not desire to have a floor system that is overly deep, because this adds to the height and cost of the building. Fire performance is also another important consideration, and a thicker flange produces a higher fire rating. Therefore, to insure a 1-hour rating a three-ply flange with one cross lamination and two outer laminations parallel to span should be used.

Description of MATLAB Parametric Study

Using MATLAB, an analytical model was used to produce graphs where one variable was changing while all others were held constant. This was done in order to clearly see the effect that different variables have on overall performance. Each of the design equations were plotted based on the height of the panel required. Therefore, the higher the line means that a deeper panel is required. Also, the highest line at any point tells what the required height is and which design equation is controlling because the deepest panel represents the strongest panel required from all the equations. Sometimes balancing these equations would be too complex or not make much of a difference. Not many designers or owners would care if they get a little extra stiffness that results in less sag in a floor because strength limits the design. Likewise not many would care if a floor system has a little more strength than needed because a serviceability limit controlled. The design of massive timber floor panels is controlled by vibration rather than either strength or deflection. There can be a large difference between a panel designed for strength and one designed for deflection or vibration serviceability. A higher stiffness is desired since stiffness closely relates to vibration and deflection and this would help improve the vibrational performance. Therefore if these design equation were balanced so that vibration was less controlling, a more efficient panel could be designed.

For these calculations, two assumptions were made: 1) a fully composite section and 2) the web and cross layers did not contribute to any stiffness. These assumptions are illustrated in Figure 3.2 along with variables that describe the cross section. Width was not a variable that needed to be considered because everything was being analyzed in 2D and therefore the width was a constant value of 12 inches. Also, the design was produced for a 30-foot simple span beam. The thickness variable describes just the thickness of the parallel to span layers while ignoring the negligible effects of the cross layers. The normal value for thickness was 2.75 inches - two layers of planed 2x6 lumber. Depth was approximated as the distance from the middle of the wood parallel to the span in the top flange to the middle of the wood parallel to span in the bottom flange. The normal value for depth was 11 inches which was set so that a 2x6 web member along with planed 2x6 cross layers could be used. MOE and MOR values were set to the properties of Grade #2 Southern Pine 2x6. All other variables were consistent with what was listed in "Design Equations and Assumptions" section of this Chapter.

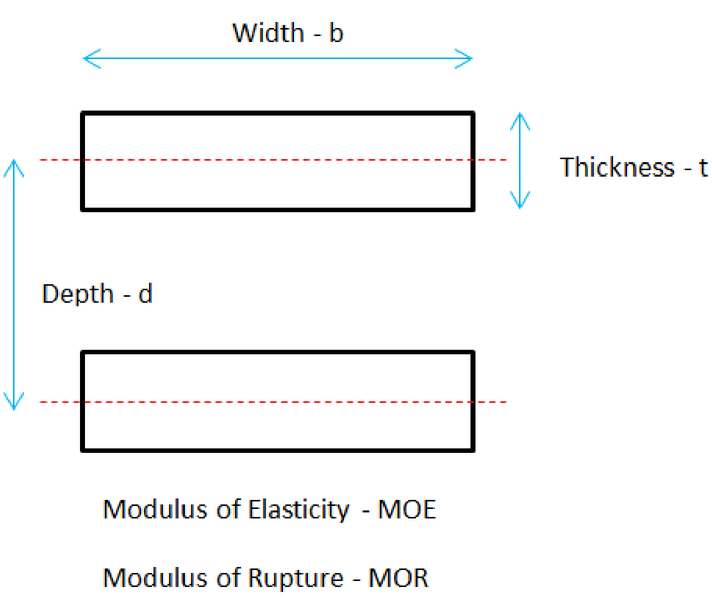

Figure 3.2: Approximate Cross Section of a Hollow Massive Timber Panel

MATLAB Results and Discussion

Graphs made changing multiple variables are shown and discussed in this section. Also, qualities that are most desirable and the practical ways of obtaining these values are discussed. In the end, specific combinations of lumber grade and cross section will be graphed to show possibilities that can be used in creating a HMT panel and what are the advantages and disadvantages of each.

It can be observed in Figure 3.3 that the thicker the parallel-to-span layers are, the less depth is required for a panel. A deeper beam is usually more economical but if it is desired to limit the depth of a beam, then thicker flanges are needed. More importantly, it can be seen that the gap between the design depth required by strength and vibration is getting smaller when the thickness of the flange is reduced. At around 1.08 inches in thickness, strength and vibration equally control, while strength had already surpassed what the deflection limits required. This is an important finding that could allow a more efficient cross section to be made which would optimize the performance of massive timber floor panels and potentially reduce the cost.

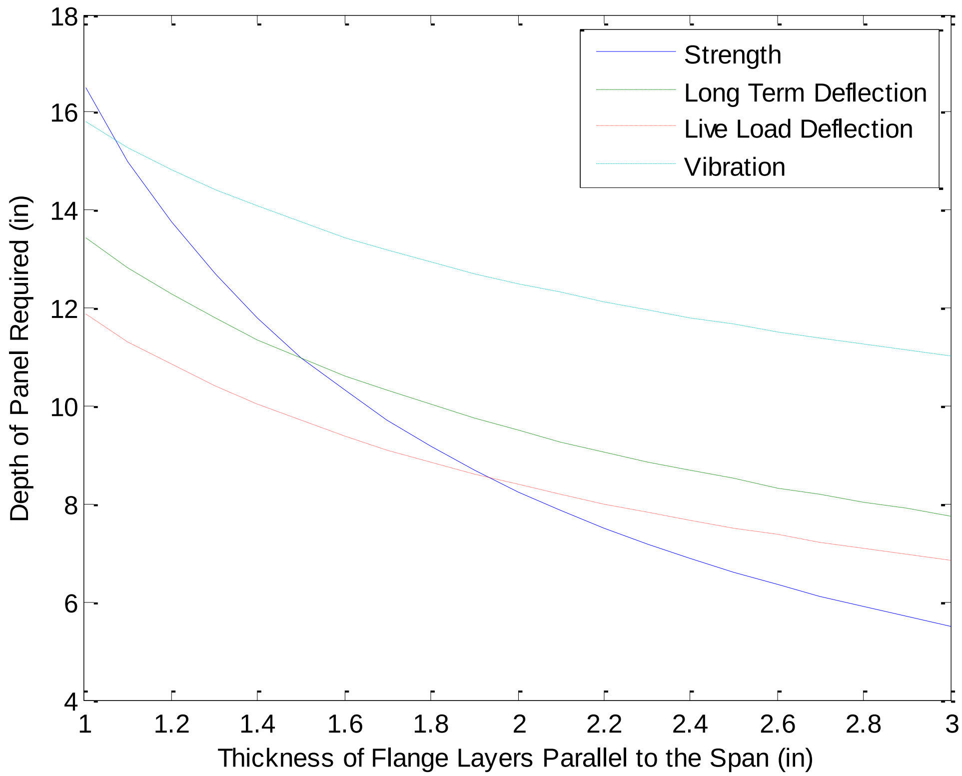

Figure 3.3: Depth of Panel Required vs. Thickness of Flange Layers Parallel to the Span

It can be observed in Figure 3.4 that with the increase of total load, the strength of a panel controls more. The required depth for strength surpasses deflection limits and the vibrational requirement, which was not changed at all by a change in total load. This can be logically confirmed because the magnitude of the design load does not affect the vibration design. Therefore, as the design load increases strength design is more critical. Since the loading of the panel is not in the control of the designer, but rather given, the load cannot be changed in order to get a more efficient cross section. Nevertheless, this information could be used by a designer to correctly respond to different loadings in order to make a more efficient design.

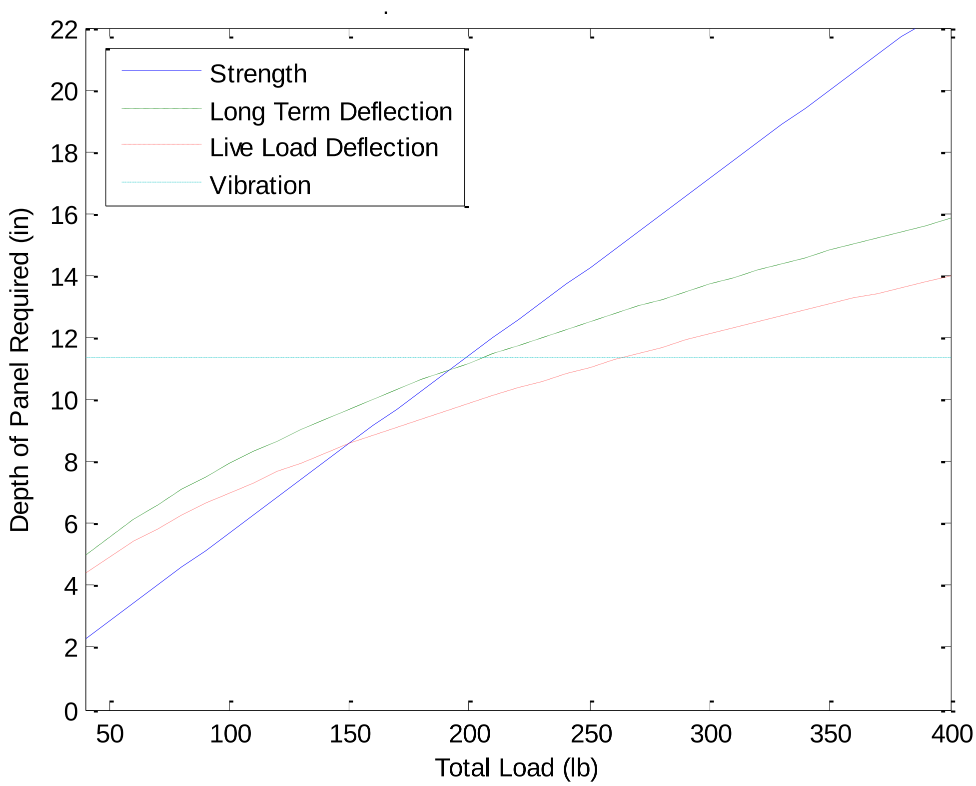

Figure 3.4: Depth of Panel Required vs. Total Load

It can be observed in Figure 3.5 that an increase in the ratio of live load to total load will close the gap between strength and vibration a small amount. This is because live load has a higher load factor than dead load and therefore produces a higher ultimate load. Also, it can be seen that the depths required to meet the live load deflection and long- term deflection limits coincide at one point on the graph. This is because as the ratio of live load to total load increases, there is less sustained loads which decrease the long-term deflection and more live load which increases live load deflections.

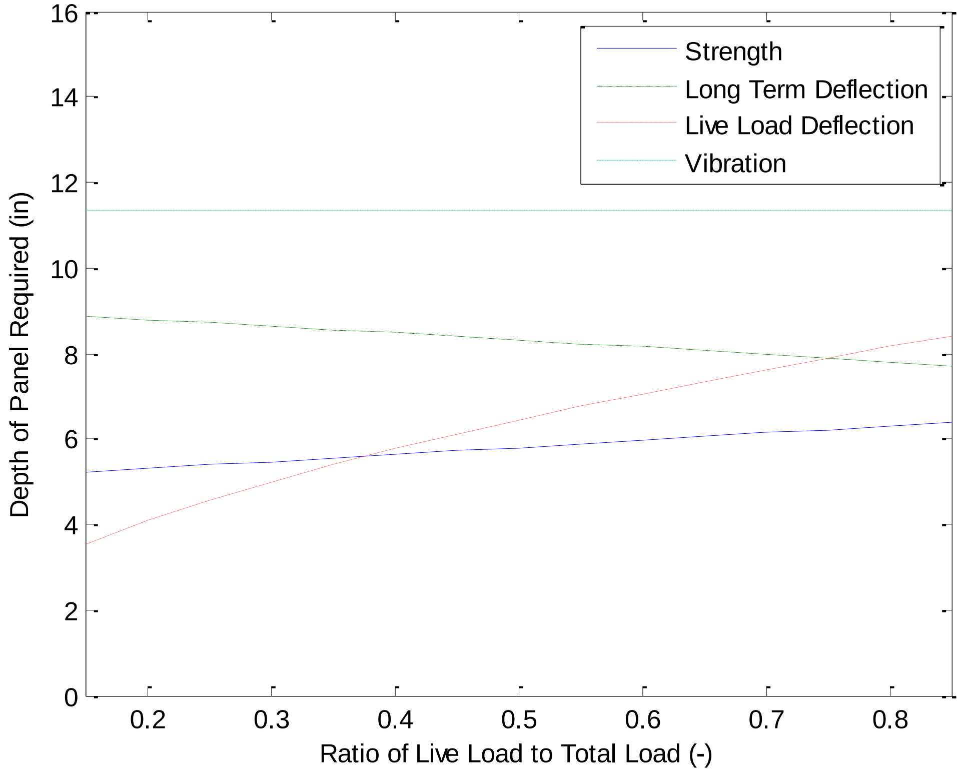

Figure 3.5: Depth of Panel Required vs. Ratio of Live Load to Total Load

Figure 3.6 shows a graph in which the live load changes while the dead load remains constant. This is a more practical graph than those shown in Figures 3.4 and 3.5, because most likely the dead load will be a more consistent value, but the live load could have a wider range of values because it is based on occupancy. Similar to Figures 3.4 and 3.5, this graph shows that because of the increase in load and the higher live load to total load ratio, there will be a smaller gap between vibration and strength design. Since strength is not controlling the design of the panel, an increase in the load does not change the overall design with respect to bending.

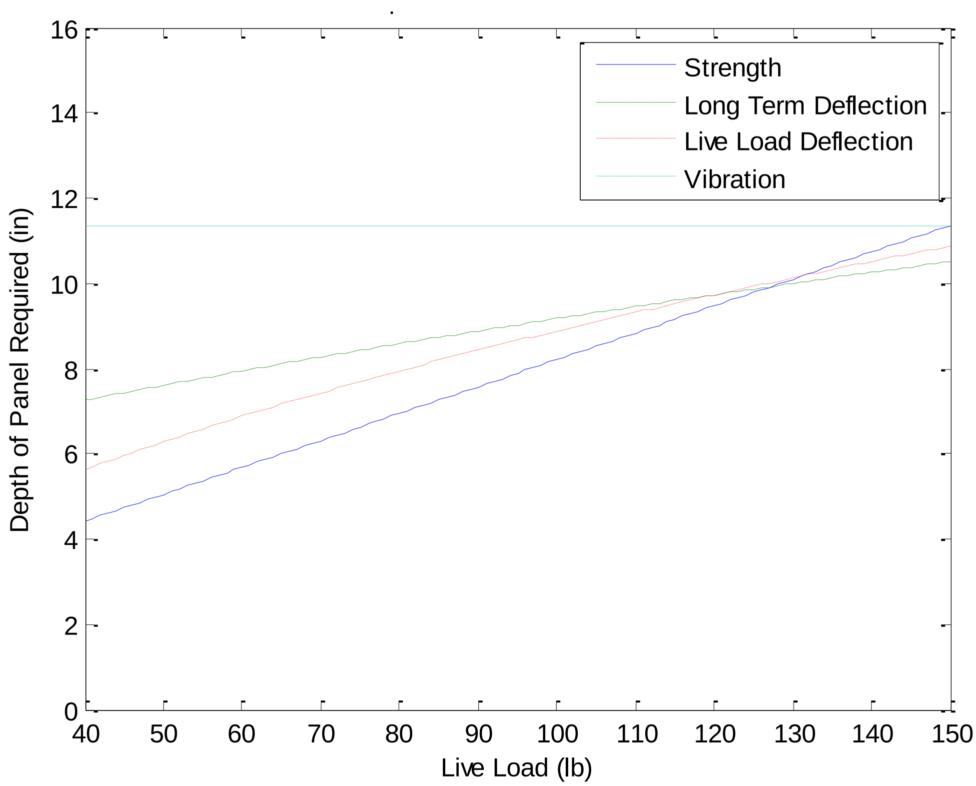

Figure 3.6: Depth of Panel Required vs. Live Load

The graph shown in Figure 3.7 changes the MOE of the lumber which can correspond to a change in grade or species of lumber. This graph does not change the Reference Design Stress that also corresponds to the different grades of the lumber, but rather keeps it constant. What was observed was that only stiffness properties changed which agrees with the fact that MOE only affects stiffness values. Although there are not significant changes in various design depths, a higher MOE had an overall positive effect on the design of the panel because the change in MOE had an effect on vibration design that was the controlling design requirement.

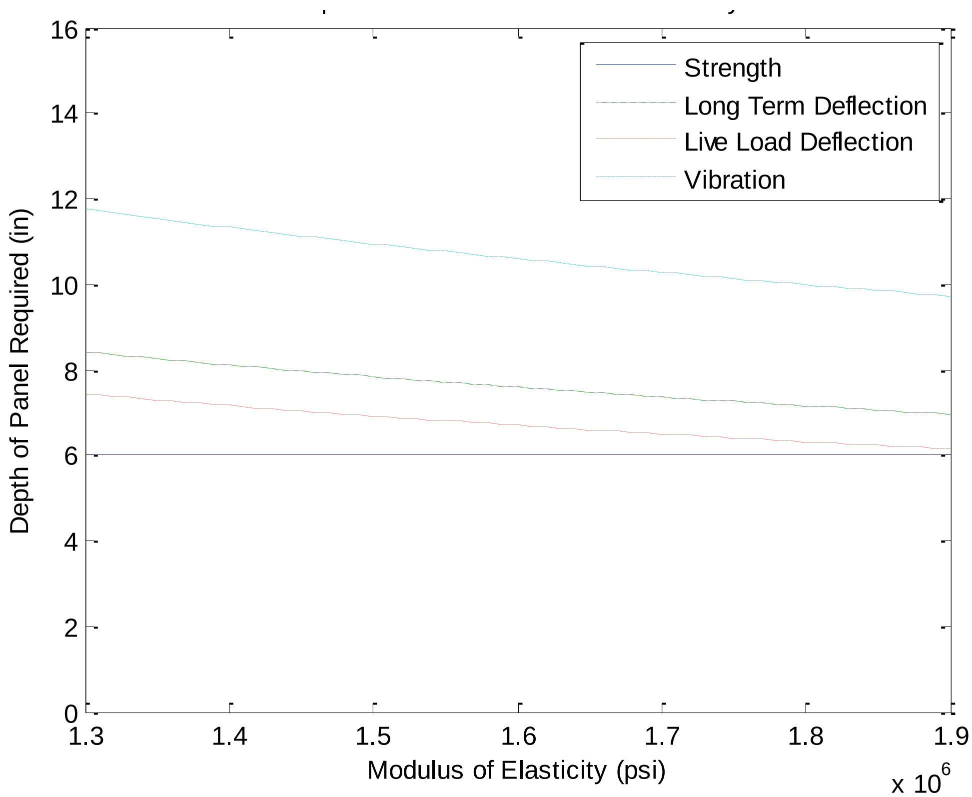

Figure 3.7: Depth of Panel Required vs. Modulus of Elasticity

Seen in Figure 3.8, this graph changes the reference design stress of the lumber, which can correspond to changing the grade of the lumber. This graph does not change the MOE of the lumber, but rather keeps it constant. The reference design stress corresponds to LRFD resistance tensile stress allowed in the lower flange element. When changing the reference design stress that corresponds to the grade of the lumber, it can be seen that a lower the grade of lumber reduces the difference between strength and vibration designs. This change is because the strength and stiffness properties of lumber do not change at the same rate through the range of grades for lumber. When comparing Figures 3.7 to 3.8, this difference in how much strength and stiffness properties change with grade can be seen. From this, it can be concluded that in general a lower grade should be investigated for this massive timber panel, instead of higher grades. This reduction of strength does not affect the overall design of the panel since strength is not a limiting design equation. If a lower grade lumber could be used, this could reduce the cost of the panel, but as can be seen there is not an overall positive effect on panel performance because there is not a reduction of depth.

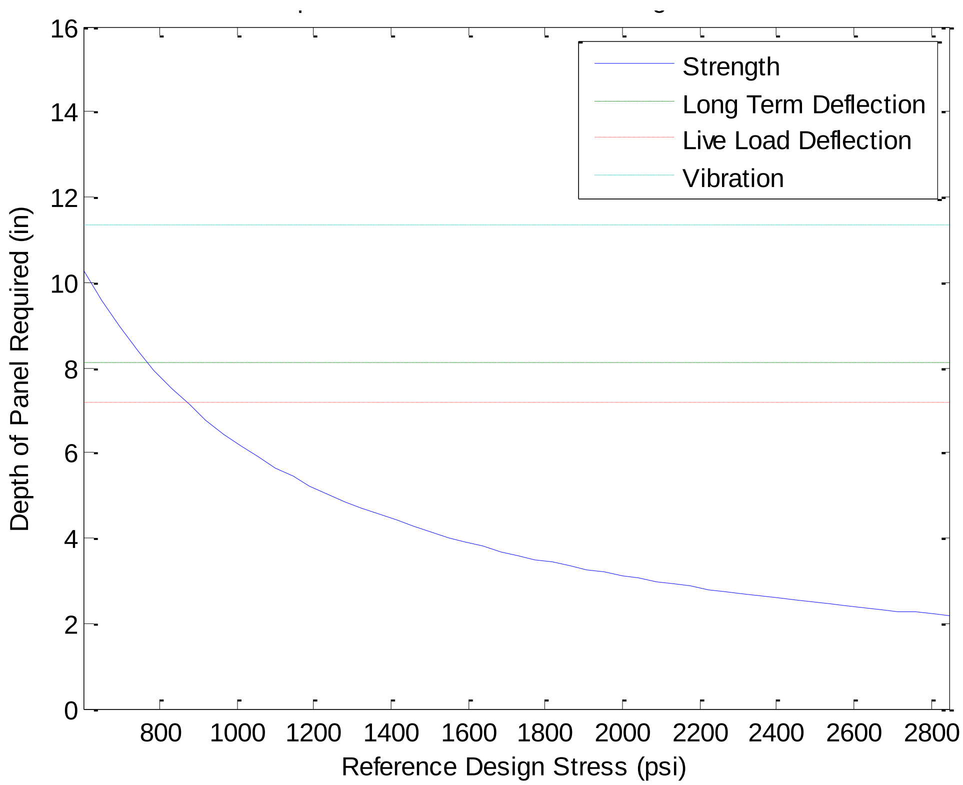

Figure 3.8: Depth of Panel Required vs. Reference Design Stress

In Figure 3.9, the graphs from Figure 3.7 and 3.8 are in theory combined. The assumptions for these graphed lines were that the strength and stiffness properties of the lumber varied linearly from the lowest grade to the highest grade. The markers show the real location of actual lumber grades and show that they closely do follow this approximation. This graph clearly shows that a lower grade should be investigated in order to allow for the difference between strength and vibration design to be reduced.

Now, four graphs are shown that represent four different possible configurations of cross section and lumber grade for a HMT panel. These combinations will be formed by a matrix that mixes grade of the lumber and the thickness of the flange. It was shown in the graphs above that the lower the grade, the better the balance of equations can be obtain up to a certain point. Since it is not desired to use a higher grade lumber, the two grades that are possible to use are #3 and #2 Southern Pine. Also, since it was observed that the thinner the flange the better the balance of strength and vibrational equations, only one-layer and two-layer systems will be used where those layers are placed parallel to the span and one crosswise layer is used. Other thicknesses could be used but this is within practical limits because these thicknesses can be made with common dimensional lumber available. These graphs show the impact of span length.

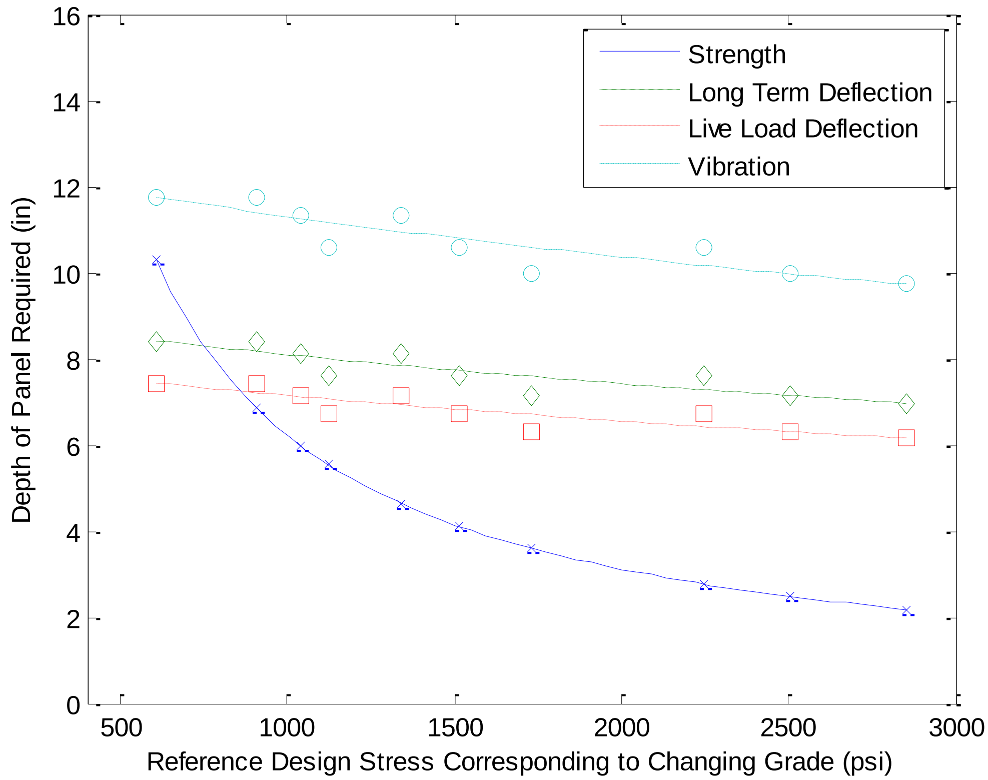

Figure 3.9: Depth of Panel Required vs. Reference Design Stress Corresponding to Changing Grade

From Figure 3.10, it can be seen that the depth required for strength design increases at a different rate as the deflection limits. From this it could be concluded that longer lengths help strength to get closer to controlling the design, which in turn helps move the strength design closer to the vibration design. This trend can be seen in all four graphs that display the four different possible panel configurations. Along with this, it can be seen that the strength requirement tracks very closely to the vibration requirement. This is an advantage of this configuration because this means that the equations are closely balanced and this optimizes the design. Also, it is beneficial that there is a slight excess of strength because vibration still controls. Even though there are efficiencies dealing with how much material is used in the configuration, there are also some disadvantages. The first disadvantage is that a deeper section is required that requires the building to be taller which can be costly. The second disadvantage is that when a thinner flange is used, there is less wood to be used to resist fire due to charring. Together these advantages and disadvantages have to be considered when trying to make a successful floor system.

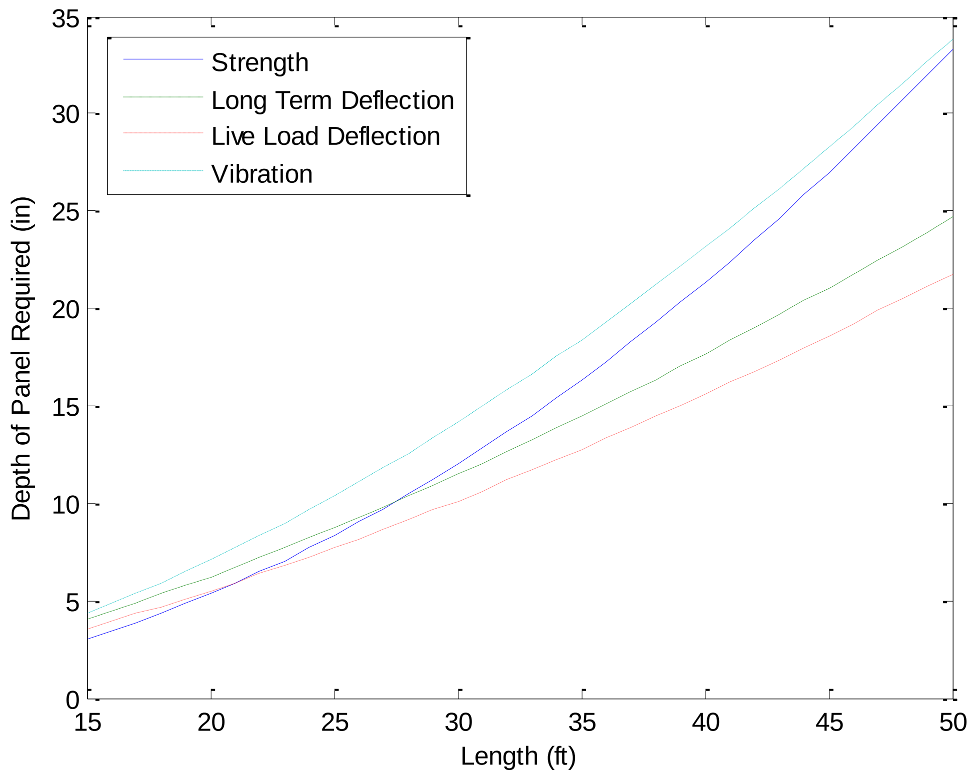

Figure 3.10: Floor Panel with a Flange Consisting of One Layer of Grade #2 Southern Pine Lumber Parallel to the Span

From Figure 3.11, it can be seen that strength controls the design of this configuration by a large amount and this is not desired. Therefore, it can be concluded that a thin flange of lower graded lumber, which both increase the amount that strength controls, produces a configuration that has gone past the balance point of strength and vibration. This is not desired and will be ruled out for the final cross section for the HMT panel.

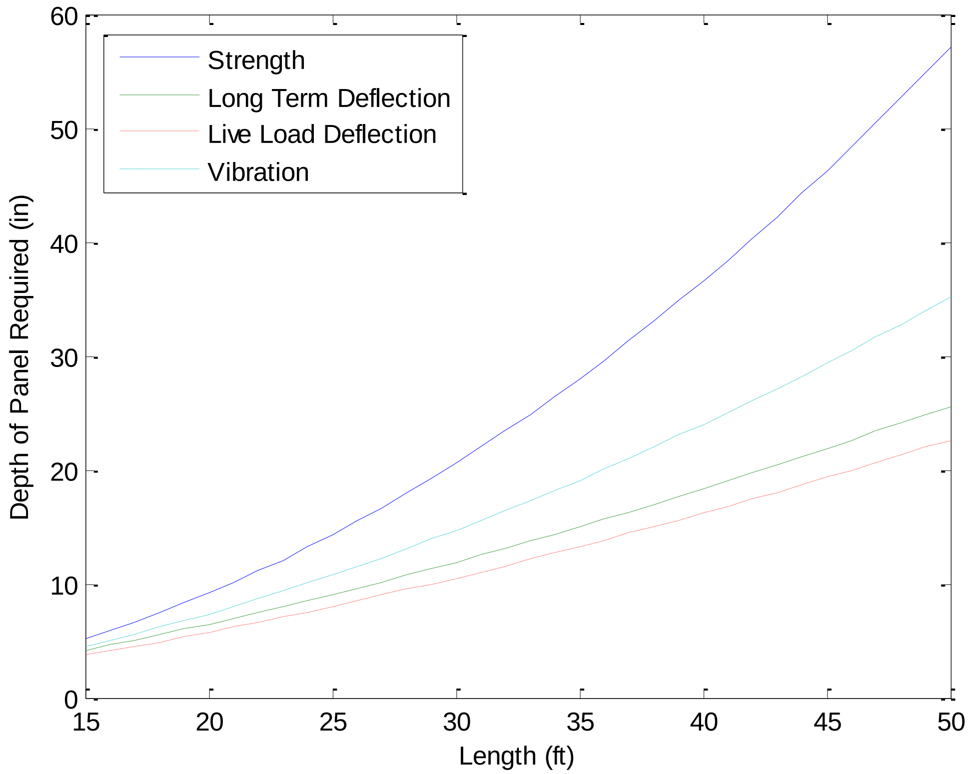

Figure 3.11: Floor Panel with a Flange Consisting of One Layer of Grade #3 Southern Pine Lumber Parallel to the Span

From Figure 3.12, it can be seen that vibration closely relates to the deflection limit equations but is still far below controlling when compared to the vibration limit equation. If the vibration limit equation changed because a HMT panels vibrational property improved, then this could be a very efficient configuration, because the necessary design depth because of vibration would decrease and place it very near to all of the other equations. The largest advantage of this configuration is the thickness of the flange. This is because with two parallel to flange layers and one cross-wise layer, makes up a thick enough section that is able to char for a predicted amount of over an hour while still resisting necessary loads and allowing for the fire to not spread into the hollow portion of the panel. The advantages of this design warrant further consideration.

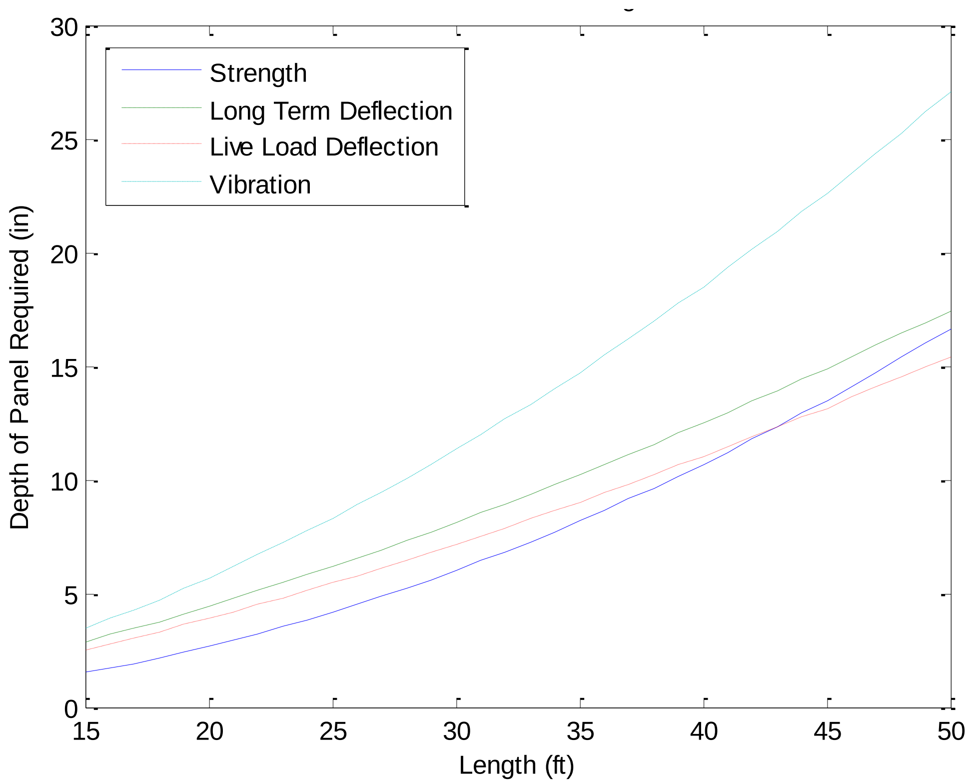

Figure 3.12: Floor Panel with a Flange Consisting of Two Layers of Grade #2 Southern Pine Lumber Parallel to the Span

From Figure 3.13, it can be seen that the strength requirement tracks very closely to the vibration requirement much like in Figure 3.10 that makes it similar to the floor panel with flange consisting of one layer of Grade #2 Southern Pine lumber parallel to the span. This configuration has the advantage of closely balanced equations that optimizes the design along with the added comfort of a slight excess of strength because vibration will control. The other benefit is that it has the extra flange thickness increases its fire performance, but this is not quite the same as the floor panel with two layers and Grade #2 Southern Pine lumber because, that panel has higher quality lumber that would have excess strength to resist the design load longer. This is small advantage that this panel doesn't have.

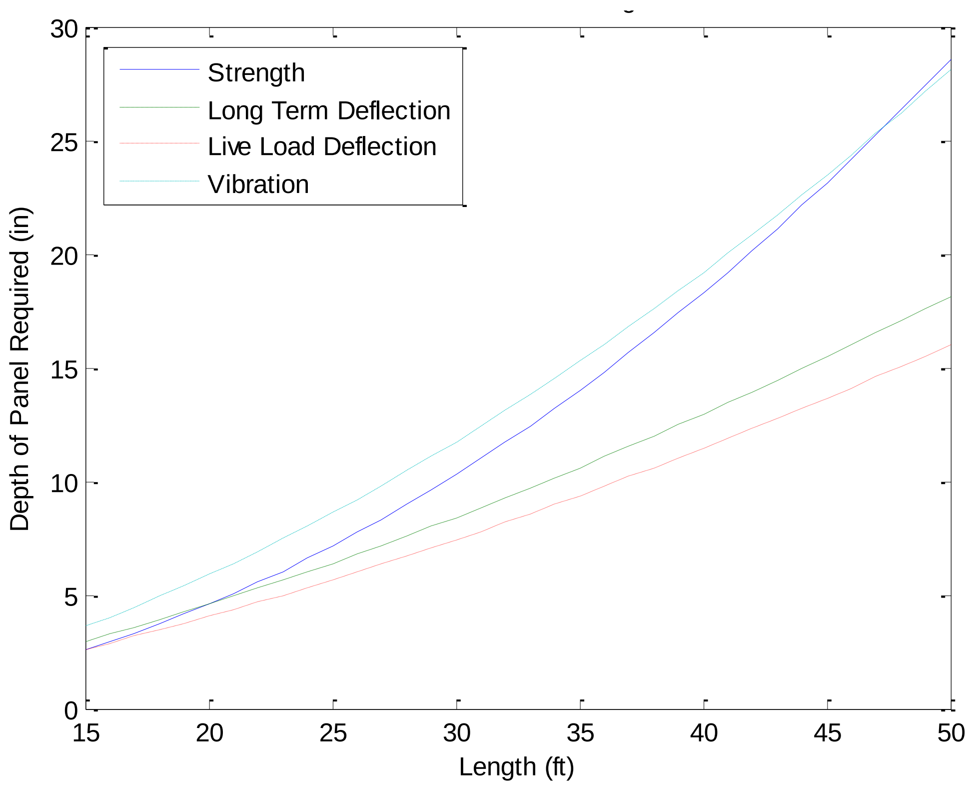

Figure 3.13: Floor Panel with a Flange Consisting of Two Layers of Grade #3 Southern Pine Lumber Parallel to the Span

In conclusion to the results of the MATLAB parametric study, a panel with a flange consisting of a two layers of Grade #2 Southern Pine lumber parallel to the span will be used for the rest of this research. There are multiple reasons that have been mentioned in the prior discussion of the graphs. This configuration has improved the balance of equations even though it is not to the extreme of others possible configurations. Moreover, there is the possibility that the vibrational requirement could be less stringent because of possible improved vibrational characteristics. This would allow for the design equations to be more balance and could end in a very efficient configuration. Another reason was that a less deep panel would be needed which would be more desirable to architects while also creating a smaller concealed space which would cause less complexities with fire design. Also, the fire performance was decided to be of utmost importance therefore a thicker and higher grade flange was desired to be used. Furthermore, the Grade #2 Southern Pine lumber with the thicker flange allows for some residual strength that might be needed in a fire. Also, because the use of Grade #3 Southern Pine lumber is not currently allowed in CLT panels according to the PRG-320 (ANSI/APA), Grade #3 lumber will not be used. Hence, the panel with two layers of Grade #2 Southern Pine lumber parallel to the span was decided to be used.

SAP2000 Computer Model

A computer model was made in SAP2000 to be able to model shear deformations in the panel along with connection slippage at the flange to web interface. Springs were placed between board members to simulate this deformation. Shear deformations from parallel to grain shear along with rolling shear were found to be important to account for when designing CLT because it can account for a considerable difference in strength and stiffness of a panel. Therefore, this is predicted to also have considerable effect on HMT panels and should be taken into consideration when designing them. Normally, shear deformations can be neglected in most cases like a normal wood floor joist because there is a small difference in deflections when they are accounted for. Nonetheless, when boards are placed in rolling shear it has more of an effect on the overall panel since rolling shear has a much smaller shear modulus than parallel to grain shear. This action in a CLT panel can be seen from Figure 3.14, which shows how CLT deforms due to shear stresses. Besides accounting for the shear stiffness of the wood, the springs are placed in the model to allow for any slippage between the flange and the web member that would cause the panel to act partially composite.

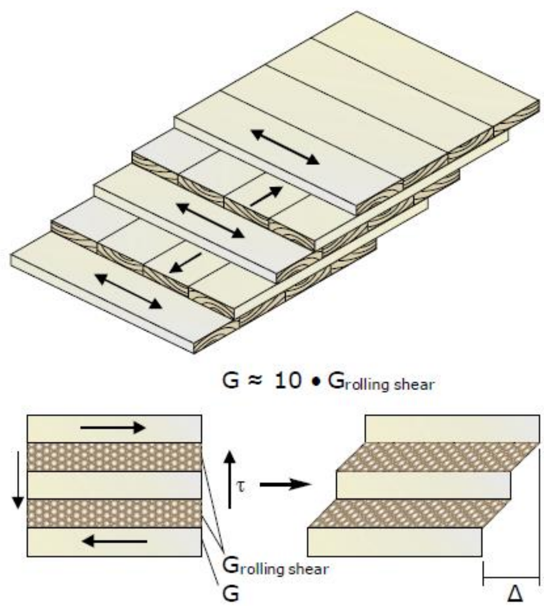

Figure 3.14: Effect of Shear Deformations in CLT (Ross, Gagnon, and Keith)

The predominant method used to analyze CLT panels is the shear analogy method, because it has been shown to be the most precise design, according to literature (Ross, Gagnon, and Keith). Also, out of various methods to account for shear deformations in CLT panels, this method was selected for PRG-320. Because of this, the shear analogy method was used in this present research for comparison to the SAP2000 model.

Some differences in assumptions between the SAP2000 model and the shear analogy method are that the SAP2000 model does not consider the bending stiffness of the crosswise board. The shear analogy method assumes a MOE of $^{1}/_{30}$*E, where E is the MOE parallel-to-grain. Because of the ratio is $^{1}/_{30}$, there is minimal stiffness added to the cross section and therefore, a crosswise board was not added as a frame element in the SAP2000 model and therefore does not contribute this small amount of bending stiffness. Even though the crosswise board wasn't accounted for when considering bending stiffness, it was considered for shear deformations exactly as the shear analogy method does.

The shear stiffness in each model was calculated considering the parallel-to-grain boards placed in the interior of the panel and all of the crosswise boards in the panel. In addition to this, a half of the outside parallel to span layer is considered. The values of the shear modulus' of wood were taken as consistent with the CLT Handbook which sets the shear modulus parallel to the grain to be $^{1}/_{16}$*E and the shear modulus for boards in rolling shear to be $^{1}/_{16}$*$^{1}/_{10}$*E (Ross, Gagnon, and Keith). Even though rolling shear has a smaller shear modulus, parallel to grain shear can cause significant deformations. This is particularly true for the web because the shear stiffness is affected by the width of the member. Since the web is thinner, it has less wood to resist shear deformations. Also, since the web makes up a significant depth of the panel, it can have a greater effect. Both of these aspects are represented in the Equation 3.3, which is shown below and is an example of how the shear stiffness of the springs was calculated. Also, in order to add together all the different shear stiffness's of the different parts of the member in a cross section, Equation 3.4 was used. This Equation relates to adding together spring stiffness when the springs are in series.

$$k_{spring} = \frac{shear\ modulus * spacing\ of\ springs * width\ of\ member}{height\ of\ member} \tag{3.3}$$

$$k_{series} = \frac{1}{\frac{1}{k_1} + \frac{1}{k_2}} \tag{3.4}$$

In order to get the correct mechanical action of the panel, the linear springs that connect the three different elements were restrained rotationally in the R3 direction, axially in the U1 direction and were set to have a translational stiffness in the U2 direction that corresponds to the shear modulus of the wood and the connector stiffness. This translational action of the spring can be seen in Figure 3.15.

Shear deformations were still considered in the frame elements of the SAP2000 model by allowing the shear area in the model to be not modified. All other applicable aspects are consistent with what is stated in the design equations and procedures section in this Chapter. Below in Figures 3.15 and 3.16 are pictures of models.

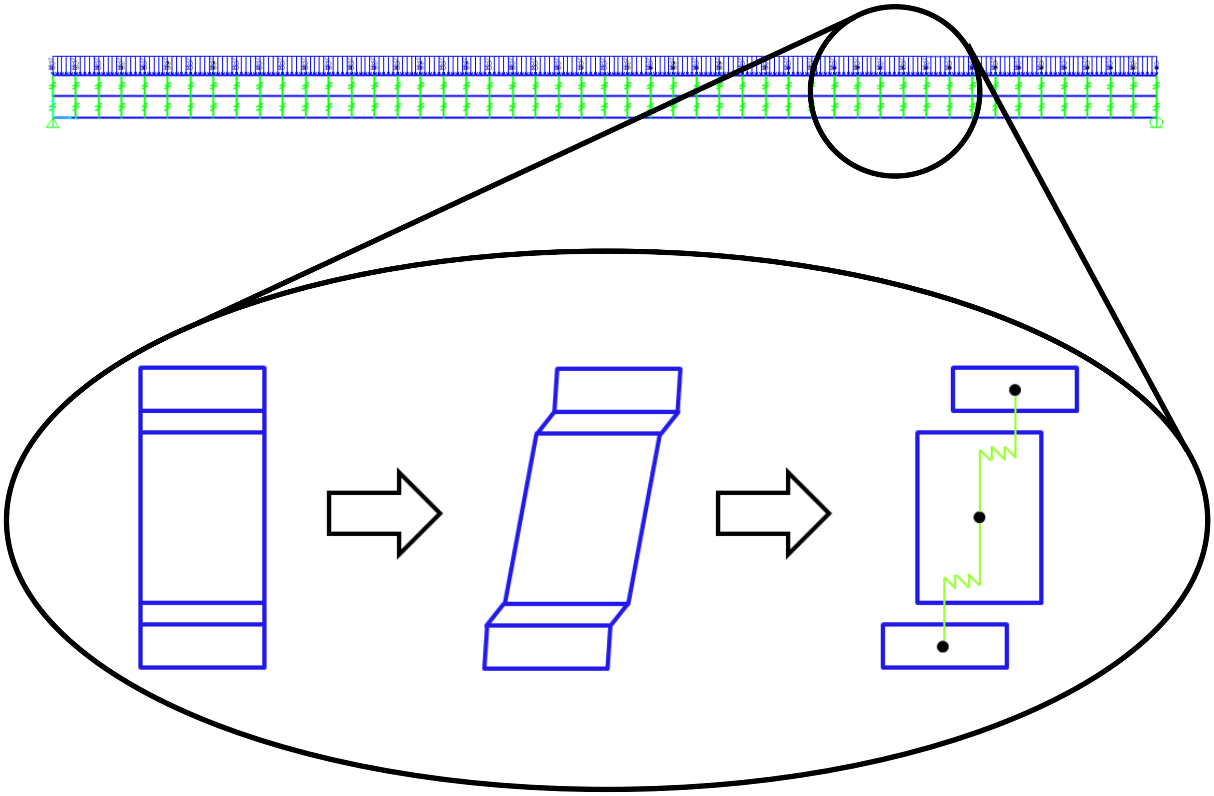

Figure 3.15: SAP2000 Model of Hollow Massive Timber Panel

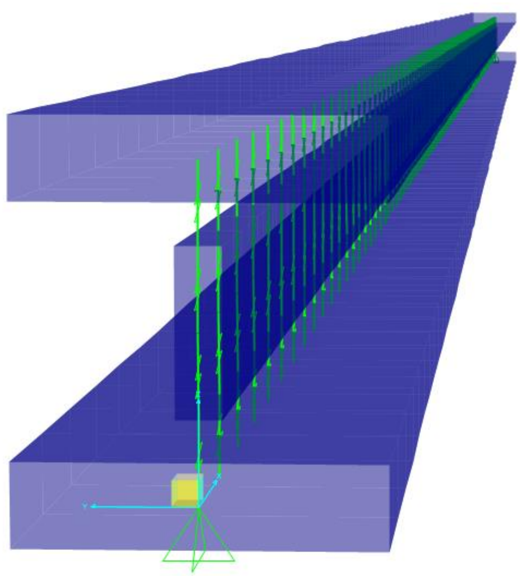

Figure 3.16: Extruded SAP2000 Model

Verification of SAP2000 Computer Model

Since the proposed HMT panel is very similar to CLT, the model was verified by modeling both a CLT panel and a HMT panel with the SAP2000 model and the shear analogy method. The maximum deflections resulted from both methods were then compared and the percentage differences were calculated. This was done for both a fully composite panel that assumed zero shear deformations as well as a panel considering slippage caused by shear deformations. Figure 3.17 shows a picture of the CLT model that was verified and Table 3.1 shows the maximum deflections taken from the analytical model used, which was the shear analogy method, and the SAP2000 computer model. Also, this Table shows the percent difference of the results.

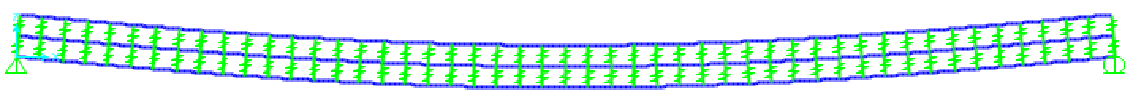

Figure 3.17: Modeled Bending of CLT Panel for Verification of Model

Table 3.1: Comparison of SAP2000 model to Analytical Model

Calculation Description	**Analytical Model Max Deflection[1] [in]**	**SAP2000 Model Max Deflection [in]**	**Percent Difference [2,3] [%]**
Fully Composite CLT Panel	0.364	0.371	+1.85
CLT Panel with Slippage	0.406	0.404	-0.57
Fully Composite HMT Panel	0.924	0.943	+2.05
HMT Panel with Slippage	1.107	1.077	-2.69

[1]The Analytical Model that was used for calculating max deflection was the Shear Analogy Method

[2]Percent Difference was calculated as (SAP2000 Model Max Deflection – Analytical Model Max Deflection) / (Average of SAP2000 Model Max Deflection and Analytical Model Max Deflection)

[3]Positive values show that the SAP2000 model over predicts deflections and negative values show that the SAP2000 model under predicts deflections when compared to the analytical model

From Table 3.1, it can be seen that the SAP2000 model over predicts deflections when comparing fully composite panels, and under predicts deflections when considering slippage due to shear deformations. Also, it can be seen that the percent differences are very small and confirms that this model can accurately predict the panel performance. Some possible sources of error could be related to multiple small differences. These differences are related to 1) the SAP2000 model does not consider the contribution of the crosswise board to bending stiffness, 2) an infinite number of springs were not used in the SAP2000 model, but rather enough were placed till there seemed to be diminishing returns and 3) both methods are approximate solutions.

Accounting for shear deformations in a model according to the CLT handbook for two specific depth-to-length ratios, can cause a difference max deflections of 11 to 22 percent depending on the ratio. These numbers agree with what is shown in Table 3.1 if a full composite panel is compared to a panel with slippage caused by shear deformations. When comparing these values for a HMT panel, a 16.76 percent difference was obtained for the analytical calculations. Therefore, it can be seen that the error introduced in to the model is small considering the overall amount shear deformations make. Consequently, it was concluded that the model developed was accurate enough for predicting the performance of HMT panels in this research.

Besides the values for shear modulus of wood published in the CLT Handbook, the Wood Handbook published by the Forest Product Laboratory has published values for specific species of wood and with respect to the orientation of the wood (Kretschmann). To be more precise with solutions, the Wood Handbook values for shear modulus were used for all SAP2000 computer models. Since the Wood Handbook only list values for three of the species that are in the species grouping of Southern Pine, only the three values given were combined using a weighted average. Loblolly, Long Leaf, and Short Leaf species were given, but not Slash. The average was taken by weighting the standard timber volume grown in the United States for each species. These values are the volumes given in million cubic feet (MMCF), which were found in ASTM D2555 (ASTM, *Establishing Clear Wood Strength Values*).

Description of SAP2000 Parametric Study

The verified SAP2000 model was then used to perform a more detailed parametric study that considered web bending stiffness and strength, and the shear deformations discussed in the previous section. The effects of varying these shear deformations and the effects of connection stiffness at the flange-to-web interface were both considered. This allowed the model to show how the panel performs with partial composite action between the flanges and web. All the variables affecting the properties of the cross section are shown in Figure 3.18. All of these could be treated as variables in the parametric study, but it was found that these graphs would closely resemble what was produced with the MATLAB parametric study. Also, this study will be able to farther compare to the predicted strength and stiffness gains of the MATLAB model and this study will also be used to design panel sections to see potential design lengths for certain depths.

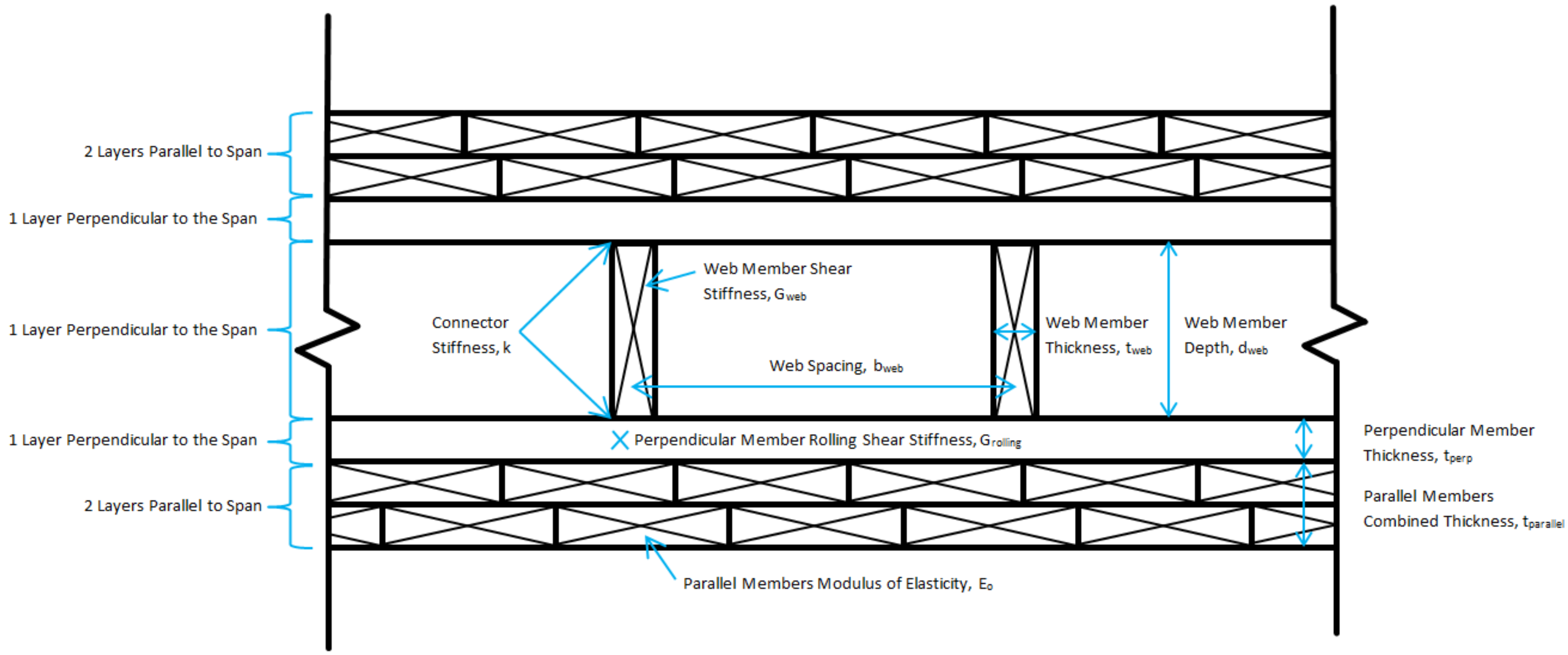

Figure 3.18: Cross Section and Variables Used In SAP2000 Parametric Study

In order to obtain multiple results from changing model parameters many times, a MATLAB script was made to run SAP2000 using API functions. This script was able to open SAP2000, open an existing model, change what needed to be changed, run the model, extract necessary results and process the results in order to find the desired output. This script was done in a loop and then the results were graphed.

A design length for a panel cannot be solved for like when analytical calculations were being used. This is because a computer model is being used to consider shear deformation and non-composite action. Because of this, multiple values must be graphed along with their limiting equations to see where they intersect. At the point of intersection is the design length for that design requirement. The shortest of these lengths when comparing strength, long-term deflection, live load deflection and vibration, is the design length.

If the connector stiffness between the flange and the web is not changing, it was assumed to be a rigid connection. Also, the depth of the web was assumed to be 5.5 inches, which corresponds to a 2x6 on its edge. All other factors are consistent with what is stated in the design equations and procedures section in this chapter and in the SAP2000 computer model section.

SAP2000 Results and Discussion

First, it was desired to see how the MATLAB model output that used analytical calculations compared to the SAP2000 model. Figure 3.19 represents this comparison and shows the difference in these models. As can be seen, both follow very similar trends, but the predicted stiffness and strength gain from the SAP2000 model was slightly less than from the MATLAB model. This was as expected, because shear deformations were added in the SAP2000 model. What was unknown was the amount of difference between the two models and how much the addition of the variables would change the results. Because of the addition of shear deformations in the model, a reduction in stiffness of 13.9 to 22.9 percent was observed along with a reduction in strength of 11.9 to 20.4 percent was observed. Therefore, it can be seen that stiffness is affected slightly more than strength is in terms of percentages but noticeably more in terms of change in ratios.

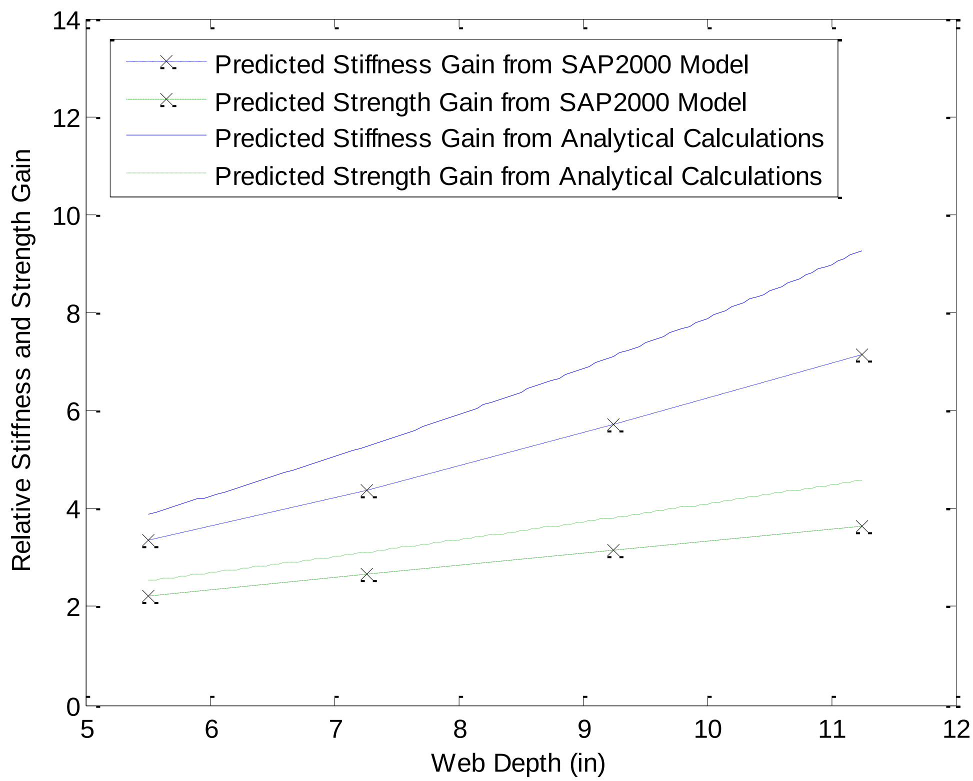

Figure 3.19: Comparison Between MATLAB and SAP2000 Models

In Figure 3.20, comparisons of different assumed shear moduli are compared. The fluctuating line in the Figure is graphing the assumed range of values that could be used for shear modulus. The CLT Handbook states that the ratio of $G_{parallel}$ to E is equal to a range from $^{1}/_{20}$ to $^{1}/_{12}$ (Ross, Gagnon, and Keith). This range was used to graph this line and shows the implications of these assumptions. Also, lines were graphed with the Wood Handbook values for each species in the Southern Pine species group along with using their weighted average. This shows that the CLT Handbook assumption is fairly conservative and if the weighted average Wood Handbook value is used, there could be an increase in stiffness of approximately three percent.

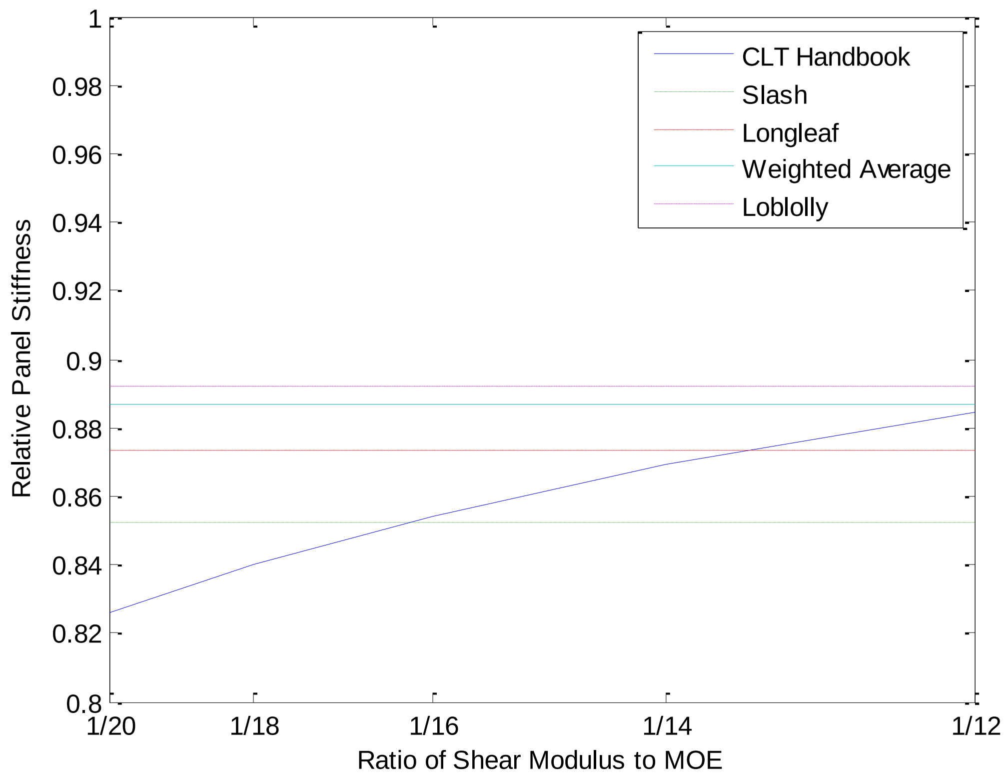

Figure 3.20: Comparison Between Range of Shear Modulus

Figure 3.21 shows how much shear deformations from each component of the cross section of a HMT panel affects the panel performance. These values were obtained by varying the ratio of $G_{parallel}$ to E for each of the three different sections. If there is a larger change in stiffness of the panel, then this part of the cross section affects the performance of the panel more. Therefore, it can be seen that the web, unlike with CLT, affects the amount of shear deformations the most. Then the crosswise boards affect the amount of shear deformations the second most and then the outer flange layer affects it the least.

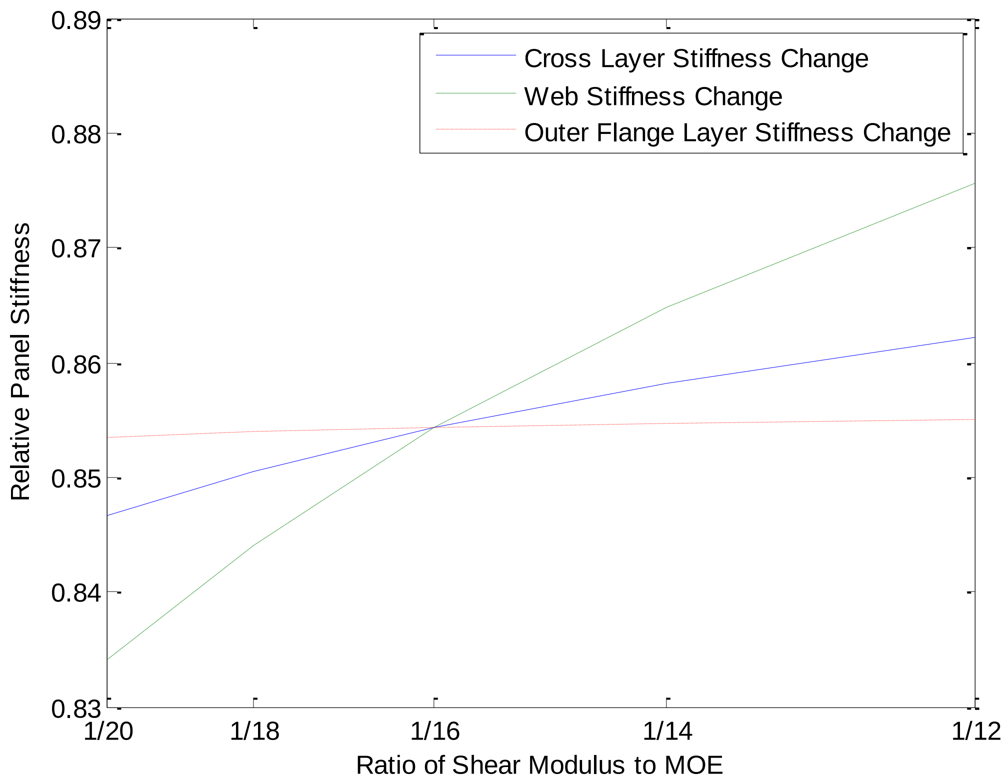

Figure 3.21: Effect of Range of Shear Modulus

Figure 3.22 shows the relative panel strength and stiffness when changing the connector stiffness at the flange to web interface. In this figure, the connector stiffness values correspond to the stiffness assigned to the individual links that were placed at 3 inches o.c. in the SAP2000 model. The strength and stiffness values are relative to a fully composite section, and therefore can be a maximum of unity. It can be seen that both approach a value less than unity and this is because even if the connector stiffness was infinite, the shear deformations would still exist because of how the wood deforms under shear. Also, this graph shows that there is diminishing return after a certain point in connector stiffness.

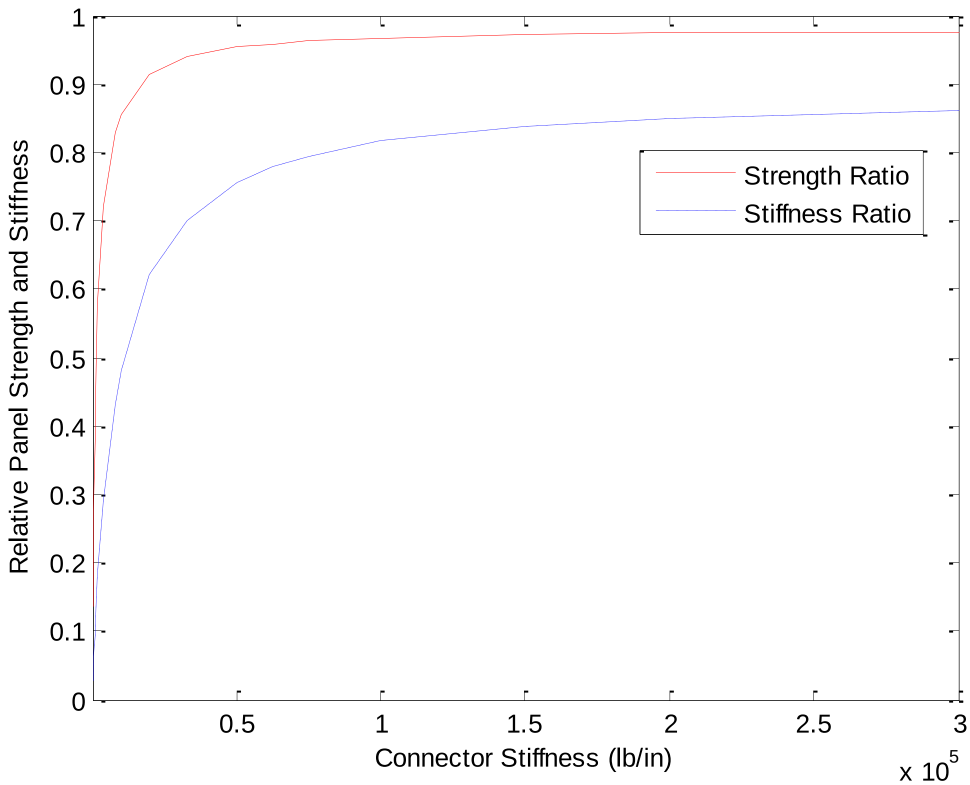

Figure 3.22: Relative Strength and Stiffness Gain vs. Connector Stiffness

Figure 3.23 contains the same values as Figure 3.22, but it is graphed on a semi-log scale where the x-axis is in log scale. This helps show clearly the relationship of connector stiffness to relative strength and stiffness of a HMT panel. It can be seen that not only the strength and stiffness approach values when the connector stiffness went to infinity, but also when the connector stiffness approaches zero. This corresponds to the HMT panel acting completely non-composite. This means that each of the three sections, two flanges and one web, act separately to resist the loads and this is much weaker than if they were connected together. Also, it can be seen more clearly that there is a spot when there is rapid gaining of strength and stiffness as the connector stiffness is increased. This could also be looked at negatively, because it would be very easy to lose overall strength and stiffness of a panel if the connector stiffness dropped a small amount. Furthermore, it has been shown that the connector stiffness is the factor that most effects the performance of HMT panels. Because of this, it will be very important to find the correct connector to obtain meet not just stiffness objectives but strength, cost, and constructability objectives. It would be optimal if the connector stiffness was on the top part of the curve where it really starts to level out. This would allow for a high performance to be reached and allow for more consistency in the overall performance, because a little drop in connector stiffness wouldn't drop the whole panels strength or stiffness much.

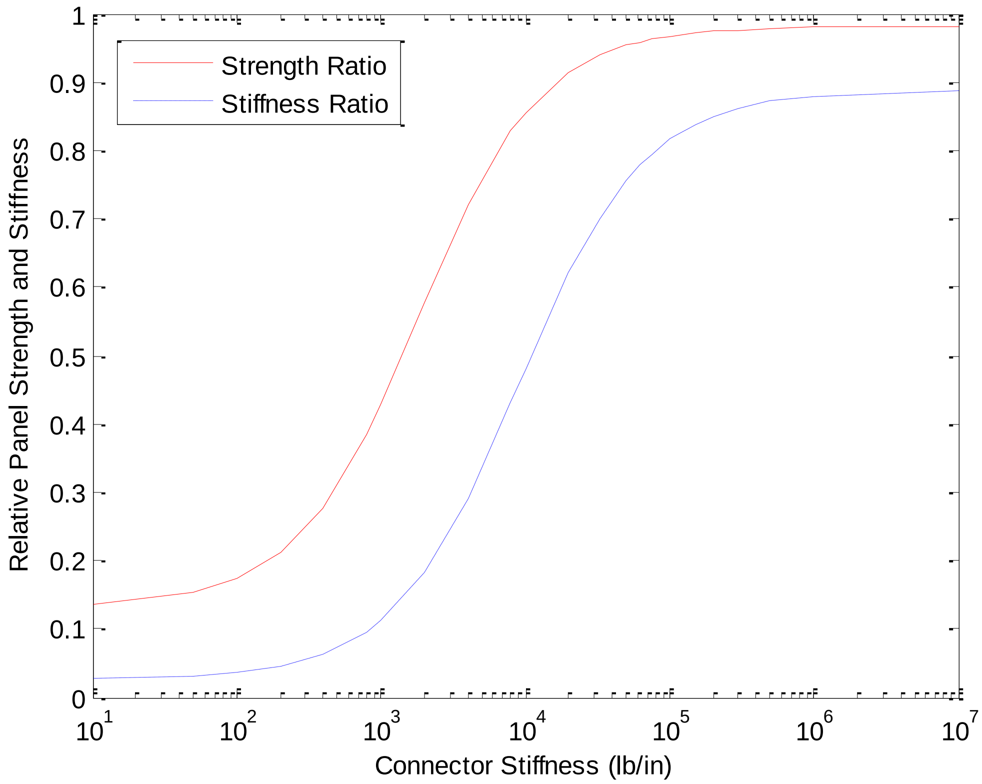

Figure 3.23: Relative Strength and Stiffness Gain vs. Connector Stiffness in Semi-Log

The following part of this section will show predicted panel design lengths using a 2x6 web member and a 2x12 web member, so that the more extreme ranges can be seen. All these graphs assume shear deformations of the wood but not slippage due to the connectors.

To find the design length of the panel, two graphs are made. Each graph will have two plots. The first graph for a panel will include the bending strength and vibrational design. The second graph will include the long-term deflection and live load deflection design. The design length is found when the requirement of the panel intersects the resultant output property of the panel. Then by comparing to see which design length is the length for the different requirements, and overall design can be made. The graphs were made by plotting values at five-foot increments.

Figure 3.24 shows that vibration controls the design length over strength by a few feet. The actual design of the panel for vibration is 34.7 feet and for strength is 37.2 feet. With only a 2x6 web member, this is a considerably long floor span and shows that this panel could be potentially used for many long span applications.

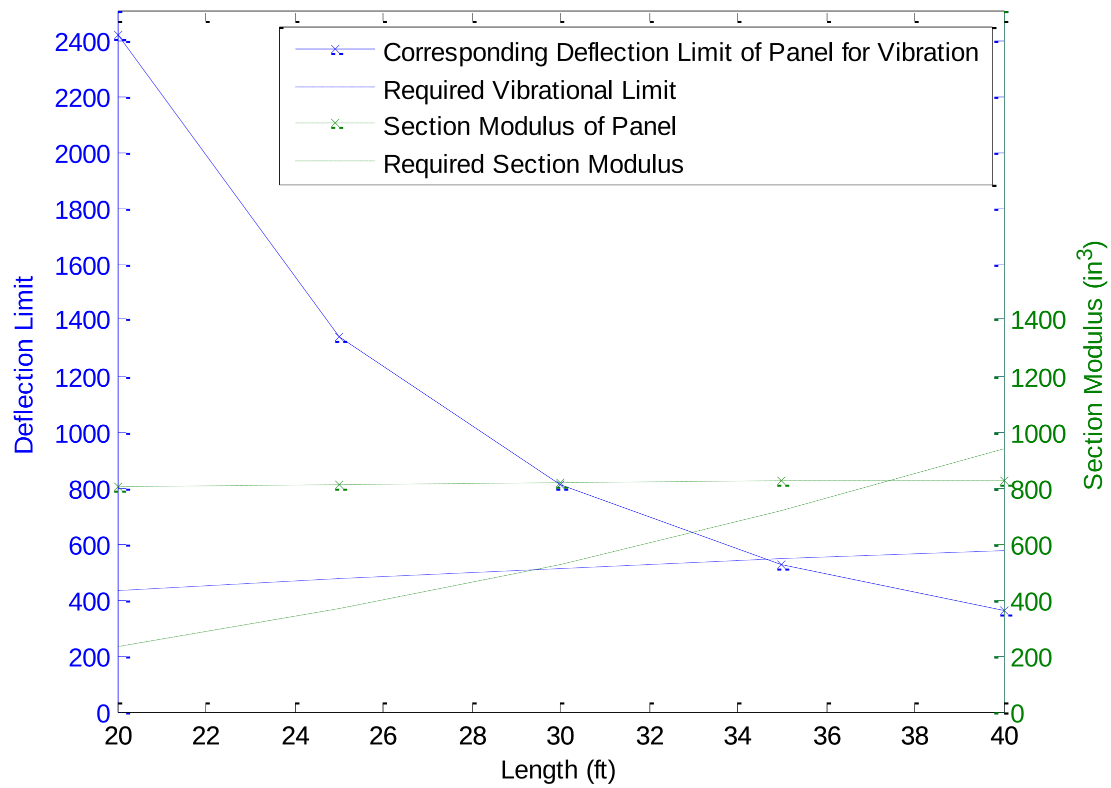

Figure 3.24: Vibration and Strength Design of HMT Panel Using 2x6 Web Members

Figure 3.25 shows that both live load deflection and long-term deflection have design lengths of near 40 feet. Therefore, they are clearly not controlling the design equations. This can be viewed as something very beneficial because this means that this HMT panel is a very stiff panel, and deflections that cause serviceability issues like cracking of sheet rock should not be an issue, along with the fact that it is predicted that the long-term deflections due to creep will surpass the requirements by a large amount. When comparing all four plots on the two graphs, it can be shown that the predicted span length using a 2x6 web member is 34.7 feet.

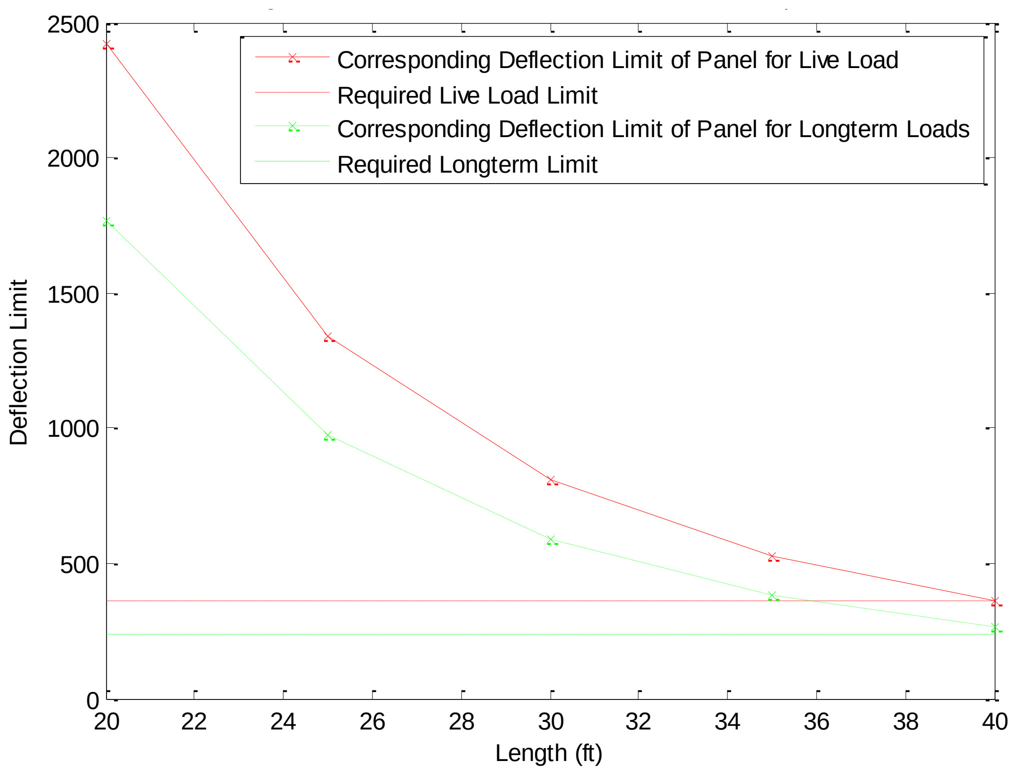

Figure 3.25: Live and Long-term Deflection Design of HMT Panel Using 2x6 Web Members

Figure 3.26 shows the strength and stiffness gain of a panel when comparing it to an equivalent solid panel. From the graph, it can be seen that there are slight stiffness and strength increases as the length gets larger. This same phenomenon is seen with CLT panels. This happens because there is more wood to resist shear as the length grows and therefore produces a slightly larger moment of inertia and section modulus. Also, it can be seen that the stiffness and strength are to be increased approximately 3.75 times and 2.25 times, respectively.

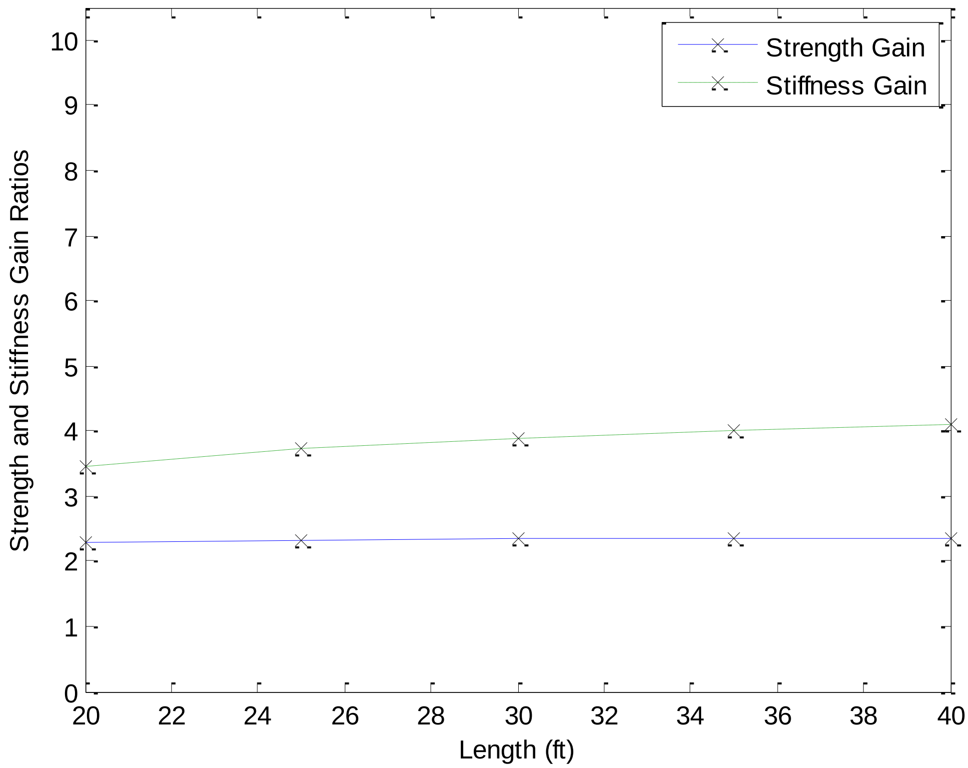

Figure 3.26: Strength and Stiffness Gain of HMT Panel Using 2x6 Web Members

Figure 3.27 shows the strength and vibration design using a 2x12 web member. As can be seen, vibration controls the design of the panel similarly to the panel being designed in Figure 3.24. The actual design of the panel for vibration is 43.5 feet and for strength is 46.8 feet. For using a 2x12 web member, this is considerably long for a floor span length and shows that this panel could be potentially used for many long span applications.

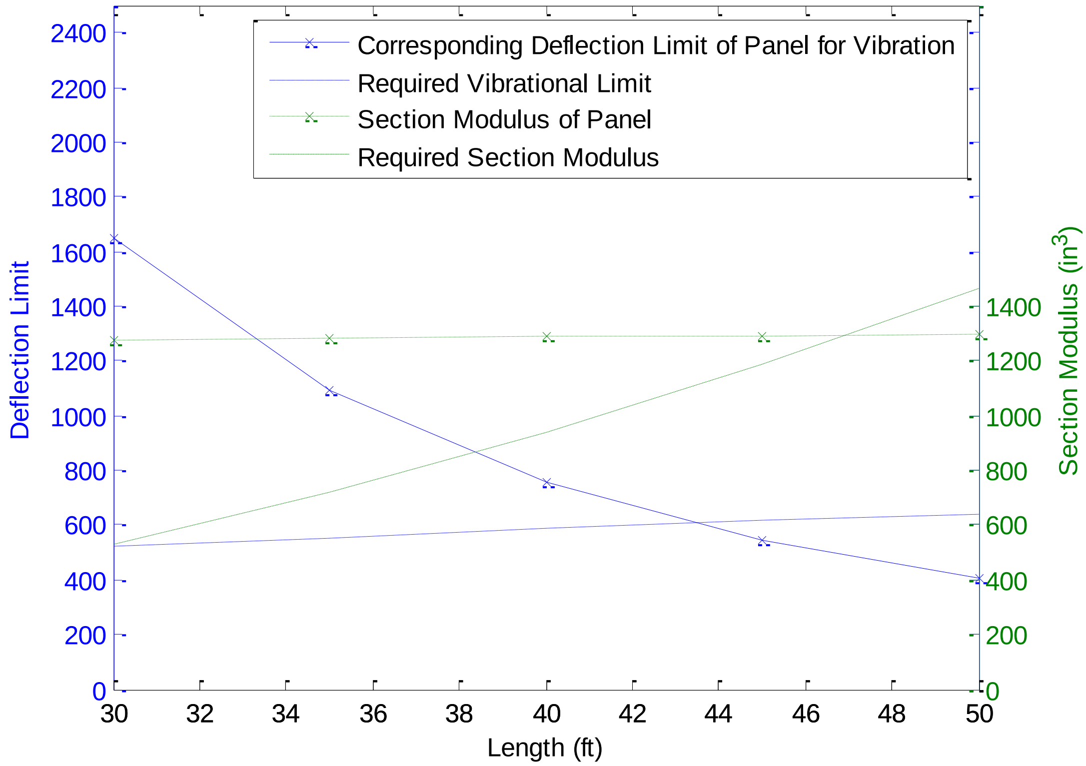

Figure 3.27: Vibration and Strength Design of HMT Panel Using 2x12 Web Members

Figure 3.28 shows the design of a panel using 2x12 web members for live load deflection and long-term deflection. As it can be seen, these clearly don't control the design equations, because the design lengths would be greater than 50 feet.

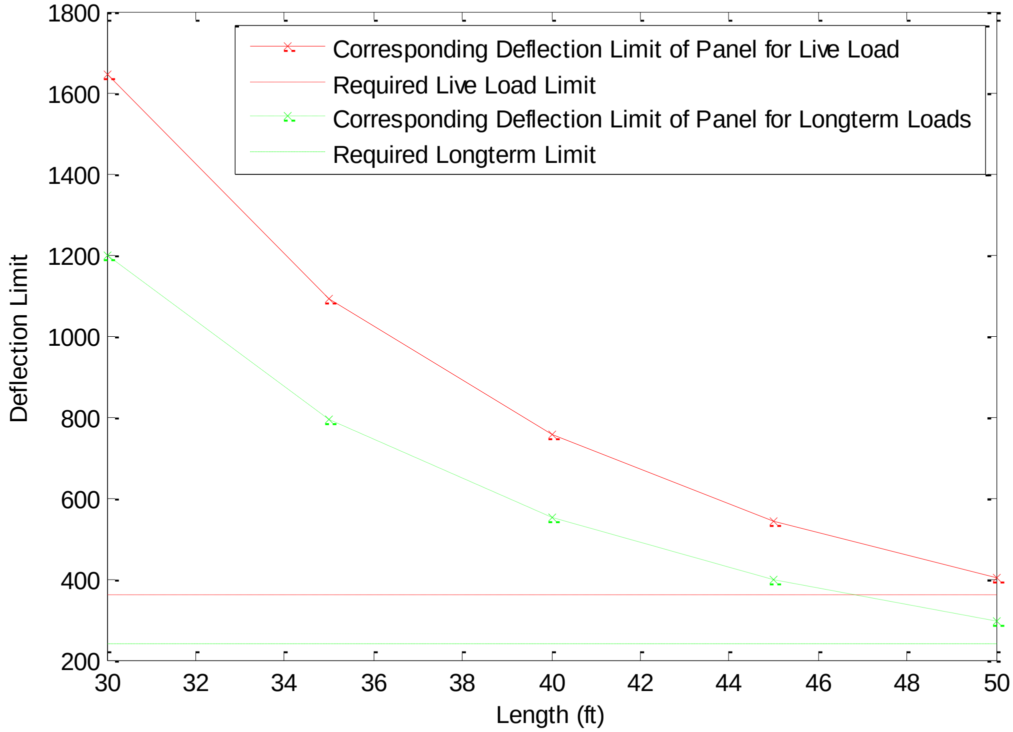

Figure 3.28: Live and Long-term Deflection Design of HMT Panel Using 2x12 Web Members

Figure 3.29 shows the strength and stiffness gain of a panel when comparing it to an equivalent solid panel. Similarly to Figure 3.26, it can be seen that there are slight stiffness and strength increases as the length gets larger. Also, it can be seen that the stiffness and strength are increased approximately 8.5 times and 3.75 times, respectively. This is an extremely large amount that shows the potential efficiencies of this panel.

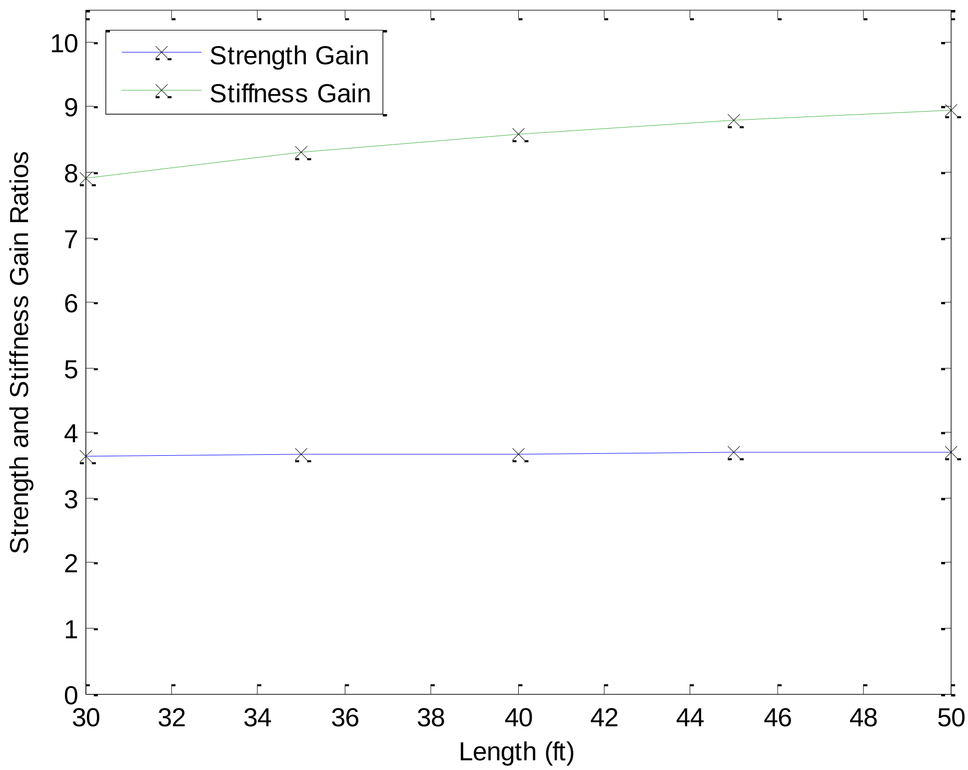

Figure 3.29: Strength and Stiffness Gain of HMT Panel Using 2x12 Web Members

Conclusions

From the studies performed, many properties were investigated and many characteristics were learned about the properties of a hollow massive timber floor panel. The conclusions drawn from these learned properties were 1) a 3-ply flange should be investigated using Grade #3 2x6 Southern Pine lumber for the cross-wise layer and Grade #2 2x6 Southern Pine lumber for a double layer of boards in the flange that run parallel to the span, 2) the flange-to-web connection effects the performance of the panel more than all of the other variables, 3) the SAP2000 model can produce accurate predictions of hollow massive timber panels and 4) HMT panels show very good potential to be a long-span solutions.

CHAPTER FOUR

SHEAR TESTS

Introduction

The parametric studies suggested a desired configuration for the hollow massive timber panel, but the web cross section and the actual connection of a flange to the web were not chosen. Both of these issues were found to be very important topics through the parametric study, and will be address in this Chapter. The web member is very important because it was shown to cause the majority of the shear deformations even though it was not loaded in rolling shear. Also, the web is a very important element because it has to have the strength to carry most of the shear in the panel. Likewise, the flange-to-web connection stiffness and strength heavily influences the performance of the panel. Because of the importance of these items, a closer investigation was made in order to choose the best web member and connectors for the configuration of a HMT panel that was decided on in Chapter 3.

In order to achieve the goals stated above, different web members were analyzed on the bases of strength and non-structural performance. Once a web member was chosen, different web-to-flange connections were investigated. Since connection performance was such an important aspect of the HMT panel, it was decided that an experimental study should be done to find the strength and stiffness of each different possible web-to-flange connection. Descriptions of the studies that were conducted and the conclusions that were drawn from them for the best web member and flange-to-web connection are presented in this Chapter.

Selection of Web Member

Ideas for web members were found by looking at a wide range of products already manufactured by the wood industry. The list of possibilities for the web considered:

1) Dimensional Lumber,
2) I-Joist,
3) Glulam,
4) Structural Composite Lumber,
5) Wood Truss , and
6) Open-Web Truss (wood flanges with a steel truss web).

If it was possible, these options were analyzed with the same shear flow concepts presented in the *Design Equations and Assumptions* section of Chapter 3. Also, these options were analyzed with respect to strength and non-structural issues. The overall shear strength does not just relate to the parallel to grain shear strength of the web, but also to the rolling shear of the wood on the flange that the web connects to. Therefore, it was considered optimal if the shear could spread out to a larger area so that there would be smaller stresses on the lumber placed in rolling shear or if the rolling shear in the crosswise layer could be completely avoided. Both of these concepts were attempted in the tests so that rolling shear would not control the design.

One non-structural issue is cost. It was generally assumed that all options would have similar costs similar, since the greater the shear strength, the greater the cost per web member would be, but also the greater the spacing of the web members could be. Therefore, just because a member was stronger, doesn't mean it would cost more than other options. Other important non-structural issues identified were constructability and fire performance. Constructability was considered in regards to how the web affects the ability to make a connection to the flange and how it affects the difficulty of building a panel. A solution that impairs the ability to make a simple and strong connection to the flange could result in inefficiencies and therefore to higher manufacturing and construction costs. On the other hand, if a very simple solution was chosen there would be the added benefit of shorter construction or manufacturing time, depending if the connections were made in the field or in the shop. Fire performance was considered to be of high importance because if this floor system didn't have adequate fire performance, it will not be accepted as a viable construction material for buildings with greater heights and areas that are not included in Type V construction of the 2012 IBC ("2012 IBC"). Also, if the web member doesn't have adequate fire performance on its own, it could be costly, to make adjustments to the hollow portion of the panel to accommodate for this. Therefore, all options were analyzed according to these requirements.

When designed for a 30-foot span along with standard loads, dimensional lumber with a nominal thickness of a 2x could have a design spacing of 20 inches on-center Also, I-joists were analyzed and found to have a design spacing of around 10 to 12 inches on-center depending on the OSB thickness used for the web of the I-joist. Without considering other aspects of the performance of these members, these two can be ruled out as possibilities of being the web member because of the impractically of having so many web members that are spaced so close together.. This would greatly reduce the constructability of the panel because of the number of members and the amount of connections that will have to be made on relatively small contact surfaces. Also, these options were not considered to have a high fire performance, due to the thinnest of these members.

Different reasonable thicknesses of glulams were considered assuming a conservative value of 175 psi for a reference shear design value for Southern Pine. Normally, Southern Pine glulams have a reference shear design value of 300 psi because the distribution of strength reducing characteristics which results in higher strengths than solid sawn lumber, but these higher reference values were allowed for because of extensive testing. A reason why the more conservative shear reference value was chosen is because there is a parabolic shear stress distribution through the cross section of normal rectangular members while the proposed HMT panel will have a more uniform shear stress throughout the web much like a steel wide flange beam. This could affect the design value because more defects could be engaged and this could increase the probability that a weak board anywhere in the depth of the glulam could affect the shear strength of the web. Also, it was assumed that the glulam would be made of all Grade #2 Southern Pine lumber because higher graded lumber wouldn't be needed for bending like in a normal glulam beam. This also could reduce the value for shear resistance down from 300 psi. Therefore, it is believed that a more acceptable design value is between 175 and 300 psi, but 175 psi will be used for design in this research. From analysis, it was found that a 2.5-inch wide glulam could be spaced at 32 inches on-center and this would allow for three glulams to be placed equally in an 8-foot wide panel. This was a desirable option, because it met all requirements in terms of constructability and fire performance.

The only thing that was not satisfied was that a large surface area was not given at the connection and therefore rolling shear a might control the design.

Structural composite lumber comes in many forms, but a LSL was decided to be considered because of the cost compared to other SCL products and because it has a higher shear strength than other SCL products which is what is needed for the web member that is predominately placed in shear in a HMT panel. This option was eventually rejected because of testing done by Max Closen with inclined screws. Through withdrawal resistance testing of STS, it was observed that these screws had a tendency to split the beam section because the SCL lumber was so dry (Closen), "ASSY Screw Testing"). LSL beams do have advantages like very large shear capacities, the ability to be fabricated straight and long and they can also be cost-competitive with glulams. However since splitting occurred very easily in SCL members, it would not allow for inclined screw connections to be made. Also, because of LSL strength, a thinner section could be allowed, and this wasn't desirable because of fire performance and the fact that a larger area was desired for the flange-to-web connection. A 2.5-inch thick glulam was already thin enough.

Wood trusses and open-web trusses were ruled out because of two fire performance issues: web member provided no compartmentalization within the hollow portion of the panel and the webs would be very thin. Lack of compartmentalization could allow fire to spread throughout the panel and the building if fire were to get inside of the panel. Also it states in NFPA 13 that sprinklers can be omitted in concealed spaces under three scenarios: 1) physically infeasible to place a sprinkler within the space, 2) the space is completely filled with insulation, or 3) area is fire stopped with volumes no bigger than 160 cubic feet. Because fire blocking would have to be installed in order to compartmentalize the hollow space, the two options that include open webs were not desirable. The second reason would be that if fire got into the hollow area, these members would lose strength more rapidly because there is no protection against fire due to the small cross section compared to surface area.

In conclusion, a glulam web of 2.5-inch width will be used with a design spacing of 32 inches on-center This could be a very cost effective option because two 2.5-inch wide glulams could be made from ripping one 2x6 stock glulam. Also, this glulam would be made of Grade #2 Southern Pine lumber, and therefore would be cheaper than a typical glulam. Moreover, this would allow for a heavy timber member to be used that would provide adequate compartmentalization and slower reduction of strength if exposed to fire. One potential issue could be that rolling shear could limit the design of the flange to web connection. This will be farther investigated through experimental shear tests on the flange-to-web connection.

CLT and Glulam Production

CLT panels and glulam beams had to be produced in order to make specimens for testing the flange to web shear connections. Since at the time of the experimental testing for this research there were no producers of CLT in the United States, sufficient quality CLT panels were manufactured on Clemson University's campus. In addition to this, the needed glulam beams were chosen to be made on campus as well. Materials used to make these products as well as the process to manufacture them are discussed below.

Donated 2400f_b 2.0E Southern Pine MSR lumber was used for the production of Tests #1-16 while Tests #17-21 used Grade #2 and #3 2x6 Southern Pine lumber. The lumber was changed because the MSR lumber had an average density greater than a density based on a specific gravity of 0.55 for Southern Pine lumber. The density of the wood could affect both mechanical and glue connections and it was decided that it was important to change the wood to get a more realistic design situation. Changing to Grade #2 and #3 lumber reduced the density to approximately the design density value.

Two adhesives were used during testing. They were Melamine Formaldehyde (MF) and Emulsion Polymer Isocyanate (EPI). Benefits and disadvantages are listed in Table 4.1. Both were considered throughout experimentation and which adhesive was used will be stated for each specimen. All recommendations of the adhesive manufacturer were followed when applying and pressing the specimens.

Table 4.1: Advantages and Disadvantages of Each Adhesive Used

Melamine Formaldehyde (MF)	Emulsion Polymer Isocyanate (EPI)
- Uses formaldehyde in adhesive, but is Greenguard Children & Schools Certified and Indoor Air quality certified (“Greenguard Certification”) - Has a relatively short cold press time but longer than the recommended EPI press time - Is relatively more expensive - Has a clear glue line - Has passed adhesive standards for CLT	- Does not use formaldehyde in adhesive, but does contain potentially harmful chemical, Isocyanate. Isocyanate is not harmful after being mixed, but while still unmixed, it is important to handle properly - Has a very short cold press time - Is relatively less expensive - Has a white/clear glue line - Has a foaming effect when mixed

and glulam structural members - Is a certified adhesive for manufacture of CLT or glulam members	which gives it some ability to gap fill - Has not been certified for manufacture of CLT or glulam members

The next several paragraphs will focus on the procedure of making the specimens. Before pressing, all boards were cut to 4-foot lengths and planed down to $1^3/_8$-inch thickness by taking $^1/_{16}$ of an inch off each wide face of the board. The adhesive and hardener were mixed in the proportions recommended by the manufacturer. Both adhesives were allowed to press for 2½ hours under 150 psi of clamping pressure. A Newman press was used to press 4-foot by 4-foot, 3-ply CLT panels that were then cut up into 8 CLT flange pieces approximately 12 inches wide by 24 inches in length. A pressure of 150 psi was recommended by the manufacturer to insure all boards were flattened to produce a good bond. This is a very high pressure when compared to many wood laminating processes, but it was necessary because Southern Pine has a high stiffness and density.

Figure 4.1: CLT Panel in Press

The flanges of the specimens were made from the CLT panels produced. These panels were not edge-glued but only bonded by the face of the board. While pressing the CLT panels, there was no significant clamping device used to produce side pressure on the panel to eliminate gaps between the edges of boards. However, large shims were used to induce small side pressures to minimize gaps. Since smaller length boards were used, there was less overall curve in the boards and this also helped reduce gaps. When 2400f_b MSR lumber was used, all boards were of the same grade, but when visually graded lumber was used, Grade #2 lumber was used for the parallel to span layers and Grade #3 lumber was used for the crosswise layer. This corresponds to what is used for the V3 CLT grade in the PRG-320.

Because of the geometry of the test specimens, the CLT flanges were not highly stressed. Therefore, the CLT was not really being tested, but rather the connection of the flanges to the web and the glulam web member was subjected to high stresses. This allowed for the quality of the CLT to have little effect on the performance of the tests. What could have affected performance of the connectors being tested was that there were small gaps between the boards that made up the CLT flange and there was a large difference in the specific gravity of the wood used for testing in comparison to the design specific gravity. Gaps between the boards in the wood could have effect on both glue connections and mechanical connections, but it was decided to be minimal for all cases except for inclined screws. To reduce the effects of these gaps on inclined screws, the screws were placed at least 0.5” from the middle of the screw to the edge of the gap. In practice CLT would have side pressure applied to it which will reduce the size of the gaps, and also it would be recommended for end use that screws would be not be placed in gaps of a CLT panel.

Glulams were made by cutting 2x6 boards in to 4-foot lengths and ripping each board to make two equal widths. These were planed to $1^3/_8$” thickness prior to gluing together a stack of eight boards to make an 11-inch deep glulam. Multiple glulams were pressed at 150 psi all at once. After pressing, the glulams were planed to 2.5 inches in width and cut in half to approximately two feet in length for use as the web member for the test specimens.

Figure 4.2: Four Glulams in Press

The moisture content (MC), weight and dimensions were taken for each CLT flange and glulam web member. These were used to find average MC, densities and oven-dry specific gravities of each element of the specimen. The MC was found using a

Delmhorst J-4 moisture meter with insulated pins that were driven to an average depth of 1¼ inches. The average MC was taken for each flange and web. One sample was taken from each the top and bottom surfaces of the flange for a total of two samples. Four samples were taken on a web member so that one was taken on each end of the top and bottom surfaces. This was done because connectors were to be placed on these interfaces on both sides of the glulam. In oven-dry specific gravity calculations, moisture content was accounted for by back calculation using the Equation 4.1 that is used to calculate density of wood in the NDS. Throughout the rest of this thesis, oven-dry specific gravity will just be referred to as specific gravity (SG). Also, the average calculated values for each test are shown in Table 4.2 for the CLT flanges and Table 4.3 for the glulam webs.

$$density = 62.4\left(\frac{SG}{1+SG*0.009*MC}\right)*\left(1+\frac{MC}{100}\right) \tag{4.1}$$

Table 4.2: Moisture Content, Density and Specific Gravity of CLT Flanges

Test	Moisture Content [%]	Density [lb/ft^3]	Specific Gravity [-]
#1	15.1	41.5	0.63
#1 Redo	13.1	40.7	0.62
#2	15.1	41.4	0.62
#2 Redo	14.3	40.7	0.62
#3	13.6	39.6	0.60
#4	12.7	40.8	0.62
#5	13.2	41.5	0.63
#6	13.3	39.5	0.60
#7	12.7	40.8	0.62
#8	13.2	41.5	0.63
#9a	11.8	42.3	0.65
#9b	13.5	42.9	0.65
#10a	13.5	41.0	0.62
#10b	13.6	40.6	0.62
#11	14.6	42.3	0.64
#12	14.7	41.0	0.62
#13	13.3	39.5	0.60
#14	13.1	39.5	0.60
#15	13.4	41.5	0.63
#16	11.6	39.5	0.60
#17	12.5	35.3	0.53
#18	13.1	34.3	0.52
#19	12.0	33.7	0.51
#20	12.9	35.1	0.53
#21	13.1	34.3	0.52

Table 4.3: Moisture Content, Density and Specific Gravity of Glulam Webs

Test	Moisture Content [%]	Density [lb/ft^3]	Specific Gravity [-]
#1	14.7	41.1	0.62
#1 Redo	12.0	42.8	0.65
#2	13.1	42.7	0.65
#2 Redo	12.8	41.8	0.64
#3	13.1	41.4	0.63
#4	11.7	41.5	0.62
#5	12.5	42.1	0.64
#6	12.0	40.8	0.62
#7	11.6	41.7	0.64
#8	11.5	41.0	0.62
#9a	11.0	40.7	0.62
#9b	12.6	42.6	0.65
#10a	14.0	43.1	0.66
#10b	13.7	36.6	0.55
#11	13.1	41.9	0.64
#12	13.5	41.5	0.63
#13	10.3	43.0	0.66
#14	13.7	41.4	0.63
#15	10.6	42.4	0.65
#16	9.7	39.5	0.61
#17	12.2	37.3	0.57
#18	11.1	36.7	0.56
#19	12.1	37.2	0.56
#20	11.1	37.8	0.58
#21	11.9	37.3	0.57

From Tables 4.2 and 4.3, the difference between Tests #1-16 and Tests #17-21 is because of the difference in lumber used. Generally, the glulam web members were found to have a slightly higher specific gravity than the flanges. This is assumed to be as a result from gaps being present in the CLT flanges but not in the webs. Also, the moisture content of the wood was in the allowable MC range specified by the manufacturer of the adhesive, except for Tests #1 and #2, but these were not far outside the range and the test results from these specimens would not have changed any conclusions. A database of measured raw data along with processed data is archived in the spreadsheet titled “Moisture Content and Density” (see Appendix).

After the flange and webs were made and cut to size, each specimen was put together based upon the specifications for the specific test. Each test had used different connectors or a different configuration. All tests had the same geometry with exception of tests that used notches in the flange or a spreader board. With this exception, all that would change is the distance between the flanges or how wide the last board on the web was. A diagram of the standard set up is shown in Figures 4.3 and 4.4.

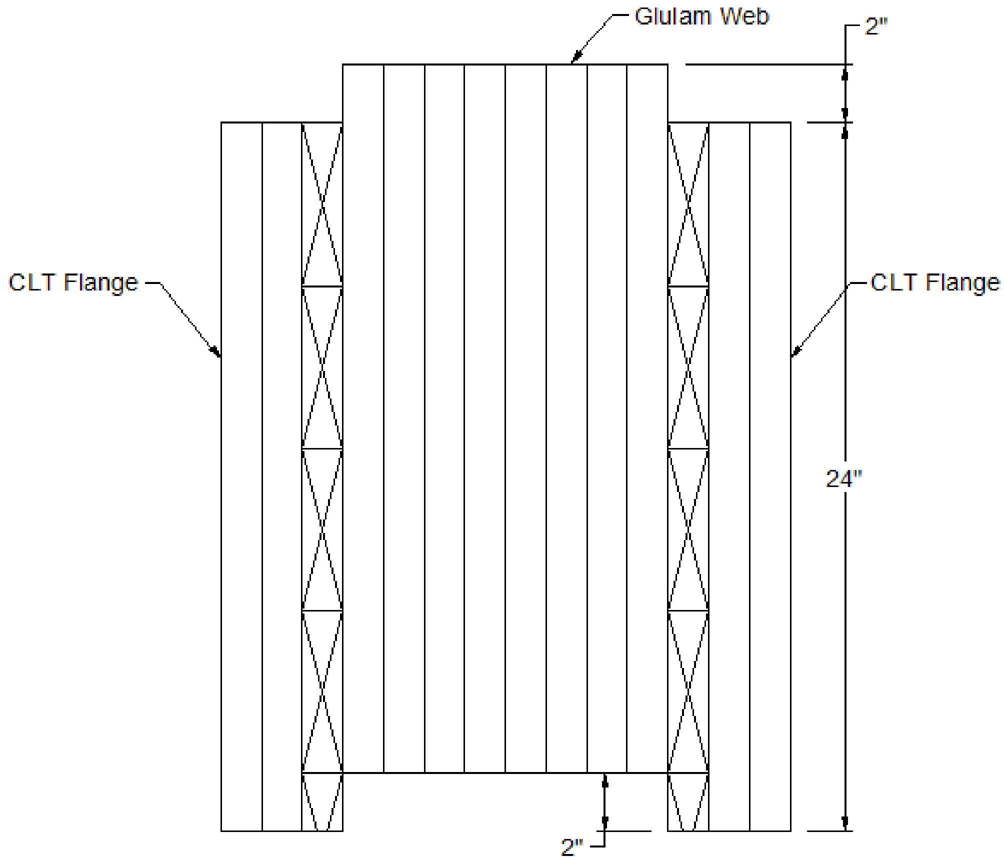

Figure 4.3: Side View of Standard Specimen

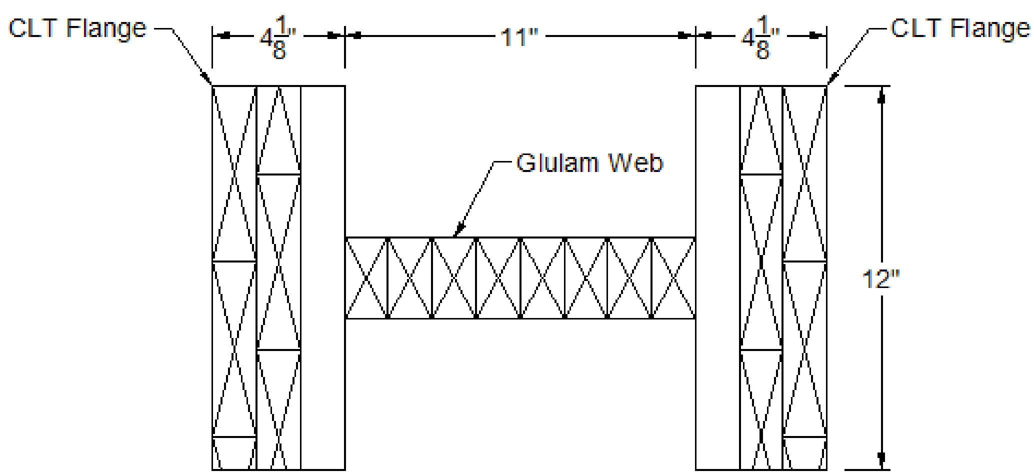

Figure 4.4: Top View of Standard Specimen

Experimental Testing Set Up

In order to test the different flange-to-web connections in shear, a force was applied on top of the offset glulam while having a reaction force on the bottom of each of the CLT flanges. This causes half of the applied force to be resisted by each of the flange-to-web connections along with half of the applied force to be resisted by each side of the glulam. This two-sided set up allows for the specimen to be symmetrical and no eccentricity applied.

A self–reacting steel frame was used to apply the load and support the specimen. A steel beam was place in its weak axis and hung from two main supporting beams by threaded rods. Since the CLT flanges sat on top of the web of the steel beam, the web was reinforced to take the two concentrated reaction forces of the specimen's flanges. An MTS actuator with a capacity of 145 kips in compression was used. A plate with angles along with a steel bearing plate was used to stabilize the top of the specimen and to apply the load to the web from the actuator. The steel bearing plate was 1½ inches in thickness, 7½ inches in length, and 3½ inches in width. The contact surface was large enough to prevent crushing of the wood fiber. The length of the bearing plate was dimensioned so that all the force would enter the wood before the last glue line in the web so that at least one glue line on each side of the web would be tested along with the shear strength of the wood in the web. Thin metal shims were placed under the outside of the flanges so that the rolling shear strength of the wood would be tested. This was thought to be effective because the shear force would have to travel through inner layers of the flange to get to the outer layers so that the reaction can take place under the outside of the flange. It was not desired for the force to travel straight down in the first layer of the flange and not have the possibility of a rolling shear failure in the crosswise layer in the flange. Figures 4.5 and 4.6 shows pictures of this test set up.

Figure 4.5: Standard Specimen Set Up

Figure 4.6: Test Framed Used for Shear Tests

Multiple sensors were placed on the specimens. Since the flange-to-web connection was being tested and the stiffness of this connection was the most important factor for the overall performance of a HMT panel, the displacement at this connection was measured with four LVDT sensors. Each was placed on either side of the web and on each side of the specimen. The LVDT was screwed to the flange while a metal bracket placed under the rod of the sensor was screwed to the web. Each plate was placed 12 inches down from the top of the web member. In Figures 4.7 to 4.9 the locations of the LVDT's are shown and how the sensors and sides were labeled.

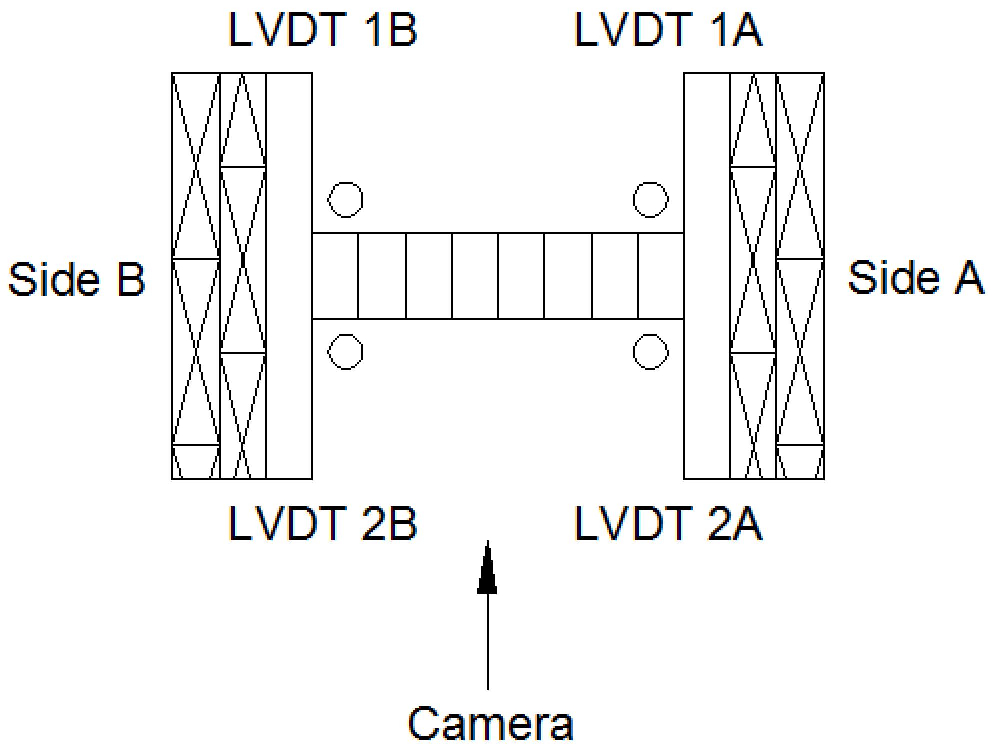

Figure 4.7: Sensor Labels and Side Labels

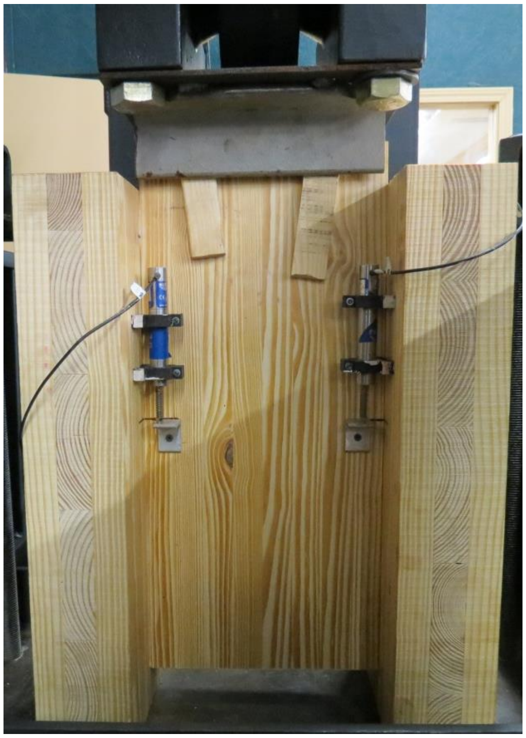

Figure 4.8: Placement of LVDT Sensors

Figure 4.9: Set Up of LVDT Sensors

The load on the specimen was recorded using a load cell that was placed between the actuator cylinder and actuator head. With both the load and the displacements at the connection, stiffness along with a maximum force could be obtained. Since the steel bearing plate on top of the web was placed in the middle of the web, it was assumed that half of the load went to each side.

During the first test series of shear tests, it was observed that the specimens would fail by splitting along the middle of the web into two pieces. This was not expected and was concluded to not be a shear failure because failure plane was not in an area of high shear stress. If a shear failure occurred, it should have occurred in the part of the web closer to the CLT flanges. This is because the entire load was not applied until outside of the steel bearing plate. Rather, a failure in the middle of the specimen suggests a bending failure by creating tension stress perpendicular to the grain. An example of this failure can be seen in Figure 4.10.

This failure would not be seen in a full-scale HMT panel resisting in out-of-plane bending. When used as a floor panel, there would theoretically be zero bending stresses in the middle of the web because this is where the neutral axis is located. Furthermore, there would be minimal bending stresses in other parts of the web because of being closer to the neutral axis than the flanges. Also, none of these bending stresses will act perpendicular to the grain unlike what has happing with this unexpected failure. Therefore, it is an error in the test setup that must be remedied.

Figure 4.10: Bending Perpendicular to the Grain Failure Mode

A similar set up with a symmetrical double-sided test was used in research conducted at University of British Columbia (Closen) and Karlsruhe Institute of Technology (Blass, Bejtka, and Uibel) except it was not believed that they used any reinforcement. Two options were investigated that would eliminate the tension perpendicular to grain failure. At first steel rods were used in order to prevent the bottom of the specimen from spreading apart and provide an alternative load path. The second option investigated was a thin plate placed under the specimen that had a tube welded along the ends of it. This plate was used with metal shims to hold the bottom of the specimen together and provide the alternative load path. The rods were used only in Specimen #3 of Test #1 and are shown in Figures 4.11 and 4.12. Specimens #1, #2 and #4 of Test #1 and Specimen #1 of Test #2 were tested without either the rods or the plate. All other test specimens used the plate with shims that can be seen in Figure 4.13.

Figure 4.11: Test Specimen Reinforced with Rods

Figure 4.12: Rod Reinforcement

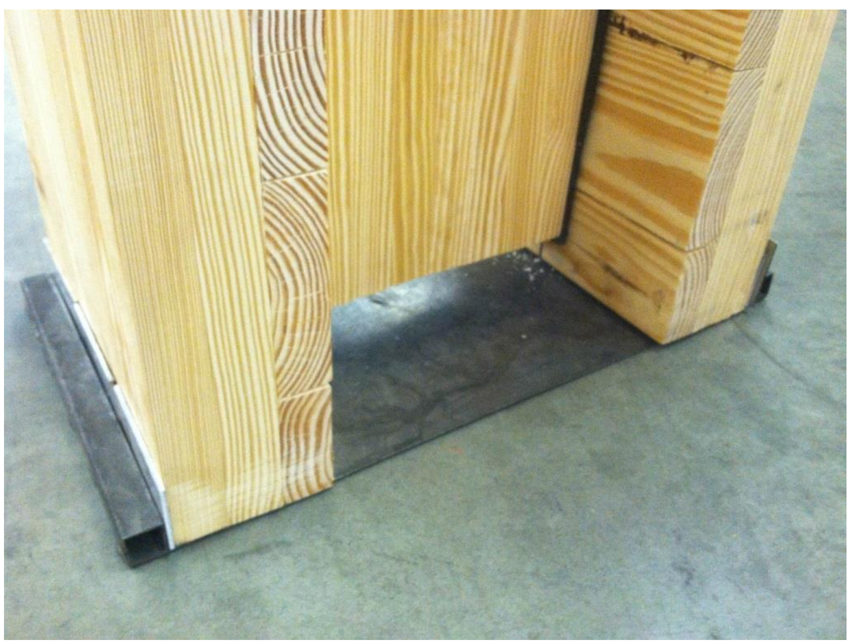

Figure 4.13: Test Specimen Reinforced with a Thin Plate with Shims

The plate with shims was used because it was easier to implement. With this test set up, the specimen was set on top of the thin plate used to restrain the spreading of the flanges and shims were used to produce a snug bearing surface to produce an immediate engagement. By doing this different stresses were produced than what the panel would see as a floor panel in bending. It was observed that usually, if the test was strong enough to get over a certain value, the web would split in the middle at the bottom. Usually this could be identified by a smaller pop that happened at about 50 kips of load. Since the web is not resisting any of the moment any more, the steel plate with shims is resisting all of the moment and stabilizes the specimen.

The potential differences in stresses are accustom to any small scale testing because of having to produce artificial boundary conditions. With a one-sided shear test, two methods can be used. Either the test set up uses a slanted specimen so that no moment is introduced or uses a test set up that includes rollers so that the moment caused by eccentricity does not rotate the specimen. Both of these methods apply a compressive force onto the joint and therefore are not any different than a double-sided test. The major variable that changes the amount of moment placed on the joint is the eccentricities of the loads.

Although there are possibilities of different stresses, these tests were not to be used for obtaining design values. These tests were mainly used to compare the performance of different connectors and to obtain the stiffness of each connector needed for full-scale modeling of an HMT panel. Small scale test specimens were chosen because only a limited amount of full-scale tests can be conducted in future research. The full-scale specimens will be used to see if there are any differences observed in the connectors properties. Overall, using a steel plate under the specimen with shims in order to keep the lower part of the flange from spreading so that an unwanted bending failure wouldn't occur was deemed acceptable for the purposes of this research. It would be suggested in further research to monitor the stiffness of the joint in a full-scale bending test of a HMT panel to compare to values obtained in this research and therefore see if any error was produced by this test method.

Testing and Analysis Procedures

The tests were run using the MTS software to control the actuator that applys the force to the test specimens. All specimens were tested using displacement controlled loading with a simple ramp function. Once a test was started, the actuator head would move at a constant displacement downward on the specimen until failure was reached. On all glue specimens or combination of glue and screw, a rate of 0.2 in/min was used. ASTM D905 requires this rate of movement (ASTM, *Adhesive Bonds in Shear by Compression Loading*). A rate of 0.1 in/min was used for mechanical connections based upon the requirements of ASTM D1761 (ASTM, *Mechanical Fasteners in Wood*).

Before testing, each specimen was inspected for defects and any were noted if found. This was done so that quality of construction could be monitored and this could explain any outliers. Also, this was done as a part of rating the constructability of a connection, so that it could be seen if it would be difficult to make a quality connection. After the test, pictures of the failures were taken and failure mode was recorded. For glue connections, an estimated wood failure was noted. The post-test inspection also helped to see if there were any hidden defects within the connections.

Data for applied force on the specimen was measured by the load cell attached to the actuator. This data was collected through the MTS software. The steel bearing plate was placed in the middle of the specimen, so that each side received half of the load. Slippage at the joints was measured by the four LVDTs and this data was collected with the data acquisition program Quick DataAcq. For all glue bond tests, data was collected at roughly 5-kip intervals and with all mechanical connector tests, data was collected roughly 2-kip intervals.

When analyzing the displacements, the average of the displacements on each end of the web member was taken. These would correspond to a displacement for side A and B of the specimen. Then both side A and B were averaged to obtain an overall displacement of the web member. Using both the displacement and force data, the stiffness could be obtained. If it was a glue connection, the stiffness was normalized by the length of the glue connection. If it was a screw connection, the stiffness was normalized by the number of screws. Other mechanical connections besides screws were normalized by length.

It was decided to simply average the displacement values in this way because it was observed that there could be several potential errors occurring. A twisting of the web from side to side was observed when side 1 and side 2 data split and went opposite ways. This can be seen in Figure 4.14. This shows that one side is moving upward and the other side in moving downward. Since the web is being pressed downward and one side was moving upward, then it could be concluded that the web was twisting.

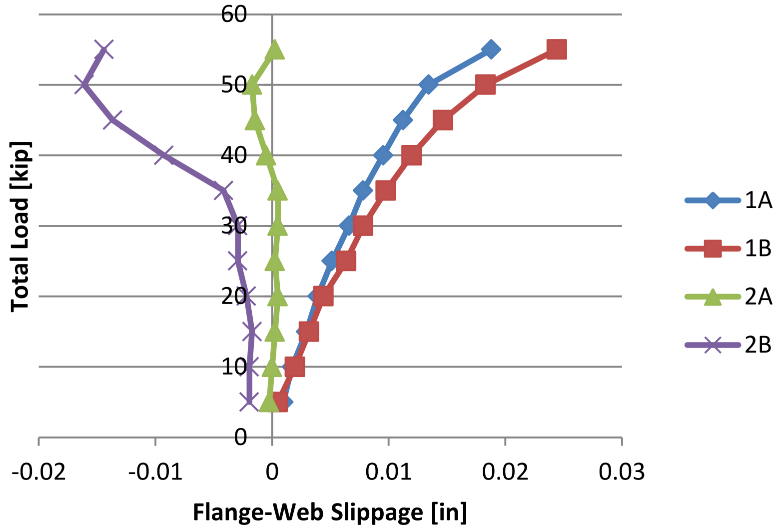

Figure 4.14: Web Twisting Side to Side

Unequal loading was likely observed when one side of the specimen was displacing more than the other. This could be allowed since the actuator head could pivot to accommodate different movement on both sides, and since the steel loading plate could have been placed farther to one side than the other. Also, because only one load measurement was taken, it was unknown how much load was on each side. This also could have been because there were different stiffness's of the connectors on each side, but if the gap was large enough it was assumed to be because of unequal loading. This can be seen in Figure 4.15.

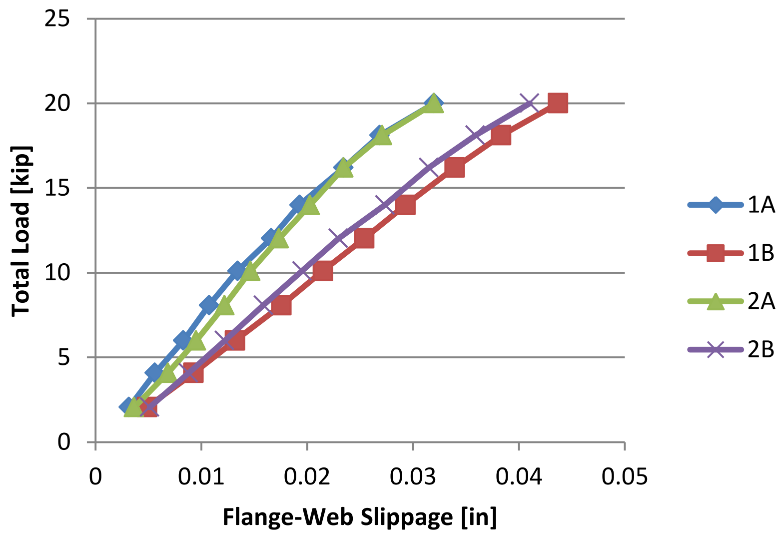

Figure 4.15: Unequal Loading of Each Side of the Specimen

A specimen that sat uneven was observed when opposite corners of the web displaced similarly but were seemed to split from the other two that were on opposite corners. This is shown in Figure 4.16 where sensor 1A and 2B have similar displacements and 1B and 2A have similar displacements and both sets move away from each other.

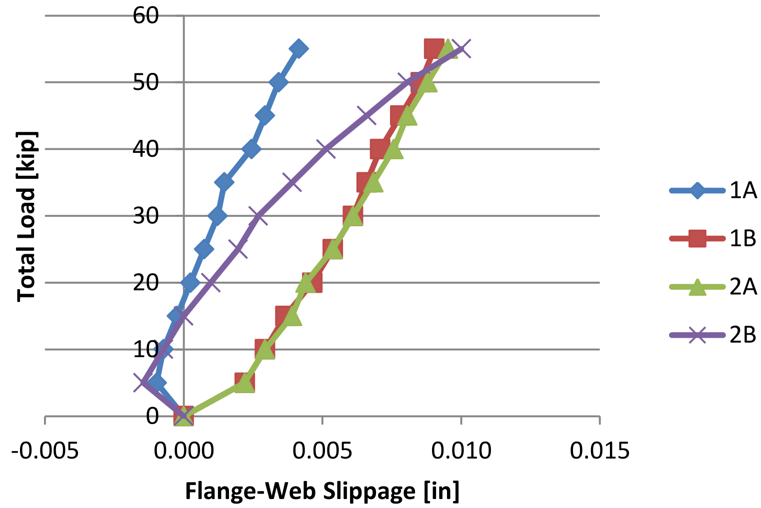

Figure 4.16: Specimen with Uneven Base

Because of these errors it was important to average the displacement of the four sensors. Also it was observed that the stiffer a connection was, the more pronounce these errors were. It was also thought that because of how the connectors attached to the wood, a slightly lower stiffness could have been obtained. This is because the LVDTs could have picked up wood deformation along with the connections deformation at the joint. Because wood deformations were accounted for in the modeling of a full-scale panel, these could have been doubly accounted for if the LVDTs were influenced by the woods deformation.

Most of the connectors had a bi-linear load-deformation behavior. A more complex model was not needed because usually there was a clear yield point and it was easier for comparison when all tests were condensed into one bi-linear model for each test. By best fitting the data with a bi-linear model, linear stiffness, non-linear stiffness, a yield point, and ultimate value was obtained. This was done for each specimen and the bi-linear models were averaged together to get a bi-linear model that represented the test series. These representative bi-linear models were used then for comparison between different tests. An example is shown in Figure 4.17.

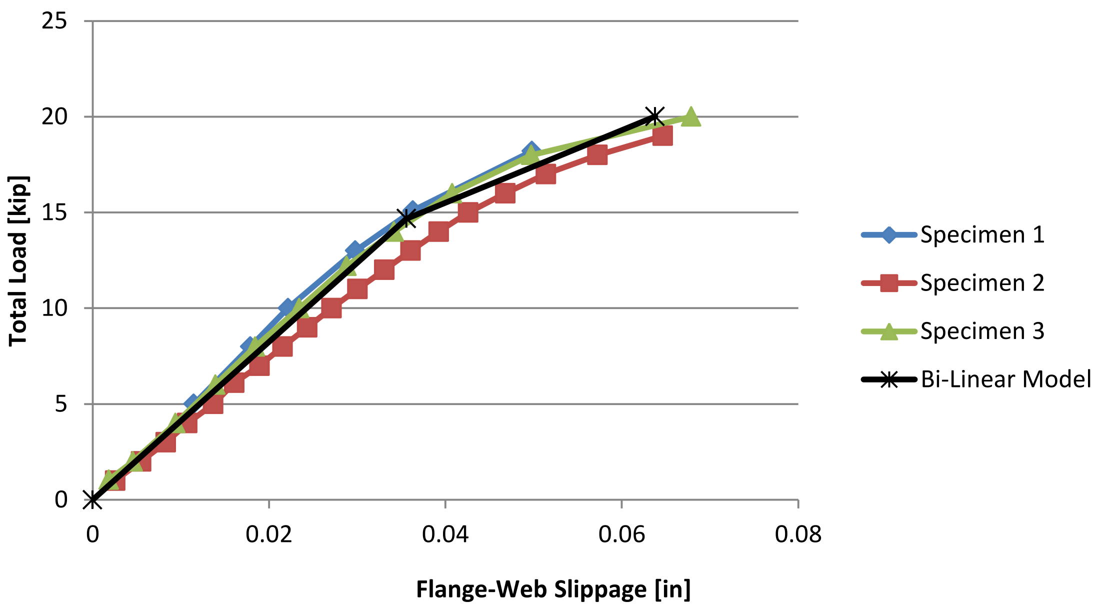

Figure 4.17: Actual Specimen Stiffness Curves Compared to Average Bi-Linear Model

<u>Design and Justification of Specimens</u>

A wide range of connection techniques was considered. In general, connections that were tested either fell into the category of glue connections or mechanical connections. A background into different connection techniques was given in the literature review in Chapter 2. In this section a description of each test and a reason for why it was tested will be given.

The screws used in the assembly of the panels are shown in Figure 4.18. From top to bottom they will be described. The first screw is an ASSY SK screw that was used to clamp the glue bond connections together. It has a wide head with the threads are in the main member and not in the side member to promote clamping action. The rest of the screws were used as inclined screws for a mechanical connection between the flange and the web of the panel. The second screw from the top is an ASSY VG Cylindrical screw that is $^{5}/_{16}$ inches in diameter by $11^{3}/_{4}$ inches in length. The third screw is an ASSY VG Cylindrical screw that is $^{3}/_{8}$ inches in diameter by $15^{3}/_{4}$ inches in length. The fourth screw is an SFS Intec WT-T that is $^{5}/_{16}$ inches in diameter by $11^{13}/_{16}$ inches in length.

Figure 4.18: ASSY and SFS Intec Screws Used for Testing

Test #1 used a normal CLT panel for the flange where the crosswise board was placed in between two layers that were parallel to the span. This test was chosen as a similar situation to what was done when making a hollow floor system in "City Academy" which was a school designed by Ramboll Engineering in the United Kingdom (G. White). It wanted to be seen if there would be a rolling shear failure inside the CLT panel because of the concentrated shear forces and what strength could be obtained when a glue bond clamped with screw pressure was used. Also, this test served as a comparison to tests that were run investigating a glue bond directly on the crosswise board. All glue bonds used for the flange to web connection were clamped with two clamping screws on each flange unless otherwise noted.

Because a planing machine that was used that was not calibrated correctly, the glulam bonding surface was not flat and resulted in a poor glue bond. For this reason and because two of the specimens in the first test failed in bending, this test was redone. In Test #1 redo, the glulam bonding surface was made sure to be flat and the steel plate with shims was used so that a bending failure would not occur. Test #1 Redo had a much higher quality glue bond that corresponded to the quality that would have been done by a manufacturer. The specimens used for both Test #1 and Test #1 Redo are shown in Figure 4.19.

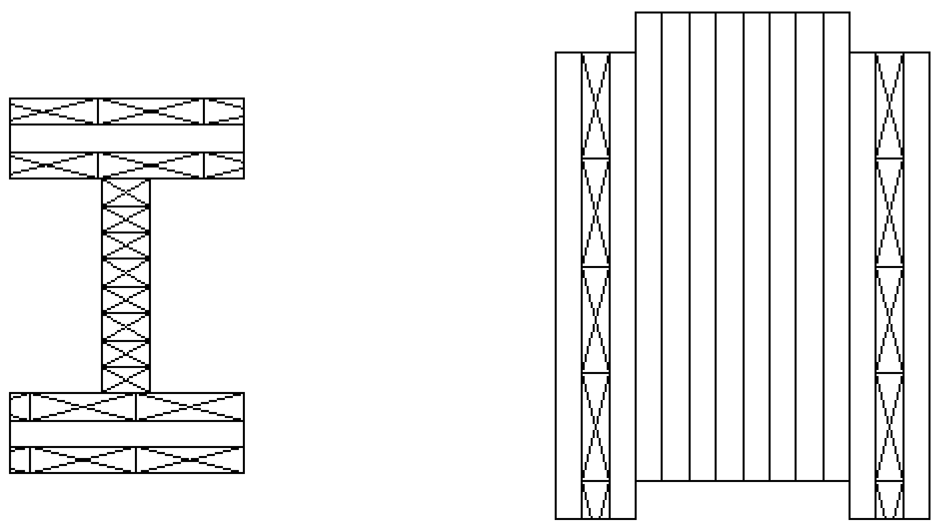

Figure 4.19: Test #1 & #1 Redo – Glue Bond with MF Using Normal CLT Flange

Test #2 used the flange configuration that was reported in Chapter 3. A negative aspect about this configuation is that this flange-to-web connection directly transfers a shear force along its length to the crosswise board in the flange in rolling shear. Unlike

Test #1 configuration, Test #2 has nothing to spread out the rolling shear stress so that more area of the crosswise board is effective. If enough strength could be obtained from this glue bond connection, a very stiff connection with high composite action would be obtained. The specimen used for Test #2 is shown in Figure 4.20.

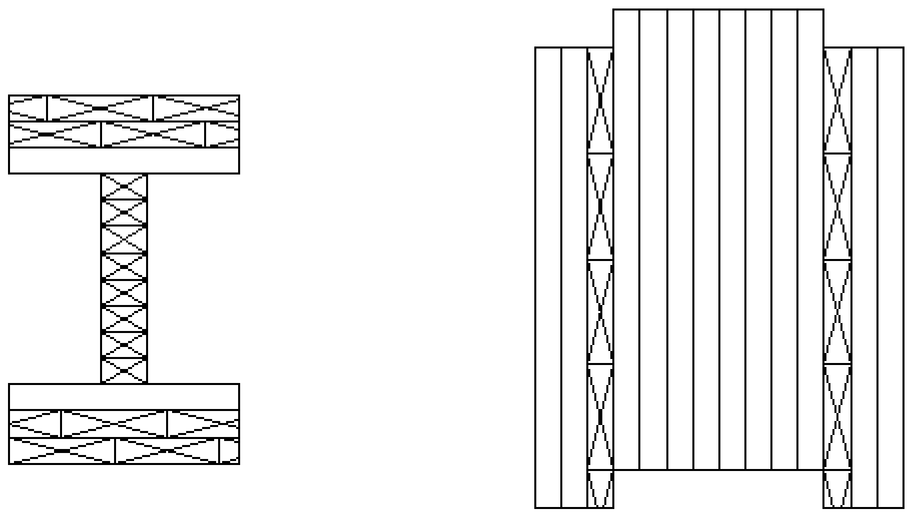

Figure 4.20: Test #2 – Glue Bond with MF in Rolling Shear Configuration

Test #2 Redo has 0.75-inch slits cut into the flange on each side of the glulam to force the transfer of shear force over a minimal width of the crosswise board to determine the low-end capacity of the connection. The specimen used for Test #2 Redo is shown in Figure 4.21.

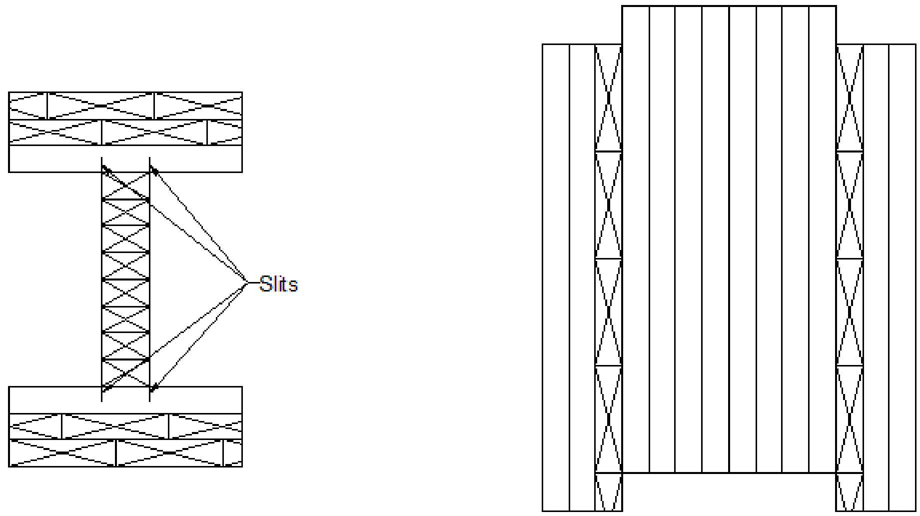

Figure 4.21: Test #2 Redo – Glue Bond with MF in Rolling Shear Configuration with Slits

Test #3 used four ASSY screws placed at a 45-degree angle down into the web at a spacing of 8 inches on-center. The screws were $^{5}/_{16}$ inches in diameter by $11^{3}/_{4}$ inches in length ASSY VG Cylindrical screws. It can be assumed that all screws were placed with half the length of the screw in the web and the other half in the flange. Also, all screw specimens were clamped together with hand clamps while screws were being placed to keep the joint tight since there wasn't much self-weight. One benefit of inclined screw connections is that there is not rolling shear issue, because the screw does not rely on the crosswise board to get the shear across to the parallel to span layers. A disadvantage of a screw connection is that it is less stiff than a glue connection and reduce the composite action of the built-up cross section. The specimen used for Test #3 is shown in Figure 4.22.

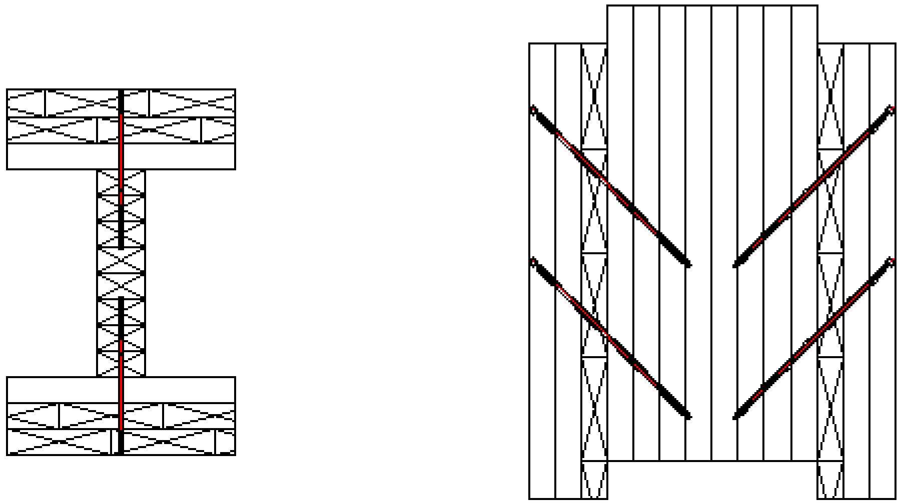

Figure 4.22: Test #3 – ASSY Screws at 45 Degrees

Test #4 used eight ASSY $^{5}/_{16}$ inches in diameter by $11^{3}/_{4}$ inches in length screws at 45 degrees installed at a 4-inch on-center spacing. This test doubled the amount of screws placed in the specimen while reducing the spacing to half the distances in comparison to Test #3. This was done to see if there were any group effects for this type of connection. It was important to see if the group effects either changed the stiffness or strength per screw. Also, issues like the potential splitting of the wood could be observed. The specimen used for Test #4 is shown in Figure. 4.23.

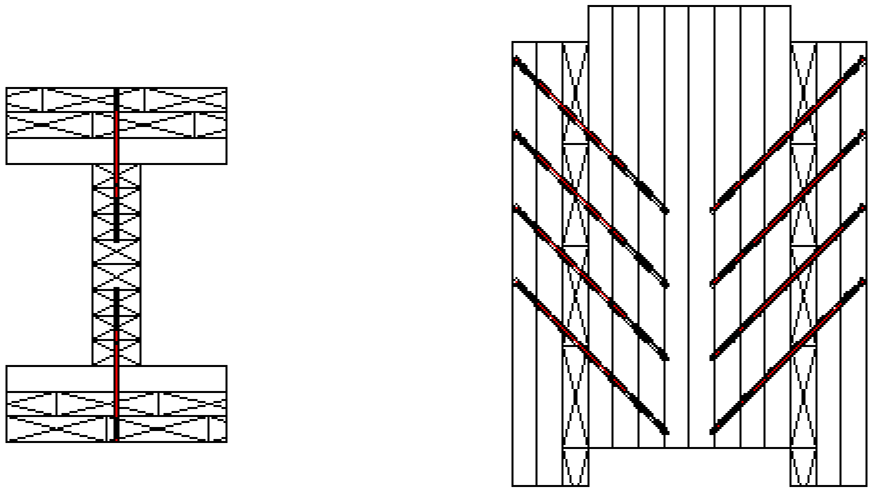

Figure 4.23: Test #4 – Group Test ASSY Screws at 45 Degrees

Test #5 used four ASSY $^3/_8$ inches in diameter by $15^3/_4$ inches in length screws at 30 degrees and spaced at 4 inches on-center This test was chosen because this was the largest screw that could be placed in a HMT panel with the 3-ply flange chosen and it wanted to be seen what were stiffness and strength results of this configuration compared to a the smaller screws placed at a 45-degree angle. Also, it wanted to be seen if a larger screw produced any problems with the thinner glulam that was chosen to be used. The specimen used for Test #5 is shown in Figure 4.24.

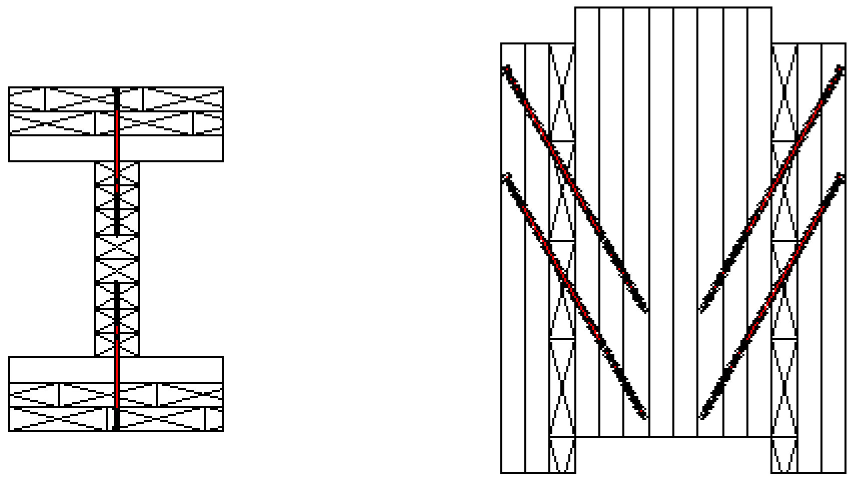

Figure 4.24: Test #5 – ASSY Large Screws at 30 Degrees

Test #6 is the exact same as Test #3 except that SFS Intec screws of the same size and diameter was used. This was chosen because the screws were very similar with a few distinctions and the difference in performance wanted to be investigated. These differences were:

1) SFS Intec screws have a smaller outside diameter of the screw head,
2) SFS Intec screws have a head that did not provide as much bearing surface for the screw bit,
3) SFS Intec screws have a thicker shaft with smaller threads to give the same outside diameter of the screw,
4) SFS Intec screws have a tighter more closely spaced thread pitch,
5) SFS Intec screws have a rougher coating,
6) SFS Intec screws are not fully threaded and have a slight change in pitch of the thread in the main member and side member to produce a clamping action without the need of a separate clamping screw, and
7) SFS Intec screws are less expensive.

From these differences, it was found that the reduction in bearing surface for the drill bit had the most important impact. This was because it was more common for this type screw to be stripped by the drill bit and then it was a major problem to get the screw out.

This was found to be a significant disadvantage because it could really slow down the construction process. The specimen used for Test #6 is shown in Figure 4.25.

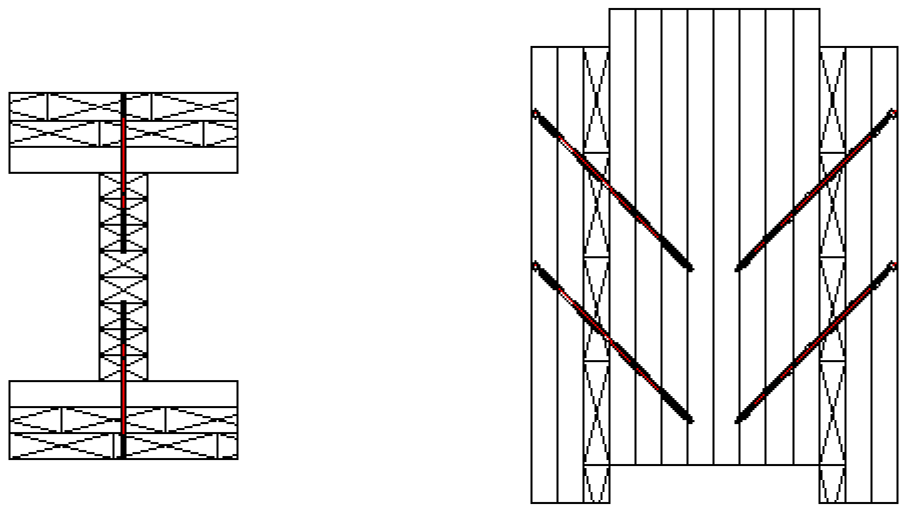

Figure 4.25: Test #6 – SFS Intec Screws at 45 Degrees

Test #7 uses SFS Intec screws that are placed at 30 degrees but are the same size as Test #6. This was chosen to see the specific difference in performance when the same screw is placed at a 30-degree angle instead of a 45-degree angle. Unlike Test #5, this allows for the angle to be varied without changing any other parameter. The specimen used for Test #7 is shown in Figure 4.26.

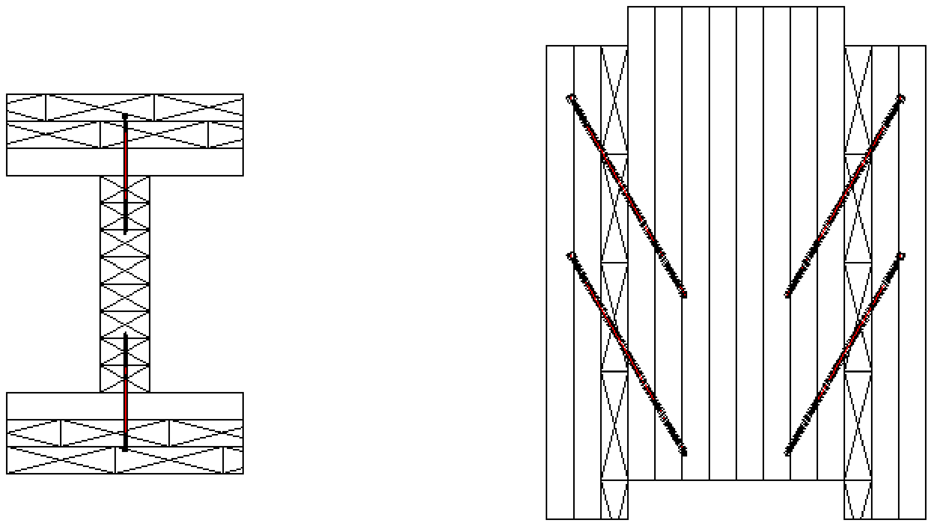

Figure 4.26: Test #7 – SFS Intec Screws at 30 Degrees

Test #8 uses the same SFS Intec screws as Tests #6 and #7 except they are placed at a 90-degree angle. This test was used as a comparison so that it could be seen the difference in a screw placed at a traditional 90-degree orientation compared to an inclined screw. The specimen used for Test #8 is shown in Figure 4.27.

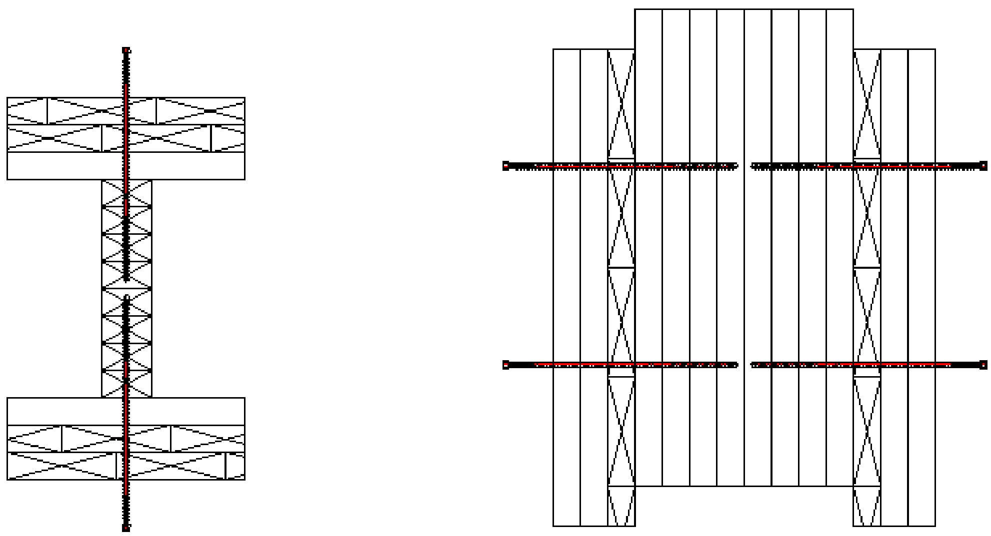

Figure 4.27: Test #8 – SFS Intec Screws at 90 Degrees

Test #9a has a ½-inch notch cut into the crosswise board of the flanges. This was chosen because this allowed the connection to have a greater surface area of glue bond and also allowed for the crosswise board to spread out the rolling shear stress more so that there was less of an effect on the performance of the connection. The specimen used for Test #9a is shown in Figure 4.28.

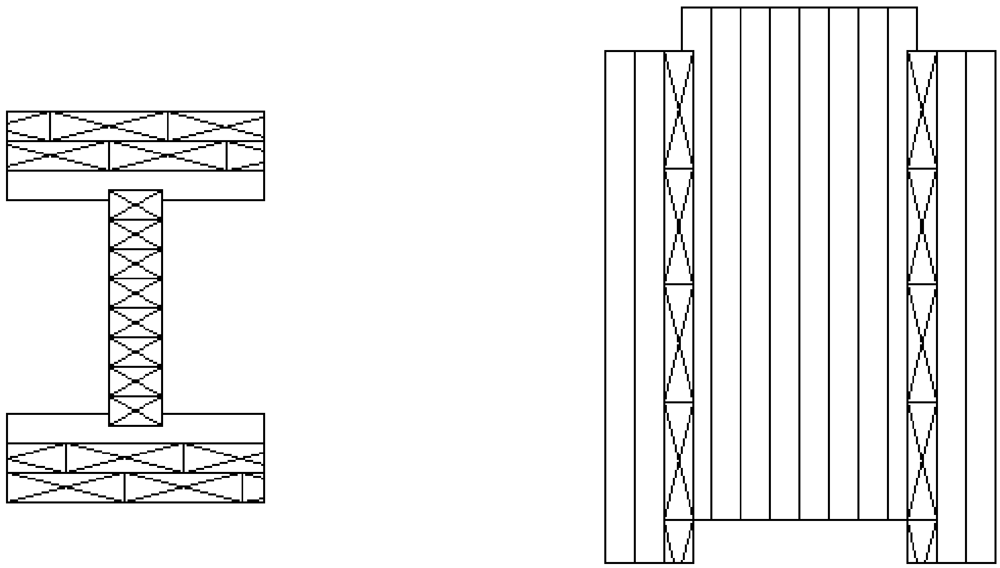

Figure 4.28: Test #9a – Glue Bond with MF in ½-inch Notch

Test #9b was the same at Test #9a except that the notch was made 1 inch deep to understand the effects of the depth of the notch. The depth of the notch was not made over 1 inch deep because some continuity of the crosswise board was important for holding the flange together, resisting gravity loads because the flange spans 32 inches from web to web, and it is important for transferring the load throughout the panel to make sure that all the boards parallel to the span can help resist bending stresses. The impact of the notch on the continuity was not tested and will be tested in the future if this connection type is chosen. The specimen used for Test #9b is shown in Figure 4.29.

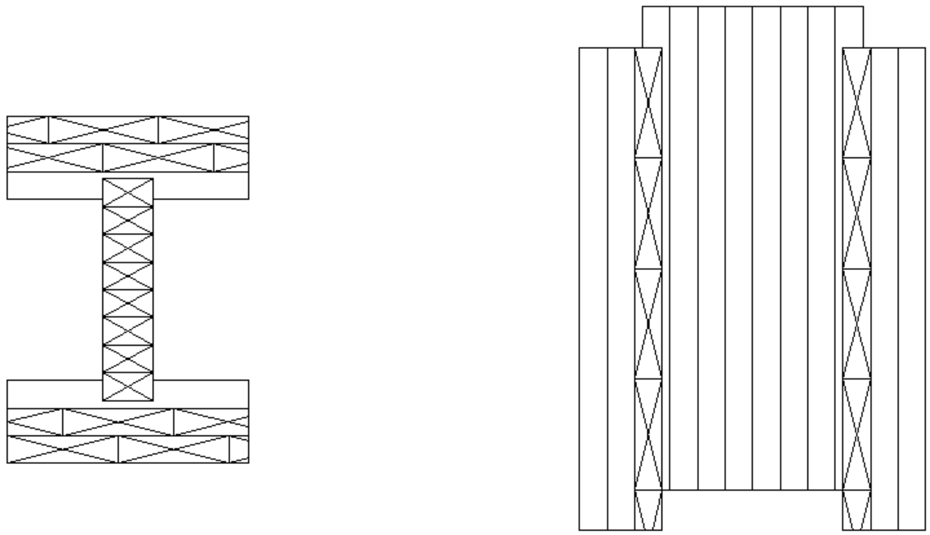

Figure 4.29: Test #9b – Glue Bond with MF in 1-inch Notch

Test #10a used a modified glulam that had a 2x6 for the extreme laminates of the glulam. This end board was pressed under full 150-psi pressure to the rest of the glulam, but the joint between the 2x6 and flange was clamped with screws. This was chosen because it was thought that this would help spread out the load so that the weaker crosswise board which is in rolling shear could resist the load better and not be the weak link of the connection. The specimen used for Test #10a is shown in Figure 4.30.

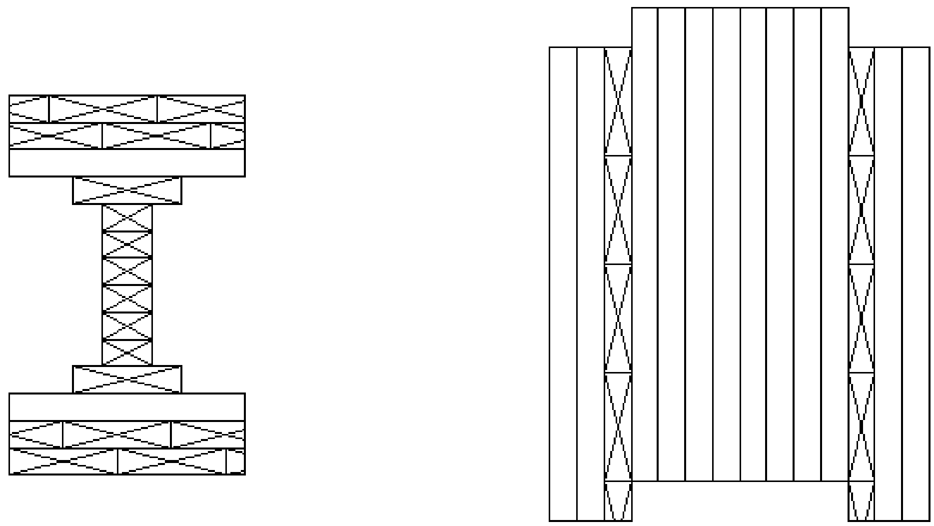

Figure 4.30: Test #10a – Glue Bond with MF with 2x6 Spreader Board

Test #10b is the same as test #10a except for the extreme laminates used a 2x8 piece of lumber instead of a 2x6 piece of lumber. It was unsure how much surface area was needed. The specimen used for Test #10b is shown in Figure 4.31.

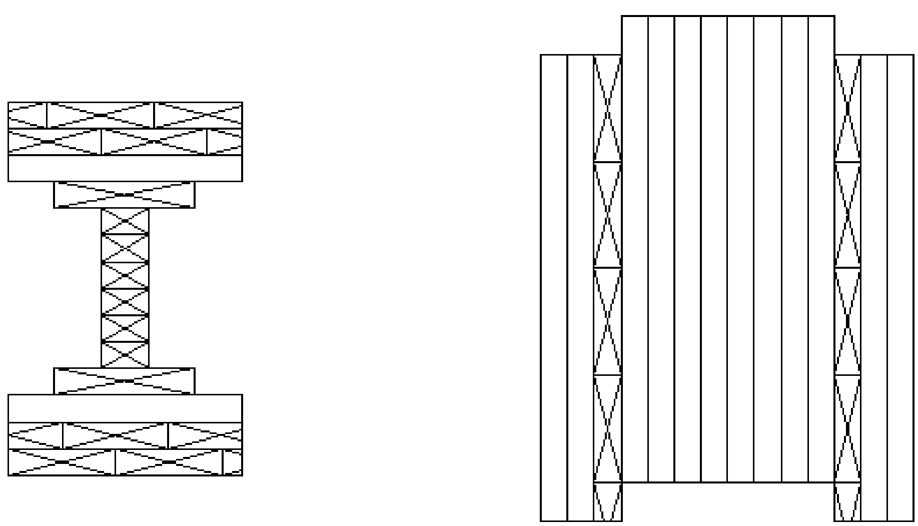

Figure 4.31: Test #10b – Glue Bond with MF with 2x8 Spreader Board

Test #11 used a combination of glue and screw connections. This test was a direct combination of Test #2 and Test #6. This was chosen because a glue and screw connector wanted to be studied to see if it had an additive effect or not. Also, there was a possibility of both connectors adding their beneficial properties to make a high performance connection. The specimen used for Test #11 is shown in Figure 4.32.

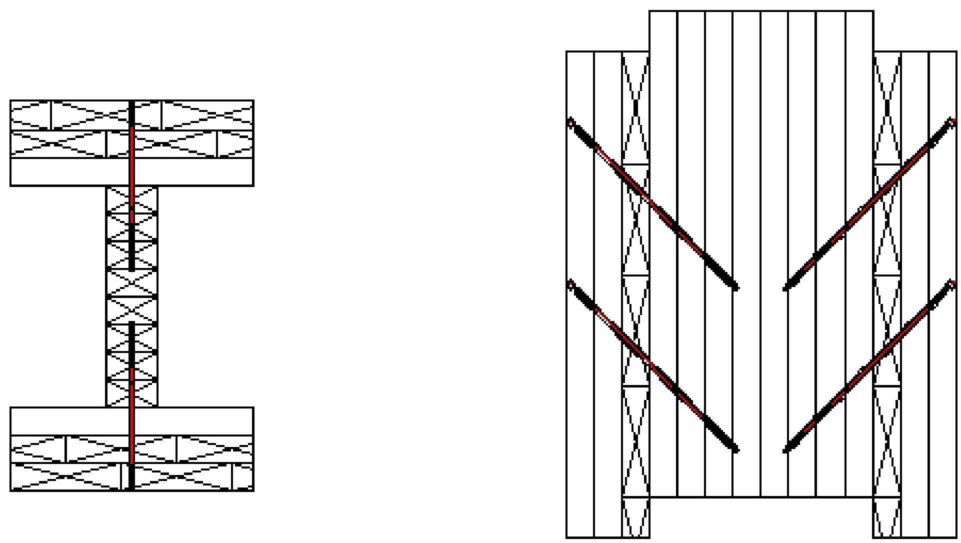

Figure 4.32: Test #11 – Glue Bond with MF with SFS Intec Screws at 45 Degrees

Test #12 used HBV that was 19½ inches in length and embedded 1-inch deep slots in both the web and the flange. The specific length was chosen based on what could fit in to the specimen and what was a typical length used for the HBV. An adhesive provided by the manufacturer of HBV was used to secure the HBV into the slots. Although the manufacturer recommended that the HBV expanded steel plate be embedded into a 1.575-inch deep slots, a 1-inch deep slots was chosen so the crosswise board was not completely cut. The specimen used for Test #12 is shown in Figure 4.33.

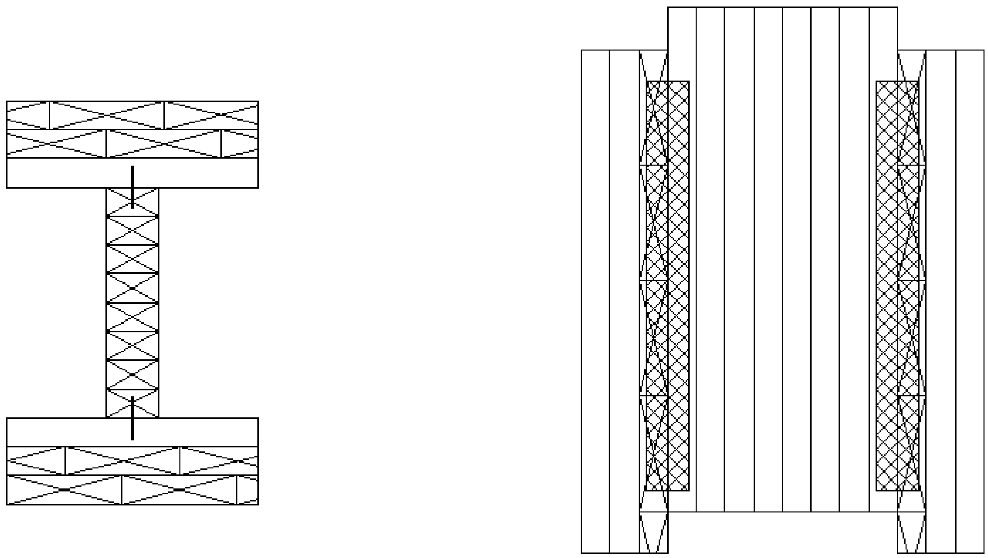

Figure 4.33: Test #12 – HBV 19½" Long without Rubber Membrane

Test #13 used SFS Intec screws at a 45-degree angle passing through a rubber membrane at the flange-web interface. The rubber membrane was $^{3}/_{16}$-inch thick QTscu and supplied by QT Sound Control. This rubber membrane is meant to be used as an underlayment for impact sound control in buildings. It is anticipated that the membrane will dampen out sound traveling through the floor system. Also, using this type of membrane could affect the vibrational properties of the panel and help dampen out floor vibrations. This could be a significant characteristic since the design of a HMT panel has been shown to be controlled by vibration. Since the membrane could affect the structural properties of the connector, this needed to be tested. The specimen used for Test #13 is shown in Figure 4.34.

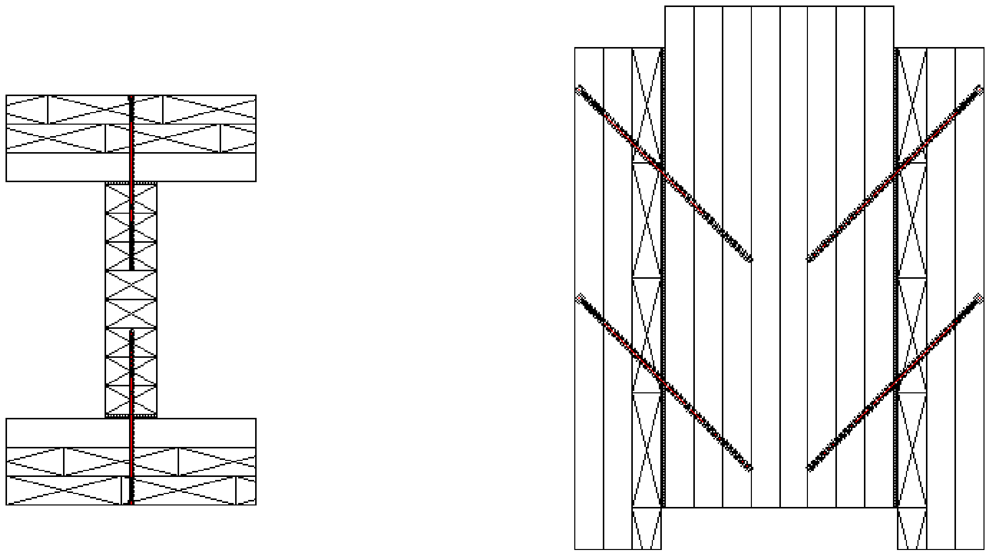

Figure 4.34: Test #13 – SFS Intec Screws at 45 Degrees Through Rubber Membrane

Test #14 was the same as Test #9a except EPI adhesive was used instead of MF. This was chosen because the difference in the performances of the two glues wanted to be seen. It was thought that the EPI glue might perform better because it foams when it cures and tends to fill gaps. This could help because there was no pressure on the side of the notch and because there was a small gap produced usually when making the notch so that the web would fit into it. The specimen used for Test #14 is shown in Figure 4.35.

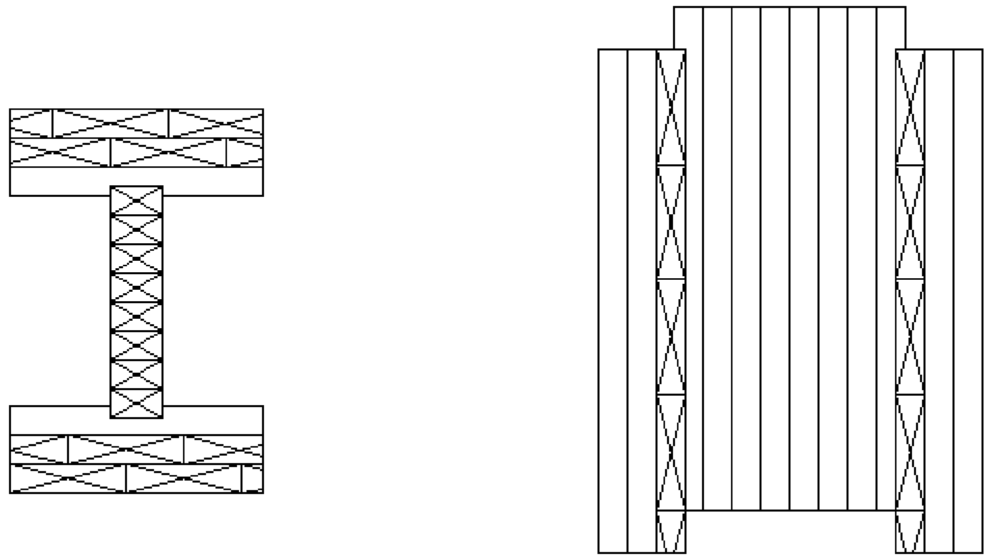

Figure 4.35: Test #14 – Glue Bond with EPI in ½-inch Notch

Test #15 used HBV that was 16" long and was imbedded into the wood 1-inch on each side. A smaller length was chosen for this test to insure that the connection was what failed and not the wood. Also, a rubber membrane was placed between the web and each flange. This was not because of acoustical reasons but because there needed to be a gap to allow for a more ductile connection along with their needing to be something that prevents the glue from directly bonding the flange to the web and provide an alternative load path. From results from Test #12, the HBV test without the membrane, it was seen that this separation was needed. It was assumed that the HBV would not have much of an acoustical difference when used with the membrane because there was still a rigid connection of the flange to the web. The specimen used for Test #15 is shown in Figure 4.36 and the making of the specimens is shown in Figure 4.37.

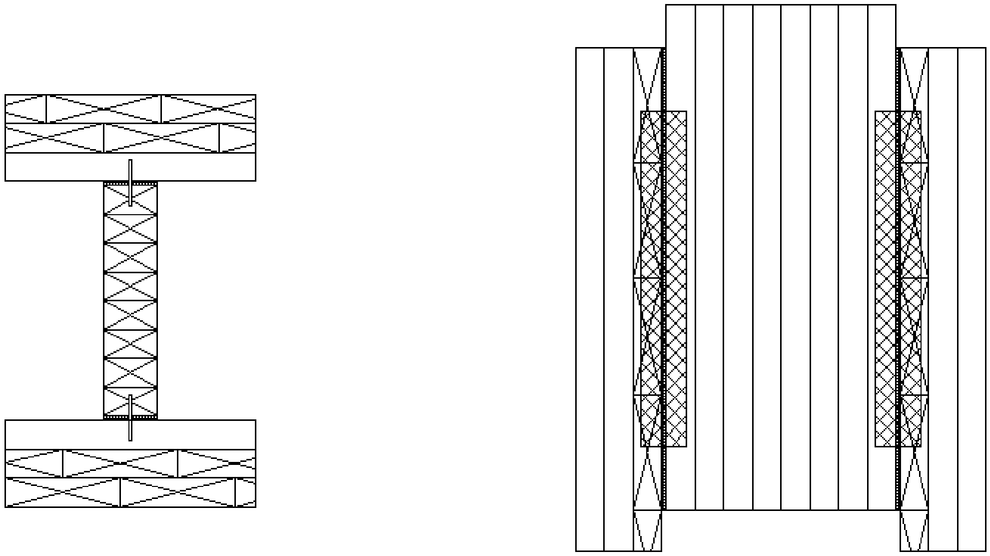

Figure 4.36: Test #15 – HBV 16” Long with Rubber Membrane

Figure 4.37: Gluing Together HBV Specimens

Test #16 used glass fiber reinforced polymer (GFRP) that had a strong direction and a weak direction. The strong direction was placed in tension by angling it down from the flange to the web at 45 degrees. In consultation with the engineering staff at Simpson Strong-Tie, the adhesive ETI-LV was suggested to be used. The length of the GFRP strip was 16 inches and the width allowed 1 inch of penetration into the flange and into the web. This test was meant to be able to compare to Test #15 with the HBV that was also 16 inches in length. When gluing the GFRP into the slits, one side of the connection was allowed to harden before the GFRP was embedded into the other side. This was to make sure that glue did not directly bond the flange to the web so that the test measured the connection performance of just the GFRP. The specimen used for Test #16 is shown in Figure 4.38 and the embedment of the GFRP fabric is shown in Figure 4.39.

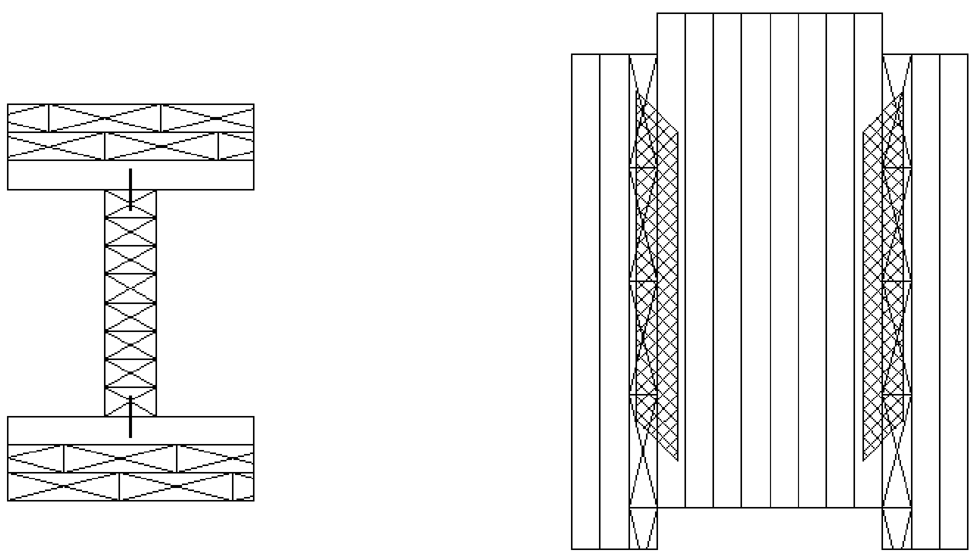

Figure 4.38: Test #16 – GFRP Fabric Glued on a 45-Degree Bias

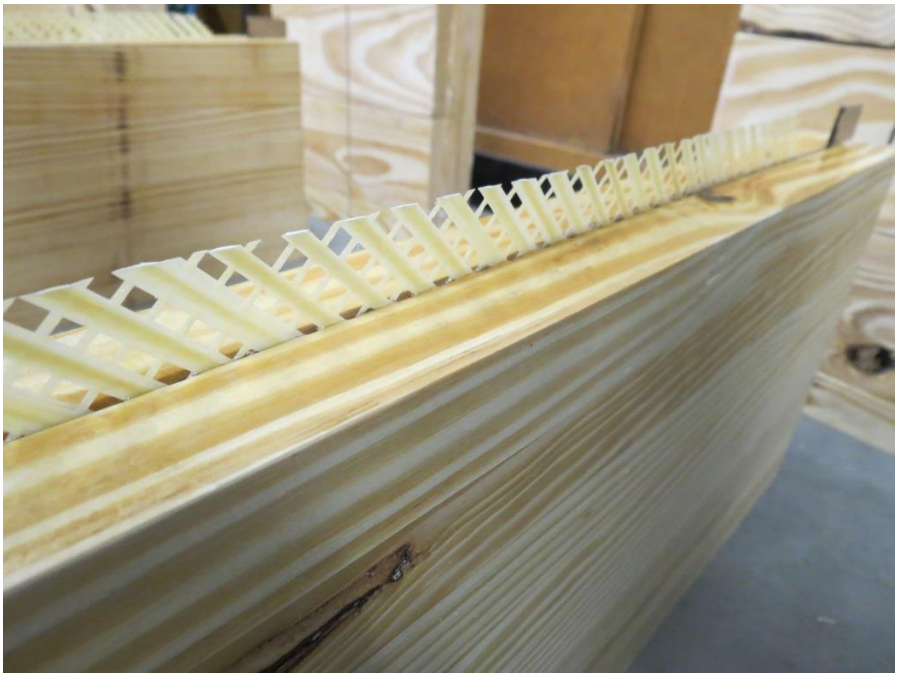

Figure 4.39: Test #16 – Embedding GFRP Fabric

Test #17 is the exact same test as Test #14 but used Grade #2 and #3 Southern Pine lumber instead of the 2400f_b MSR lumber. When the Grade #2/#3 lumber was used in any of the specimens, the Grade #2 lumber was used for the parallel to span layers of the flange, and it was used for all boards in the glulam web. The Grade #3 lumber was just used for the crosswise boards in the flange. Besides the MSR lumber having higher densities that could affect the connector strengths, the higher grade lumber used for the glulam should also increase the strength of the glulam. It was desired to have the connector strength to be higher than the glulam strength because it was thought that the glulam was more reliable, than the glue connection. The glulam had a higher quality glue bond while the glue connection proposed is just clamped together with screws, and therefore had a greater risk of not producing an adequate glue bond. If the connection is made consistently stronger than the glulam, then the design of the webs and their connection to the flange can be easily determined by the shear strength of the glulam. Therefore, this test uses Grade #2 and #3 lumber to see if this makes a difference in the glue bond and if it makes a difference in the strength of the glulam. The specimen used for Test #17 is shown in Figure 4.40.

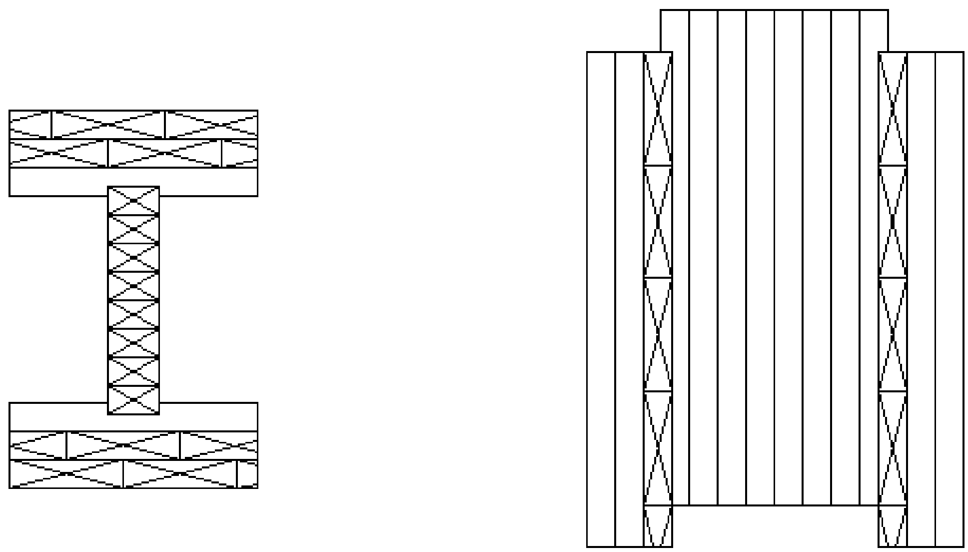

Figure 4.40: Test #17 – Glue Bond with EPI in ½"-inch Deep Notch with Grade #2 and #3 Lumber

Test #18 is similar to Test #13 except the screws that are used are ASSY instead of SFS Intec and they are placed at a 30-degree angle and only #3 lumber was used to make the specimen. Density of the wood could greatly affect the performance of the connector; therefore, it was imperative that this test be done using the correct grade of lumber that did not have high densities like the MSR lumber. In this case, the Grade #2 lumber was excluded and only Grade #3 lumber was chosen. This was done in order to be conservative and to allow for the possibility to have an all Grade #3 lumber flange. Also, because of results from other tests, the ASSY $^{5}/_{16}$ inches in diameter by 11¾ inches in length and 30-degree orientation was chosen. The specimen used for Test #18 is shown in Figure 4.41.

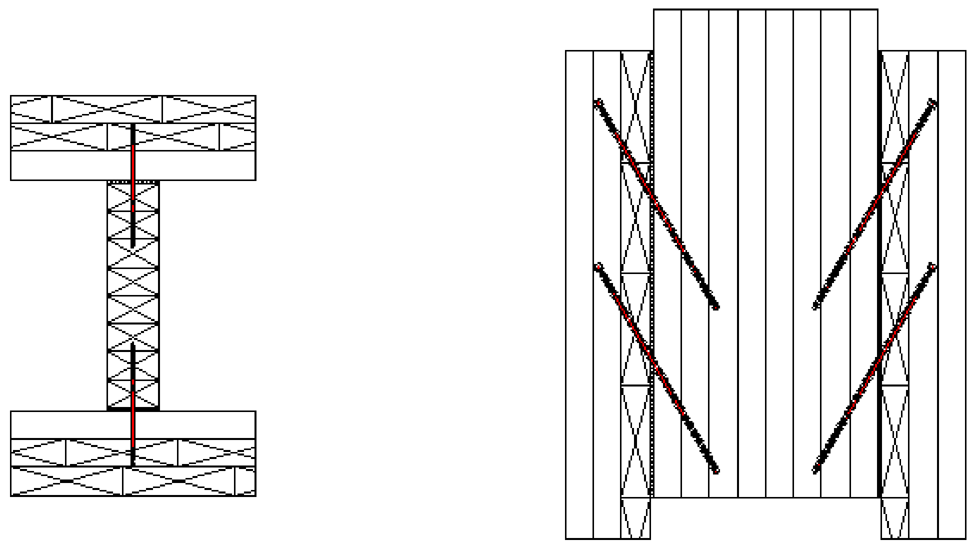

Figure 4.41: Test #18 – ASSY Screws at 30 Degrees Through Rubber Membrane

Test #19 used a flat glue bond the same as Test #2 except that a universal testing machine was used to apply 150 psi of contact pressure between the flange and web. Test #20 used the same flat bond except EPI adhesive was used and only screw pressure was used to clamp the joint. Both flanges and webs were made with Grade #2/#3 lumber, which is the grade that will be used for the final panel, in order to obtain more accurate results. It was believed that the different grade of lumber would affect the wood strength and since the failure of the wood at the connector is part of the cause for the connection failure, then a change in grade could affect the connection strength. The specimen used for Test #19 and Test #20 is shown in Figure 4.42.

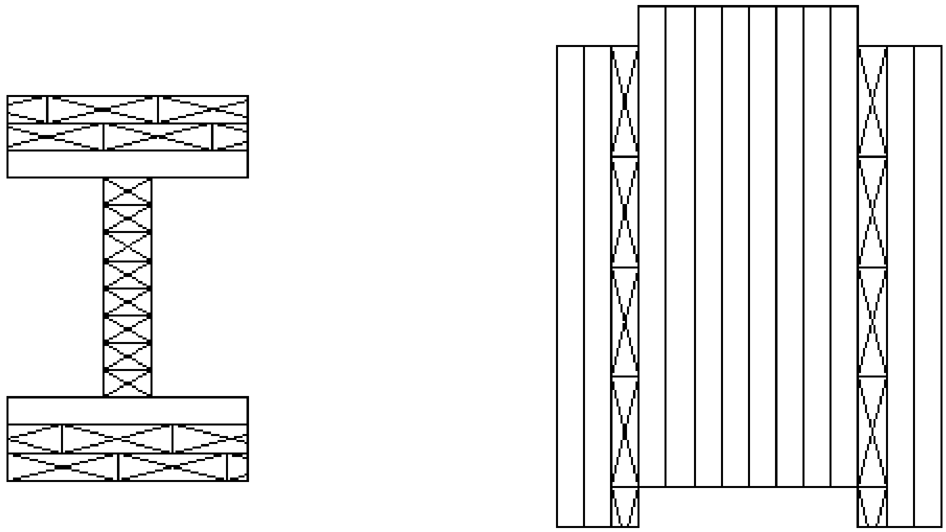

Figure 4.42: Test #19 – Glue Bond with MF Pressed with 150 psi Pressure and
Test #20 – Glue Bond with EPI Only Using Screw Pressure

Test #21 used Simpson Strong-Tie SDS with a diameter of $^{1}/_{4}$ of an inch and $2^{1}/_{2}$ inches in length to connect four 2½-inch x 2½-inch x ¼-inch angles that were 15½ inches in length. Each specimen had a different number of screws per angle. Specimen #1 had ten screws in each leg of the steel angle for a total of 80 screws in the full specimen. Specimen #2 had five screws in each leg of the steel angle for a total of 40 screws in the full specimen. Specimen #3 had 7 or 8 screws in each leg for a total of 60 screws in the full specimen. A continuous steel angle was chosen to be used instead of multiple angle brackets. The disadvantage of using short angle brackets is that the connectors also need to resist the overturning moments of the angle and by using a longer angle, the effects of overturning moments is reduced. In addition then, using a shorter leg length on the web connection would also tend to reduce the overturning moment. Also, all the screws were placed in a staggered arrangement so that the risk of a weak failure plane in the glulam would be minimized. In a line of screws for Specimen #1, the screws were spaced 3 inches apart, but since screws are coming into the web on the opposite side, there was in effect a screw shank every 1½ inches. Also, the first line of screws was placed 1 inch from the edge of the web and the second line of screws was placed 2 inches from the edge. This allowed for having reasonable edge distances. The specimen used for Test #21 is shown in Figure 4.43 and a picture of this connection technique is shown in Figure 4.44.

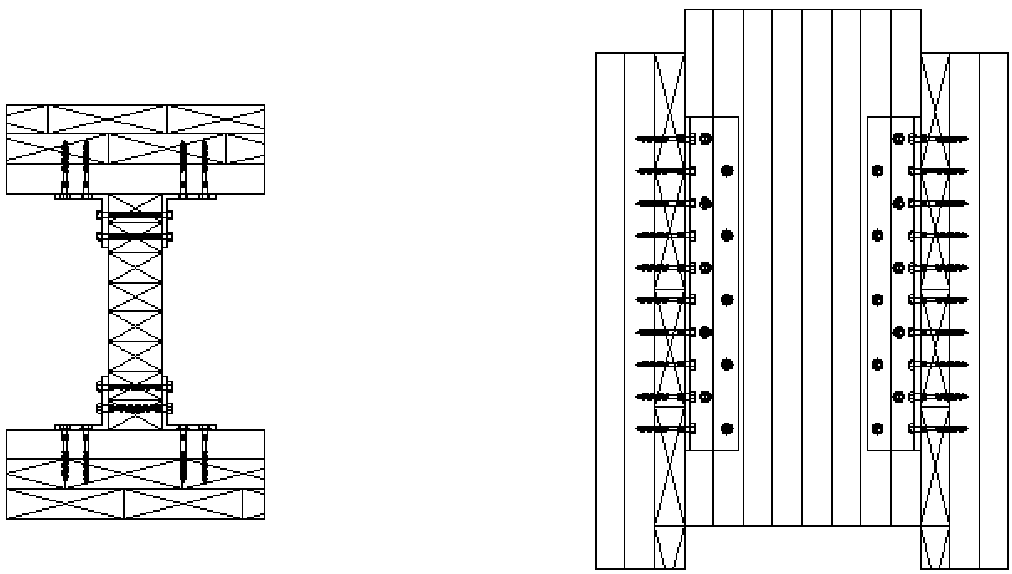

Figure 4.43: Test #21 – Simpson Strong-Tie SDS Screw with Steel Angle

Figure 4.44: Simpson Strong-Tie SDS Screws

Results and Discussion

This section will discuss the results obtained from all of the shear tests. All of the results for each test are shown in Tables 4.4 through 4.7. In order to more easily interpret the meanings of these values, the bi-linear graphs are shown and discussed later in this section. Many comparison graphs are made in order to more clearly show the meaning of the results so that conclusions can be drawn.

Table 4.4: Shear Test Results

Test Number	Test Description	Max Force [kip]	Normalized Strength[a]	Linear Stiffness [kip/in]	Non-Linear Stiffness [kip/in]	Yield Point [kip]	Normalized Linear Stiffness[b]	Failure Mode
Test 1 - 1	Normal CLT	59.09	537	7570	5716	34.76	172	Split because of Bending Stresses
Test 1 - 2		52.66	479	9595	7796	37.82	218	Split because of Bending Stresses
Test 1 - 3		59.20	538	5500	5113	59.96	125	Glue Joint on Side A
Test 1 - 4		54.58	496	7153	4427	30.63	163	Glue Joint on Side A
Test 1 (Avg)		56.38	513	7455	5763	41	169	
Test 1 Redo - 1	Test 1 Re-done	85.66	779	6015	4182	49.03	137	Glue Joint on Side B first then A
Test 1 Redo - 2		67.97	618	6869	4518	48.96	156	Glue Joint on Both Sides
Test 1 Redo - 3		69.67	633	5658	3909	40.08	129	Glulam on Side A
Test 1 Redo - 4		65.87	599	5743	3259	47.05	131	Glue Joint on Side A
Test 1 Redo (Avg)		72.29	657	6071	3967	46	138	
Test 2 – 1	Crosswise Board	50.17[c]	456[c]	no data	no data	no data	no data	Split because of Bending Stresses
Test 2 – 2		64.50	586	5508	1775	43.87	125	Glue Joint on Side A
Test 2 – 3		57.26	521	4239	999	47.47	96	Glue Joint on Both Sides
Test 2 – 4		57.03	518	3850	871	46.75	88	Glue Joint on Both Sides
Test 2 (Avg)		59.60	542	4532	1215	46.03	103	
Test 2 Redo - 1	Slits	34.08	310	2766	1513	19.53	63	Glue Joint on Side A
Test 2 Redo - 2		28.72	261	2638	1542	16.70	60	Rolling Shear Failure on Both Sides
Test 2 Redo - 3		35.58	323	2413	1170	23.85	55	Rolling Shear Failure on Both Sides
Test 2 Redo - 4		39.87	362	2700	1159	26.13	61	Rolling Shear Failure on Both Sides
Test 2 Redo		34.56	314	2629	1346	21.55	60	
Test 3 – 1	ASSY Screws at 45°	19.89	4.97	442	230	14.06	111	Rupture of Screw on Side A
Test 3 – 2		19.80	4.95	368	196	14.22	92	Rupture of Screw on Side A
Test 3 – 3		20.34	5.09	427	142	15.73	107	Rupture of Screw on Side A
Test 3 (Avg)		20.01	5.00	412	189	14.67	103	
Test 4 – 1	ASSY Screws at 45° Group Test	40.48	5.06	845	304	31.18	106	Rupture of Screw on Side B
Test 4 – 2		40.90	5.11	884	319	31.68	111	Rupture of Screw on Side A
Test 4 – 3		39.71	4.96	841	408	28.97	105	Rupture of Screw on Side B
Test 4 (Avg)		40.36	5.05	857	344	30.61	107	

Table 4.4 (cont.): Shear Test Results

Test Number	Test Description	Max Force [kip]	Normalized Strength[a]	Linear Stiffness [kip/in]	Non-Linear Stiffness [kip/in]	Yield Point [kip]	Normalized Linear Stiffness[b]	Failure Mode
Test 5 -1	Large ASSY Screws at 30°	36.49	9.12	539	141	33.54	135	Rupture of Screw on Side A
Test 5 -2		35.48	8.87	639	276	29.54	160	Rupture of Screw on Side B
Test 5 -3		35.76	8.94	430	218	31.91	108	Rupture of Screw on Both Sides
Test 5 (Avg)		35.91	8.98	536	212	31.66	134	
Test 6 -1	SFS Intec Screws at 45°	21.96	5.49	374	129	15.96	94	Rupture of Screw on Side B
Test 6 -2		23.12	5.78	336	181	17.12	84	Rupture of Screw on Both Sides
Test 6 -3		24.00	6.00	448	202	16.39	112	Rupture of Screw on Both Sides
Test 6 (Avg)		23.03	5.76	386	171	16.49	97	
Test 7 – 1	SFS Intec Screws at 30°	24.62	6.16	456	170	20.75	114	Rupture of Screw on Side B
Test 7 – 2		21.96	5.49	615	350	18.17	154	Rupture of Screw on Side A
Test 7 – 3		20.76	5.19	577	368	17.44	144	Rupture of Screw on Side A
Test 7 – 4		24.26	6.07	496	101	20.74	124	Rupture of Screw on Both Sides
Test 7 (Avg)		22.90	5.73	536	247	19.28	134	
Test 8 – 1	SFS Intec Screws at 90°	14.91	3.73	1969	16	4.61	492	Rupture of Screw on Side B
Test 8 – 2		15.96	3.99	490	17	4.68	123	Rupture of Screw on Side B
Test 8 – 3		16.35	4.09	86	19	1.48	22	Rupture of Screw on Side A
Test 8 (Avg)		15.74	3.93	848	17	3.59	212	
Test 9a – 1	MF $^{1}/_{2}$" Notch	64.41	418	6805	4883	43.68	155	Glue joint on Side B, Glulam on A
Test 9a – 2		94.73	615	7868	5686	60.56	179	Glue joint on Side B, Glulam on A
Test 9a – 5		75.12	488	9892	8435	64.77	225	Glulam on Side A
Test 9a – 6		57.82	375	13685	9528	35.17	311	Glue Joint on Side A
Test 9a (Avg)		73.02	517	9563	7133	51.05	167	
Test 9b – 3	MF 1" Notch	63.84	322	8625	3454	42.30	196	Glue joint on Side A, Glulam on B
Test 9b - 4		115.18	582	14645	8019	94.70	333	Glulam on Both Sides
Test 9b (Avg)		89.51	452	11635	5737	68.50	264	
Test 10a – 1	2x6 Spreader	66.91	276	9652	4348	48.98	219	Glue joint on Side A
Test 10a – 2		60.80	251	8282	3243	44.50	188	Glue joint on Side A
Test 10a (Avg)		63.86	264	8967	3796	46.74	204	

Table 4.4 (cont.): Shear Test Results

Test Number	Test Description	Max Force [kip]	Normalized Strength[a]	Linear Stiffness [kip/in]	Non-Linear Stiffness [kip/in]	Yield Point [kip]	Normalized Linear Stiffness[b]	Failure Mode
Test 10b – 3	2x8 Spreader	65.80	206	14964	not enough data	not enough data	340	Glue joint on Side A
Test 10b – 4		80.97	254	38972	24932	60.65	886	Glulam to Glue joint on Side B
Test 10b (Avg)		73.39	230	26968	24932	60.65	613	
Test 11 – 1	Screw and Glue	59.14	538	4402	1379	27.35	100	Glue Joint on Side A
Test 11 – 2		70.73	643	4407	2224	39.39	100	Glue Joint first in Side A then B
Test 11 – 3		58.18	529	4139	1607	40.28	94	Glue Joint on Side B
Test 11 (Avg)		62.68	570	4316	1737	35.67	98	
Test 12 – 1	HBV with No Gap	68.66	624	7826	6483	43.13	201	Glulam on Side A
Test 12 – 2		75.27	684	8098	6019	44.65	208	Glulam on Side B
Test 12 – 3		102.95	936	9785	5067	64.40	251	Glulam on Side B
Test 12 (Avg)		82.29	748	8570	5856	50.73	220	
Test 13 – 1	SFS Intec Screws Membrane at 45^{o}	22.78	5.70	235	156	17.40	59	Rupture of Screw on Side B
Test 13 – 2		25.28	6.32	240	100	21.37	60	Rupture of Screw on Side A
Test 13 – 3		25.62	6.41	262	85	18.55	66	Rupture of Screw on Side B
Test 13 (Avg)		24.56	6.14	246	114	19.11	61	
Test 14 – 1	EPI $^{1}/_{2}$" Notch	74.56	484	10719	7660	45.84	244	Glulam on Side A
Test 14 – 2		84.39	548	9306	4258	42.33	212	Glue Joint on Both Sides
Test 14 – 3		92.31	599	9223	5154	65.38	210	Glue Joint on Both Sides
Test 14 (Avg)		83.75	544	9749	5691	51.18	222	
Test 15 – 1	HBV with Gap	33.93	1.06	3007	175	31.64	94	Steel Sheared on Side B
Test 15 – 2		36.46	1.14	3912	55	32.94	122	Steel Sheared on Side A
Test 15 – 3		35.58	1.11	3973	39	33.38	124	Steel Sheared on Side A
Test 15 (Avg)		35.32	1.10	3631	90	32.65	113	
Test 16 – 1	GFRP	45.32	1.42	2811	1446	32.90	88	GFRP on Side B
Test 16 – 2		43.17	1.35	2499	903	32.60	78	GFRP on Side B
Test 16 – 3		43.09	1.35	2672	787	33.71	83	GFRP on Both Sides
Test 16 (Avg)		43.86	1.37	2660	1045	33.07	83	
Test 17 – 1	EPI $^{1}/_{2}$" Notch with #2/#3	44.29	369	8478	2932	38.72	265	Glulam Side B
Test 17 – 2		66.87	557	9732	2238	58.34	304	Glulam Side A
Test 17 – 3		58.71	489	8237	2294	50.34	257	Glulam Side A
Test 17 (Avg)		56.62	472	8816	2488	49.13	275	

Table 4.4 (cont.): Shear Test Results

Test Number	Test Description	Max Force [kip]	Normalized Strength[a]	Linear Stiffness [kip/in]	Non-Linear Stiffness [kip/in]	Yield Point [kip]	Normalized Linear Stiffness[b]	Failure Mode
Test 18 – 1	ASSY Screw Membrane at 30° with #2/#3	24.38	6.09	403	144	19.80	101	Rupture of Screw on Side A
Test 18 – 2		23.31	5.83	355	94	20.69	89	Rupture of Screw on Side A
Test 18 – 3		21.88	5.47	334	89	20.98	83	Rupture of Screw on Side B
Test 18 (Avg)		23.19	5.80	364	109	20.49	91	
Test 19 – 1	MF Pressed at 150 psi	53.50	486	6008	2610	47.85	188	Glue Joint on Side B
Test 19 – 2		58.90	535	6163	2107	49.27	193	Glue Joint on Both Sides
Test 19 – 3		47.88	435	6226	2304	40.17	195	Glue Joint on Side A
Test 19 (Avg)		53.42	486	6132	2340	45.76	192	
Test 20 – 1	EPI Flat Bond using Screw Pressure with #2/#3	52.36	476	4840	1635	44.39	151	Glue Joint on Side B
Test 20 – 2		63.35	576	4799	1619	54.86	150	Glue Joint on Both Sides
Test 20 – 3		49.16	447	6488	2129	43.58	203	Glue Joint on Side B
Test 20 (Avg)		54.96	500	5376	1795	47.61	168	
Test 21 – 1	Simpson SDS Screw with Steel Angles	58.80	1.47	519	98	42.75	17	Glulam on Side B
Test 21 – 2		29.95	1.50	186	9	25.58	6	Never Failed
Test 21 – 3		43.12	1.44	263	17	35.70	8	Never Failed
Test 21 (Avg)		43.96	1.47	323	41	34.68	10	

[a]Strength per Screw [kip/screw]/ Strength per Length of Connector [kip/in] / Shear Strength of Glue Bond or Wood [psi]
This normalized force was always given with respect to the connection being used, and never in relation to the glulam stress
[b]Stiffness per Screw [kip/in] or per Inch of Glue [kip/in/in]
[c]Shaded cells mean that these values were not used in the average

These results presented in Tables 4.4 through 4.7 cannot all be compared by simply comparing maximum force to maximum force or linear stiffness to linear stiffness because each test was not made so that they all should perform equally. Even the normalized forces cannot be compared between all of the specimens because there are significant differences that must be accounted for. Therefore, specific comparisons will be made by comparing the bi-linear graphs given below. A file of the shear test results with additional comments is archived in the spreadsheet titled “Shear Test Summary” and a file with all stiffness plots for each LVDT for each specimen along with all bi-linear model graphs is archived in the spreadsheet titled "Shear Test Summary Graphs" (see Appendix).

In Figure 4.45, Test #1 Redo shows significant increase over Test #1. This is because some of the specimens in Test #1 failed prematurely because of splitting of the web member due to bending stresses and because there was a poor glue bond due to not having a flat surface on the edge of the glulam web. Therefore, Test #1 can be ignored and just the rest of the tests in Figure 4.45 can be compared.

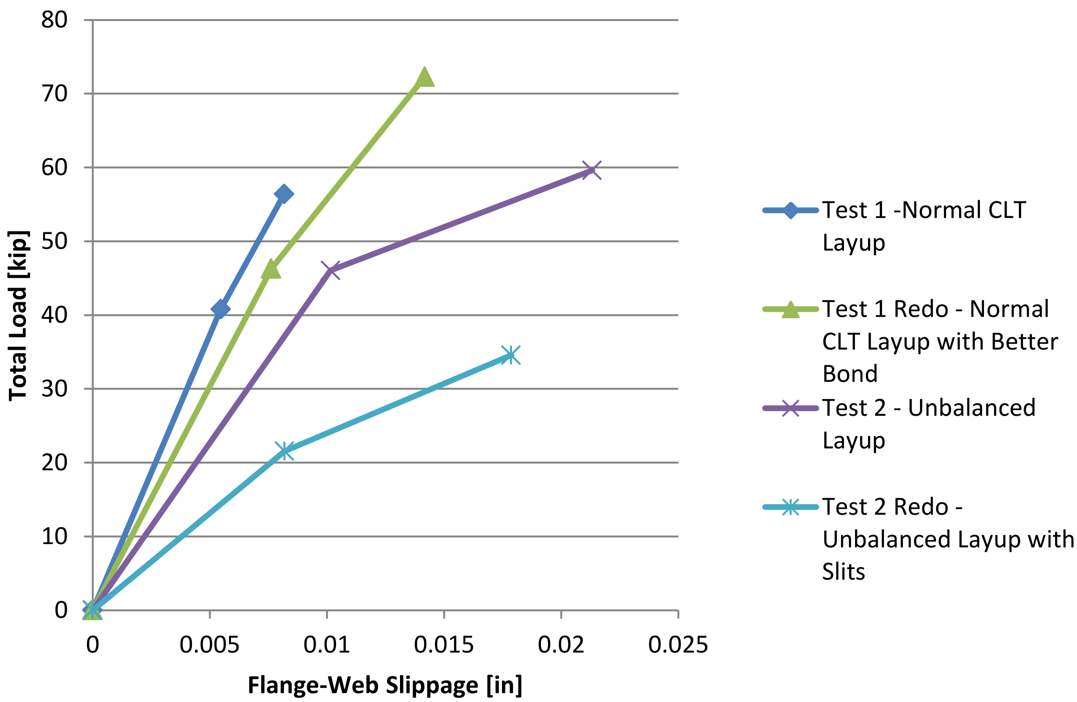

Figure 4.45: Comparison Between Panel Layups

All glue tests were found to have a brittle failure mode as expected, but some had a more non-linear behavior. This non-linear behavior can be specifically seen in Test #2.

It was believed that this behavior was due to the beginning of the wood in the crosswise board to “roll” like in a rolling shear failure. Results from Test #2 seemed to be higher than expected for a rolling shear failure, so it was decided to do Test #2 Redo that had slits cut on either side of the glulam into the crosswise board of the flange. This was to see if the higher capacity of Test #2 was from the effects from having the concentrated shear force spread out by crosswise board. Another reason that would point to a rapid spreading out of the shear force was because it was concluded that there was only a shallow rolling shear failure rather than a deep rolling shear failure where usually the whole crosswise board “rolls”. From the results of Test #2 Redo, it was found that the slits did cause a deep rolling shear failure to occur which in turn significantly lowered the ultimate shear force obtained. Pictures of these failures can be seen in Figure 4.46.

Figure 4.46: Examples of Shallow and Deep Rolling Shear Failure Modes

From these tests it was concluded that placing the crosswise board next to the connection to the web causes a much smaller reduction in shear strength than previously thought. By comparing the average ultimate values for Test #1 Redo to Test #2, there was only a drop of 17.6% whereas there was a drop of 52.2% when comparing Test #1 Redo to Test #2 Redo. Also, Test #1 Redo was much stiffer than predicted which is good because it helps produce a better composite panel. Both higher strength and stiffness gains could be attributed to the ability for the crosswise board to quickly spread out the load so that more of the wood could act to resist the weaker and less stiff deep rolling shear failure. Since Test #1 Redo uses a normal CLT flange layup, it is not desired to be used with a HMT panel, but if a greater amount of strength is required, then it may be considered.

Like what is seen in Figure 4.47, all screws tested throughout this research showed a non-linear behavior due to a short period of yielding and then a brittle rupture. Also, all screws tested failed in the screw by a rupturing of the screw. Test #3 resulted in an average of 5 kips of shear force per screw and stiffness of 103 kip/in per screw. From the results of the group test that had double the number of screws at half the spacing, it was seen that there was no significant reduction in strength or stiffness per screw. Because of this, it was assumed that the screws stiffness and strength could be additive with no significant group effects at a spacing of greater than 4 inches and with an edge distance of at least 1.25 inches.

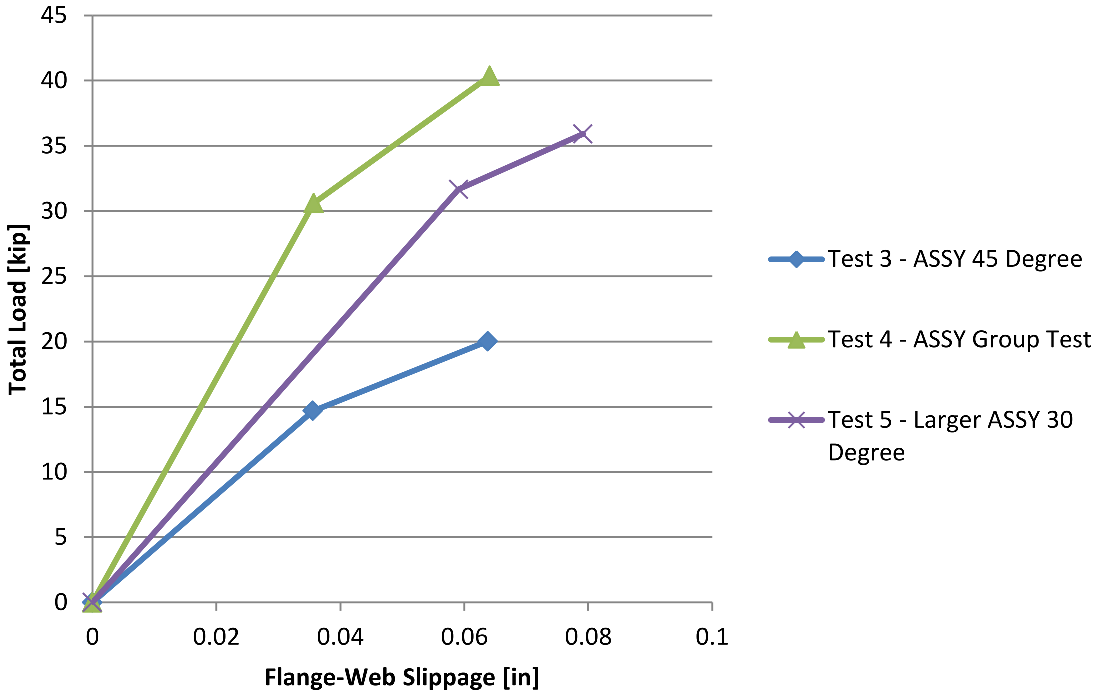

Figure 4.47: Comparison Between ASSY Screws

Also, from Specimen #3 of Test #5, the robustness of the inclined screws was seen. In this test one of the large ASSY screws was place improperly and the screw tip protruded out of the glulam. It was left installed improperly to see the outcome. The results from this specimen were that there was no reduction of strength when compared to the average for that test. Moreover, the side that failed first was the screws on the opposite side of the specimen. Also, throughout the experiments, it was observed that even if the screw was closer to the edge of the glulam, there was no reduced strength along with no observed splitting of the wood. One concern when choosing a thinner glulam for the web was that there might be a higher risk of splitting the glulam. This wasn't a concern after seeing how the screws showed no tendency to split the wood even if it was close to the edge and coming out of the side of the glulam.

The difference in Test #5 and Test #3 was 79.5% increase in strength while having only a 30.1% increase in stiffness. Test #5 had a larger variation in stiffness, and this could be due to the gaps between the flange and the web because of not adequate clamping. Because of this, Test #5 could have a higher true stiffness than the results obtained. Since the larger screw had a smaller stiffness gain than strength gain, it might not be as beneficial for the flange to web connection because a very stiff joint is needed. If stronger screws are used, then fewer screws will be needed and if the strength and stiffness gain aren't equal then an overall smaller stiffness will be obtained. This might not affect the overall performance of the panel much, but would certainly not help. These results were used for a more detailed analysis that is presented in Chapter 5 that compares different screw types cost and performance of different screw types.

Tests #6 through #8, shown in Figure 4.48, were primarily done to compare the stiffnesses and the strengths for self-tapping screws placed at different angles. When comparing Test #7 to Test #6, there was no strength gain and a 38.1% increase in stiffness. According to Equation 2.1, there should be a 14.3% increase in strength when a screw is moved from 45 degrees to 30 degrees. This is probably not seen because of possible error due to difficulties in driving the screws in at 30 degrees. It was found that it was much harder to put in the 30-degree screws, but if a pilot hole was used, then it was found to be much easier to place screws at the correct angle and depth. Since there are stiffness and potential strength benefits by using the same screw just at different orientations then it seemed beneficial to use screws at 30 degrees since theoretically there is no added cost. When comparing the two inclined screw tests to the 90-degree test, it was observed that the 90-degree test was much less stiff. This was as predicted and because of this lack of stiffness; a HMT panel would lose most of its composite action.

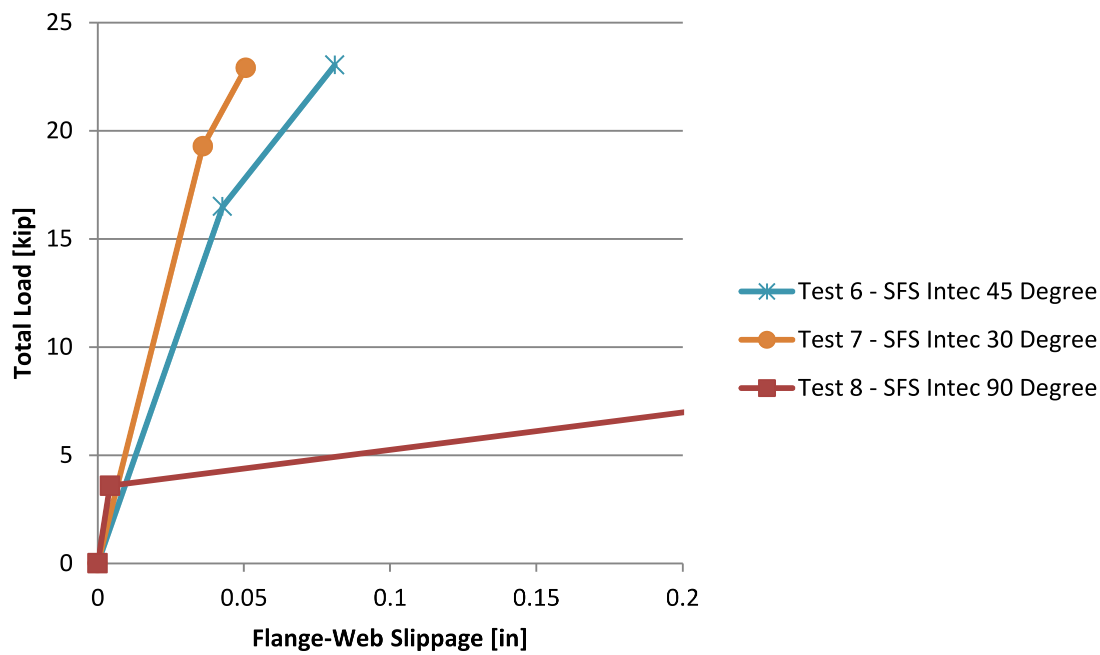

Figure 4.48: Comparison of Screws at Varying Angles

Both brands of screws were governed by the steel rupturing. Since SFS Intec had a thicker shaft, these screws broke at higher levels. To be more precise, the SFS Intec screws usually broke not right at the joint. This was because the diameter of the screw was larger right in the middle and reduced to a normal size where the threads were. Therefore, it always broke away from the joint so that where the diameter was less thick. Since all screws ruptured in the steel, it could not be concluded which screw would perform better if withdrawal was the governing failure mode. Examples of these failure modes can be seen in Figures 4.49 and 4.50.

Figure 4.49: Typical Screw Failure Modes (ASSY on Right and SFS Intec on Left)

Figure 4.50: Typical Rupture of Screw (ASSY on Right and SFS Intec on Left)

These two screw types were compared in Figure 4.51 and the SFS Intec screws were found to be slightly less stiff and this could have been because of the introduction of gaps at the connection. These gaps were believed to be caused by the joint having differential movement due to the SFS Intec screws clamping the flange to the web unevenly. This was caused because one screw was installed and then when a second screw was installed the first screw didn't want to move and therefore would get pried up and would cause a gap. This could be different if a longer specimen was used or if other clamping screws were used to hold the flange tight to the web or if all screws were placed simultaneously. Overall there wasn't that much difference and the choice between the two would most likely come down to cost and the ability to drive the screws into the wood.

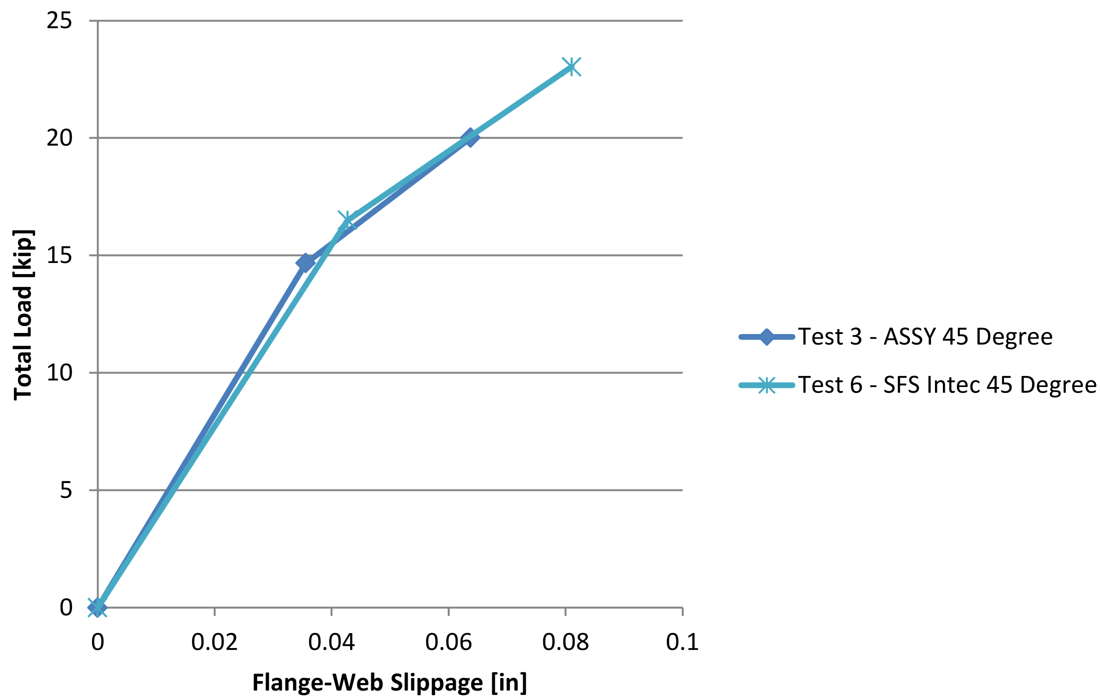

Figure 4.51: Direct Comparison Between ASSY and SFS Intec at 45 Degrees

In Figure 4.52, modified glue tests are compared. In the next paragraphs, these different tests will be discussed. With regard to the spreader board test, there were some errors introduced that highlighted some constructability issues. One issue was the flanges were glued onto the web significantly after the panels were made while not having the panels planed or smoothed in some way. As the crosswise boards dried, they bent into a cup shape that did not allow the web to sit flat on the crosswise layer. This ended up producing gaps and a bad glue bond when clamped with just screws. The second issue was that when the glulams were pressed, they were not braced well because of their unique configuration and the laminations turned out to be skewed rather than all aligned. The third issue was that the 2x8 spreader board was a Grade #2 Southern Pine, while the rest of the lumber was the MSR lumber. Even if the spreader board tests had greater strengths, it was concluded that it would be easier to obtain the same strengths with a notch connection.

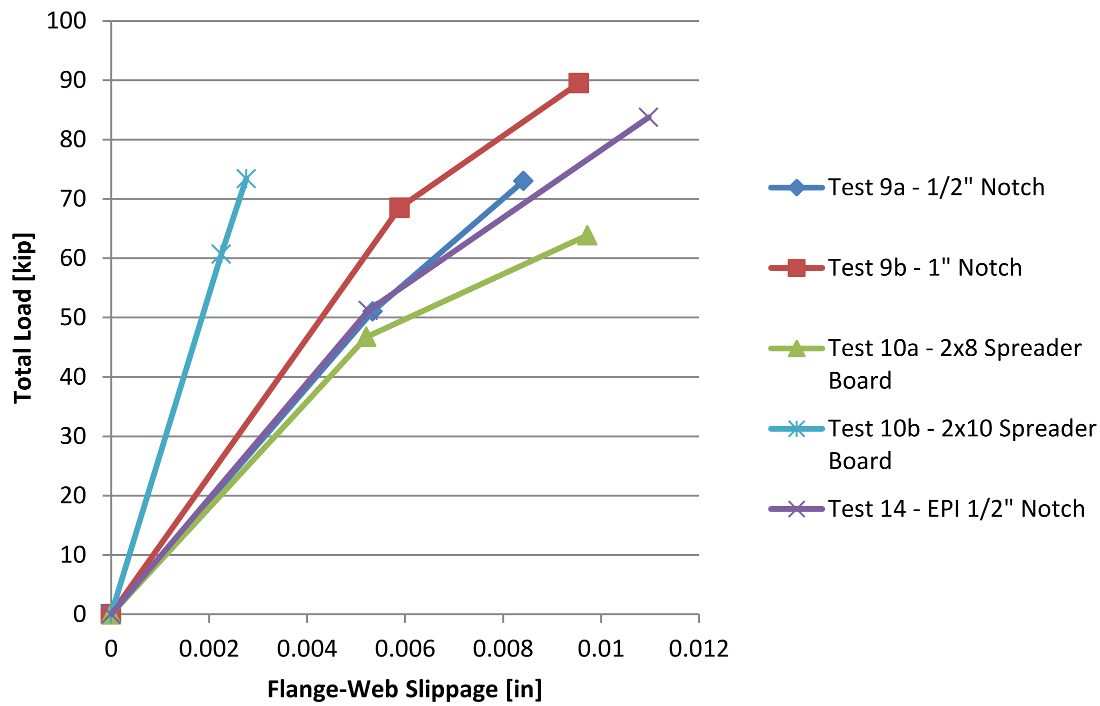

Figure 4.52: Comparison Between Modified Glue Tests with MSR Lumber

Comparing the notch connections, it was observed that the EPI adhesive performed much better than the MF. This can be seen by comparing Test #14 to Test #9a. The reasoning for this was thought to be because the EPI had some gap filling capability and would produce a much better bond on the side of the notch. Also, a higher percent wood failure was observed with the EPI than the MF when the joint was what failed. Even though the highest shear strength for all the tests was in Test #9b with 115 kips of total force, it was concluded that a full 1-inch deep notch was not needed especially when EPI adhesive was used.

Next, a comparison between all screw tests that did not use a membrane will be drawn. The results for these tests are shown in Figure 4.53. From this, it was thought that a larger screw should not be chosen because of cost, performance and constructability. Rather smaller screws in a greater amount should be used in order to get a greater stiffness value. The properties of these screws will have to be put into a model in order to truly see what ratio strength to stiffness ratio is best for a screw connection. Screws placed at 30 degrees were originally not favored because of the difficulty of drilling them, but if CNC machines were used to drill pilot holes then this could be a viable option. This would not be only useful to making it easier to start and drive the screws but would aligned the screws and make sure they are in the proper location without measuring. Also, the possibility of a smaller compression flange could be considered if an unbalanced layup is wanted because a less thick flange is needed for the connection when the screw is placed at 30 degrees.

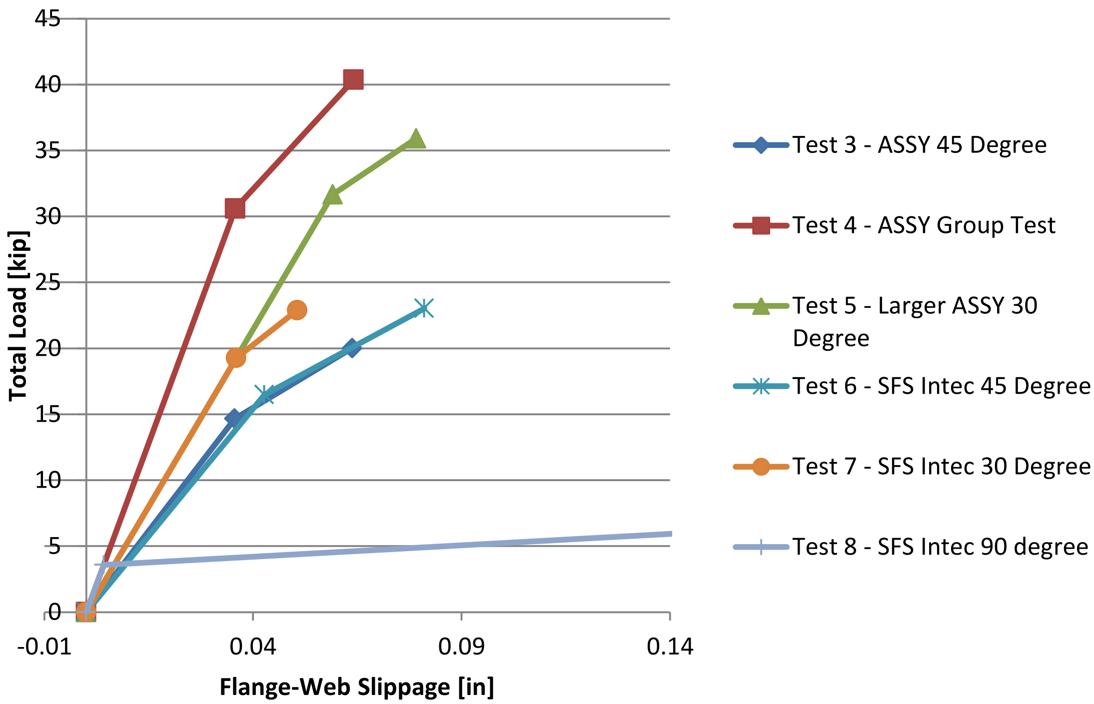

Figure 4.53: Comparison of Screw Tests Without Membrane

Test #11, which was a specimen that combined screws and glue, achieved slightly higher numbers compared to the same test with no screws. This is shown in Figure 4.54 and proves there is not an additive effect. The small amount of added capacity probably came from different amount of clamping since the SFS Intec screws were used and have the ability to clamp or possibly that there was just a small difference in glue bond quality. From observations, what seemed to happen was that the glue took the entire load, and after the glue broke, the entire load went into the screws. When the screws held the load, they were yielded very quickly and held no additional load. Because of the difference in stiffness of the two connectors, they proved to be incompatible. In conclusion, there is no reason why a designer would pay for two systems but get no additional performance.

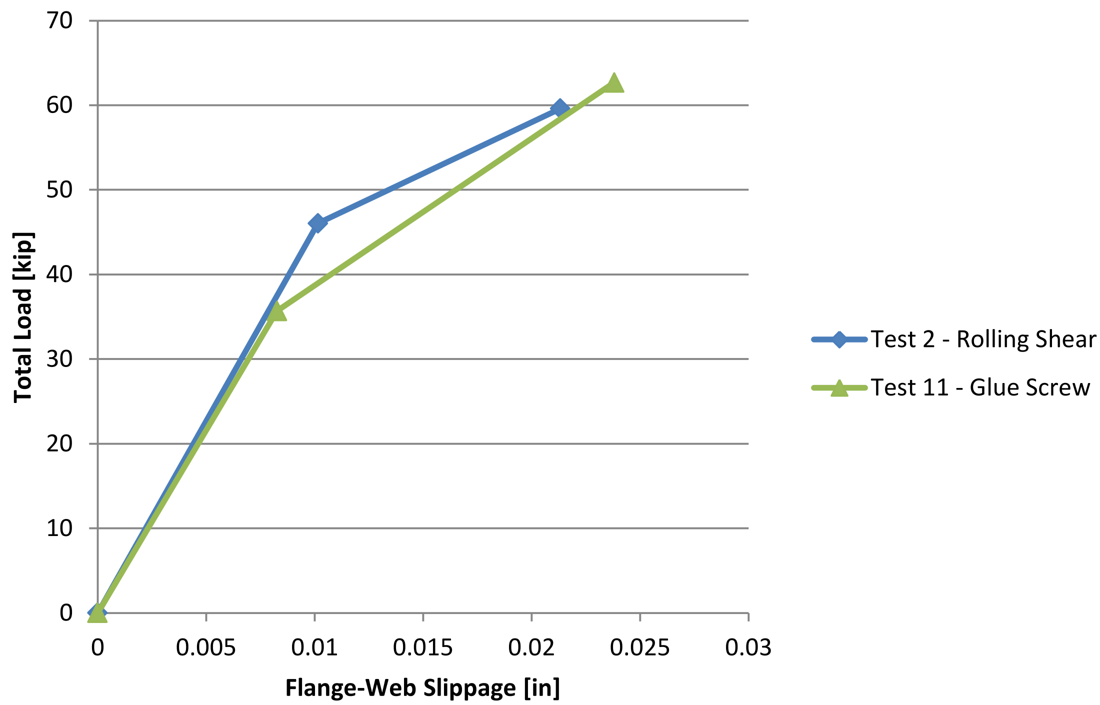

Figure 4.54: Comparison Between Screw Glue and Control

From the results of the two HBV tests shown in Figure 4.55, there was a stark difference in strength, stiffness and failure modes. It was observed that if a gap was not placed between the two sides, there was no ductile behavior at the joint. Ductility is a large selling point of this product and this shows that there needs to be a requirement that insures this performance. Therefore, it should be made known that if a membrane is not placed between the two surfaces there will be less ductility. In addition, it was observed that the HBV with no membrane acted more like reinforcement than a connector, since the glue seeped out into the joint and made it act like the joint was glued. If a very stiff and strong connection that caused failure to happen in the wood was desired, then Test #12 would be a great option. Also, another large selling point for HBV is stiffness. HBV was observed to be stiff, but the overall stiffness of the connection depends on how many are used in a panel or how far are they spaced apart. Also, it should be noted that even though the HBV was only embedded 1 inch, which was less than the 1.575 inches that the manufacturer recommended, it still failed by the steel yielding rather than the steel being pulled out of the wood. Overall, HBV that had a gap between the members to keep glue from making the connection from wood to wood, and that allowed the steel to deform, performed well with good stiffness and ductility.

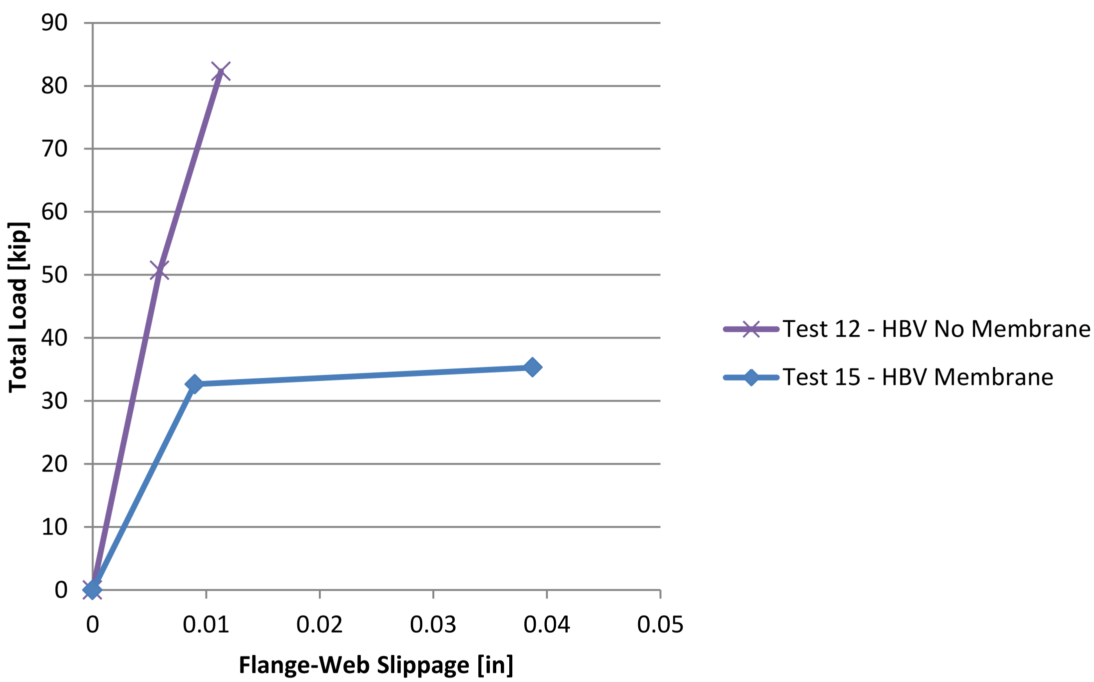

Figure 4.55: Comparison Between HBV Tests

A comparison between a screw with and without a membrane is shown in Figure 4.56. As predicted, the screw with a membrane lost stiffness because of the compressibility of the membrane that allowed the screw to translate more. However, it was not predicted that the screw with a membrane perform better in terms of strength.

This could probably be contributed to have to the screw having a slight change in angle when allowed to deform more. This change in angle would allow more of the strength of the screw to act in the direction resisting shear and could allow for a higher shear strength. The screw membrane had a 37.1% loss it stiffness, which might not be good because this will mean that the overall stiffness for the connection will be less and this will cause the panel to act less composite action. How much this will affect the performance cannot be determined until the stiffnesses are placed in a model and design lengths can be quantified. Since it was desired to gain some of the stiffness lost back, a screw at 30 degrees through a membrane was tested for a final screw connection shear test.

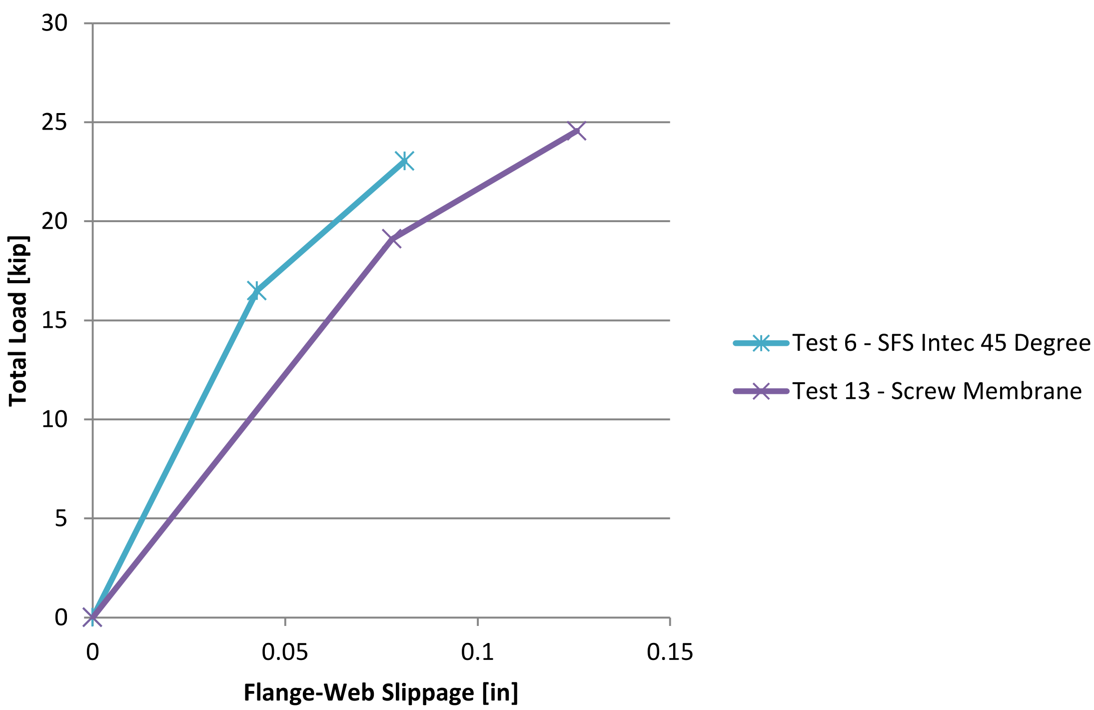

Figure 4.56: Structural Effect of Adding a Rubber Membrane

Figure 4.57 might be a bad visual comparison between Tests #13 and #15, because the total amount of connectors placed in a HMT panel will be based on the design strength of the connectors. Since the amount of each connector that will be used in a panel is unknown, the overall stiffness that the connectors would provide cannot be compared. In general, even when the HBV is spaced farther apart in a panel and when screws are placed closer together than when tested, the HBV will have a produce a significantly stiffer connection. Moreover, HBV was seen to have a decent ductile failure mechanism.

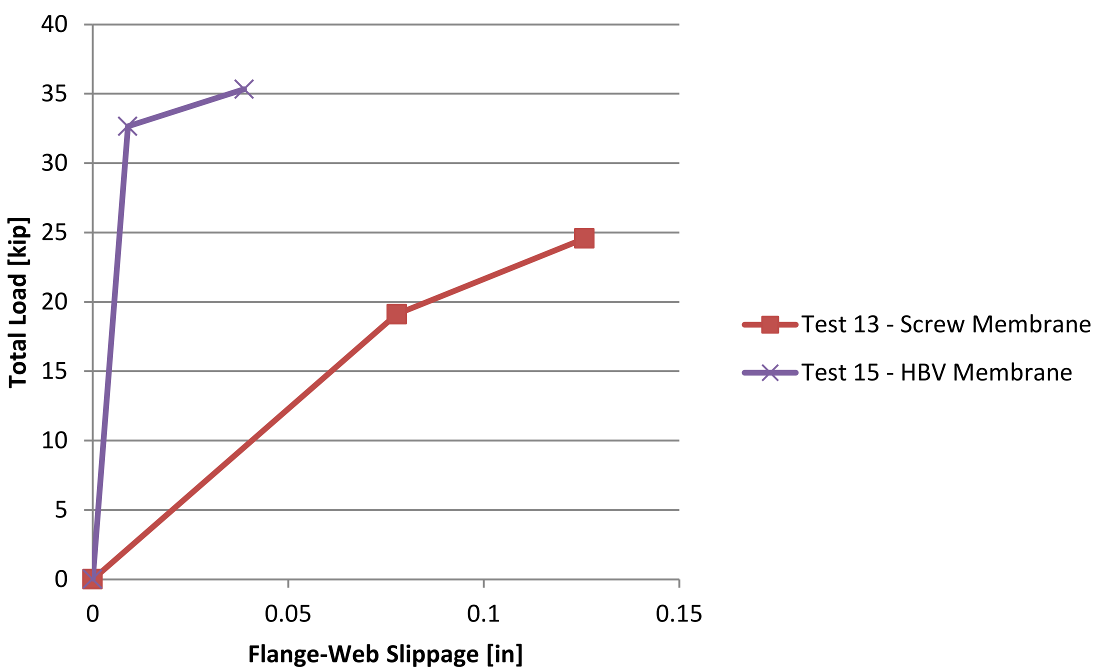

Figure 4.57: Comparison Between HBV and Screw Membrane

A comparison between GFRP and the HBV membrane is made in Figure 4.58. The GFRP did achieve a higher load than the HBV but did not have a ductile failure mode. Also, it was observed that GFRP was a more fragile of a material than steel and it might have to be treated with more care before and while installing. The GFRP did show warning signs of failure by usually making loud cracking noises well before rupturing but it also made noise under low-level loads that could be annoying to occupants. GFRP would probably be cheaper to produce and achieve a higher load, but if it was desired to be used a better adhesive should be found. The adhesive used to embed the GFRP had low viscosity and thus significant flow. Therefore, GFRP could be a very good structural solution, but it is only best for situations that match up with its advantages.

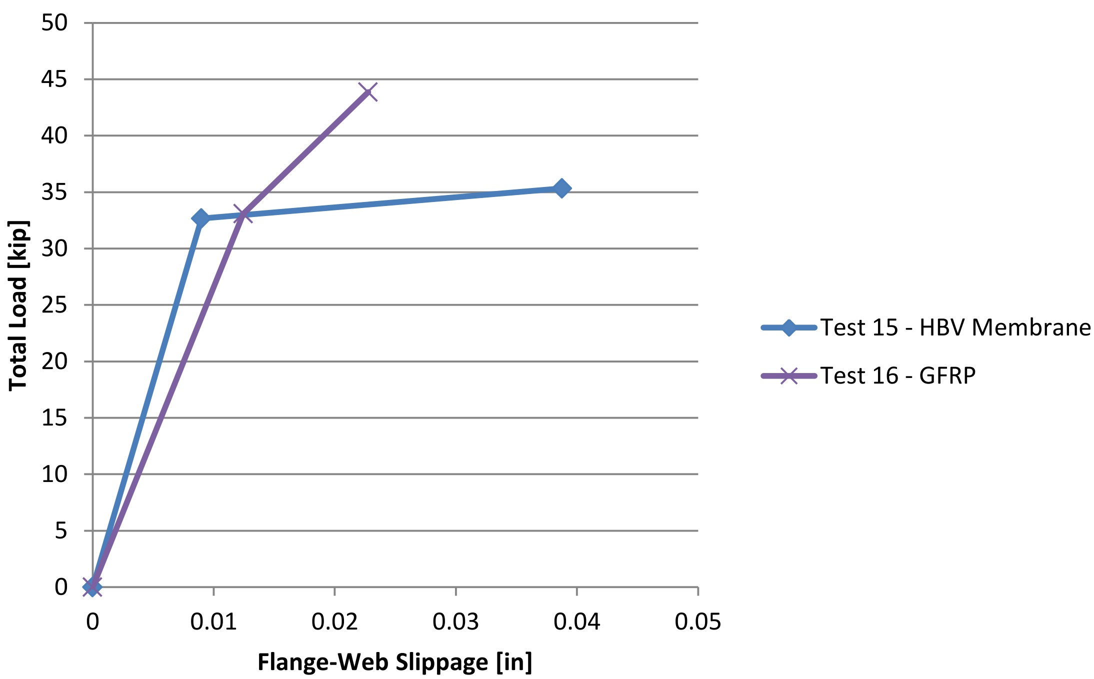

Figure 4.58: Comparison of Spline Connectors

A comparison of the results from all the notch tests is shown in Figure 4.59. It was found that there was great variability in the results of the MF notch tests (Tests #9a and #9b), which is not good for strength design. The exact reason for this is unknown but could be related to accuracy of the notch size and if insufficient clamping happened a gap might have been introduced. However, the EPI seemed to perform well by achieving high capacities while having lower scatter. This might have been because of the slight expansion of the glue that occurs when the adhesive cures, as stated before. Test #17 shows to be slightly less stiff probably because of the change in rolling shear properties because of the use of a less dense wood. These tests also broke at lower values because they broke in the glulam rather than the joint due to the glulam strength being reduced because of using both Grade #2 and #3 lumber. This is not necessarily bad to have this result because the goal of the glue connection was to try to always have the failure be in the glulam instead of web so that the shear strength just depends on the glulam rather than introducing the variability of the glue bond at the connection.

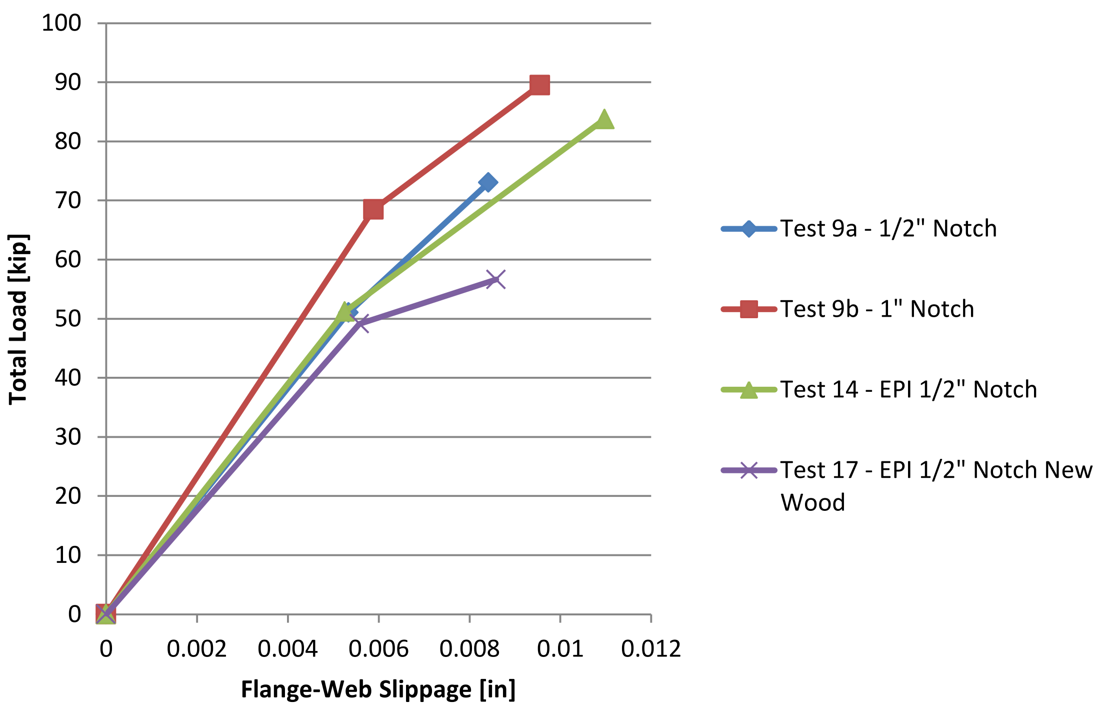

Figure 4.59: Comparison Between Notch Tests

A comparison between final modified screw tests is shown in Test #18 performed very well and by placing the screw at 30 degrees instead of 45, most of the loss due to the membrane was able to be gained back to the level of a 45 degree screw with no membrane. Even with the less dense Grade #3 lumber, all screws ruptured and did not fail by pull through the wood. As predicted, the 30 degrees increased the stiffness and strength of the screws when comparing Test #18 to Test #13 for stiffness and Test #18 to Test #3 for strength. ASSY screws were used for this final screw test because it was decided that ease of driving the screws was a priority. SFS Intec screws had a smaller head that would tend to strip more, and if it was extremely difficult to remove the screw once stripped. Grade #3 lumber was used for the whole panel for all specimens of Test #18 and there were no boards thrown out because of quality issues, and all defects were used. This was done to show that the screws performed well even when embedded into a low-grade lumber. Also, this was done to represent if the top flange that was in compression was made out of all Grade #3 lumber.

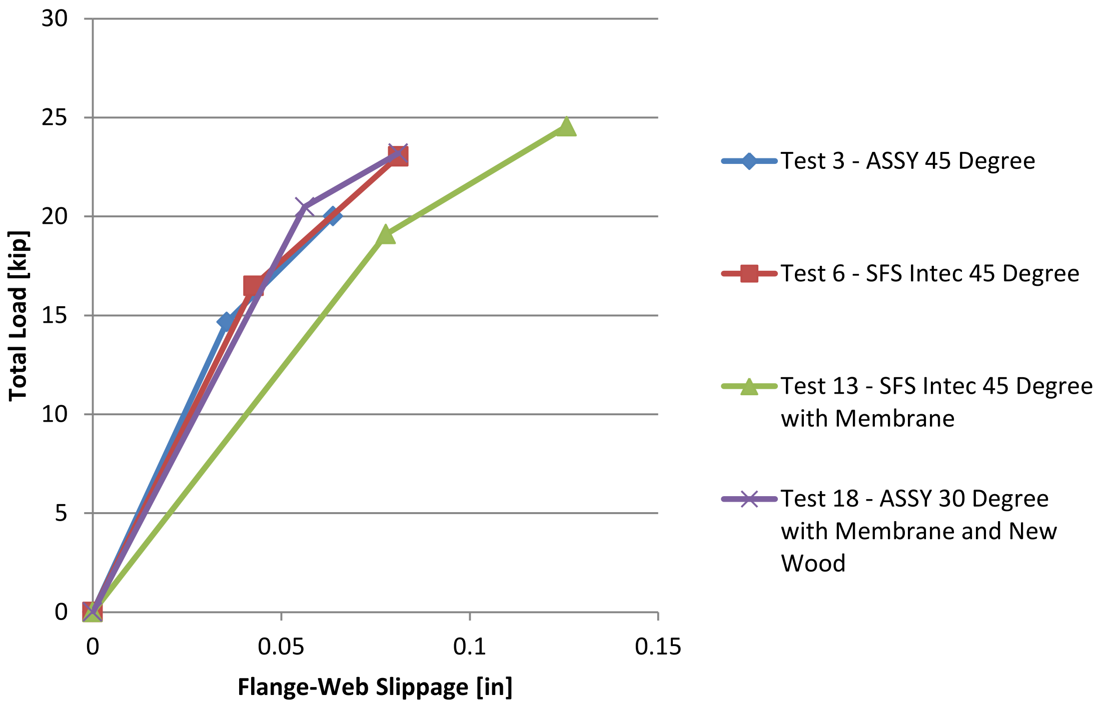

Figure 4.60: Comparing Final Modified Screw Test to Previous Results

Figure 4.61 compares the two ductile connections that would be considered for a HMT panel. Simpson screws were found to be much more ductile than the HBV, but also a good bit less stiff. The stiffness and ductility was found to change if more or less screws were used. To stiffen the SDS screw connection, screws could be added. Likewise, the HBV could be changed if desired to become less stiff and more ductile if a thicker membrane was used. Also, it should be noted that premature failure of the glulam did occur in Specimen #1 of Test #21 because of the screw spacing being too small.

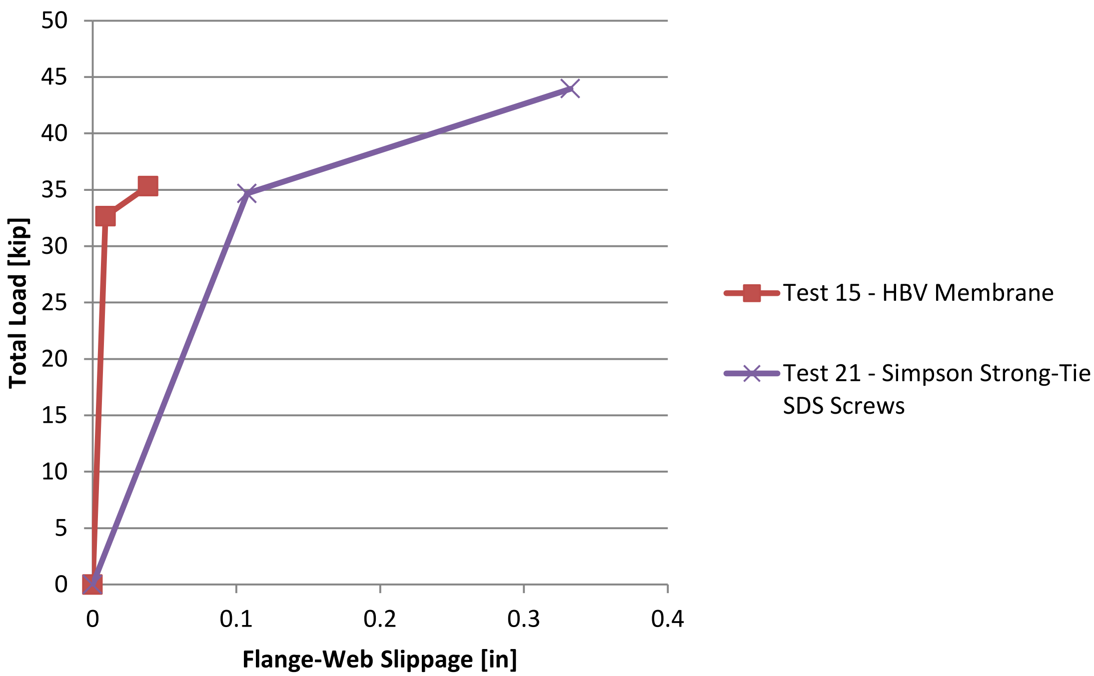

Figure 4.61: Comparison of Ductile Connections

The need for ductility in connectors might be overstated as a requirement for connection design where the connectors are used for composite action. This is because once a connector yields the joint is allowed to displace significantly which allows for the panel to lose a large amount of composite action. This effect of yielding that causes less composite action acts in a loop because when the panel acts more like individual elements then it loses strength and then the panel deflects more which causes the connectors to yield more. Since the panel doesn't get stronger when the connectors yield, it may not be a large benefit to have ductile connectors because this could cause immediate collapse soon after the connectors yield and this would not be much different than a brittle failure mode. A benefit that can be contributed to having a ductile connector in this situation would be if there could be load sharing between the web members or between adjacent panels. It should be noted that for load sharing to occur, ductile connections are not a requirement, but could add to the performance of the load sharing. Past a certain amount of ductility that allows for an increase in strength due to load sharing, there seems to be not particular benefit, because the bending element will just become weaker and fail unless the load is removed immediately. Also it should be noted that the loading that is being considered is not a seismic load that needs connectors to be ductile so that energy is being dissipated.

A comparison between two final flat bond tests, shown in Figure 4.62, were tested and compared because it was thought that a normal flat glue bond that was not modified could possibly produce an adequate connection between the flange and the web. This was because of the reduced strength of the glulam found in Test #17. All though the results from Test #19 and Test #20 were very similar, EPI with screw pressure would be chosen over MF with 150-psi pressure, because of the fact that manufactures would not have to waste time using their press to get the same properties as just being clamped with screws. This was surprising to see that two tests that had dramatically different bonding pressures had such similar results. This was attributed to EPI being able to fill in gaps that would normally be closed with large amounts of pressure. Inspecting the EPI tests did show that there were a lot of places that didn't bond because of the roughness of the CLT crosswise board that deformed because of shrinkage due to a loss of moisture much like observed in the spreader board test. If more glue was applied and the glulam and flange were put together close to the time when the CLT panel was pressed, then a much better glue bond would have resulted. The MF specimens looked like it was a perfect bond when observed but still seemed to tear the wood off right at the interface more easily. Therefore, there seemed to be no improvement that could be made on the MF bond. Also, it was observed that the EPI adhesive seemed to penetrate the wood deeper so that more wood had to be torn off and this could possibly explain the better performance.

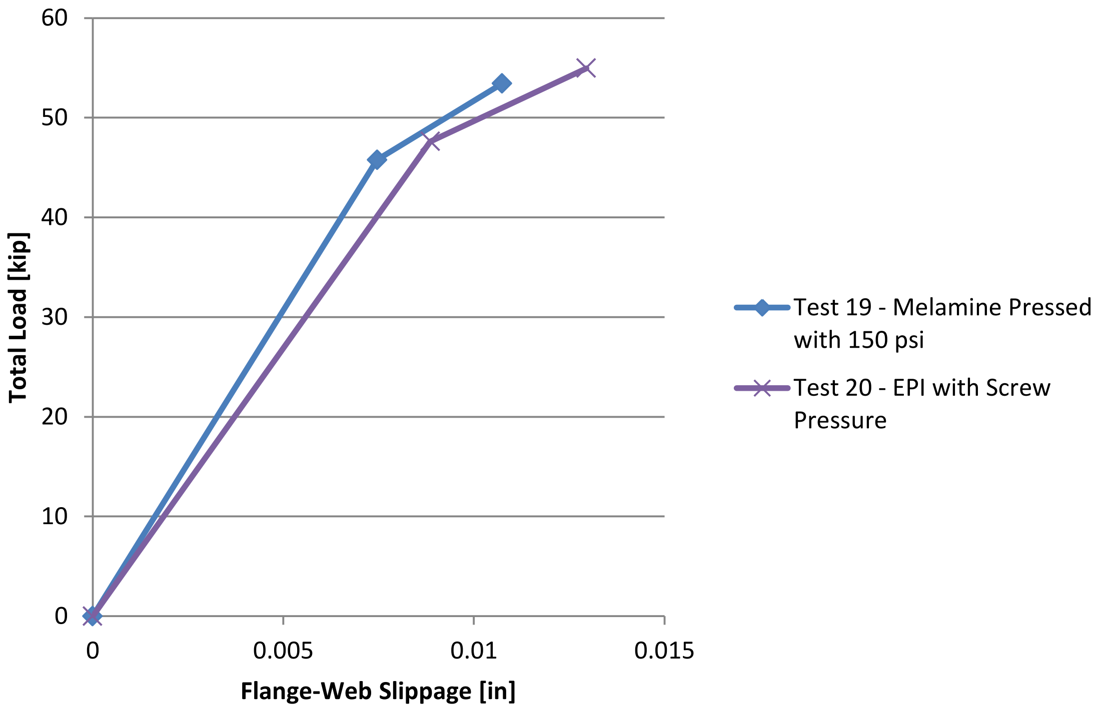

Figure 4.62: Comparison Between Flat Glue Bonds with Grade #2 and #3 Southern Pine Lumber

In comparison between all the different flat glue bonds shown in Figure 4.63, it was seen that all of them had similar stiffness properties besides Test #2 Redo which was as expected. Since a glue connection is very stiff, it was assumed that the minor variations of stiffness wouldn't matter. The shallow rolling shear failure that happens for panels that have the crosswise board at the connection can be seen quite well by the change of stiffness that is happening around 45 kips equivalent to 409 psi. This could be used to identify warning signs of a failure in shear. Also, the EPI that was pressed with screws was a very poor quality bond, and can be expected to perform much better if slightly more glue was used and a flatter bond surface was obtained by making the connection sooner after the flanges were pressed.

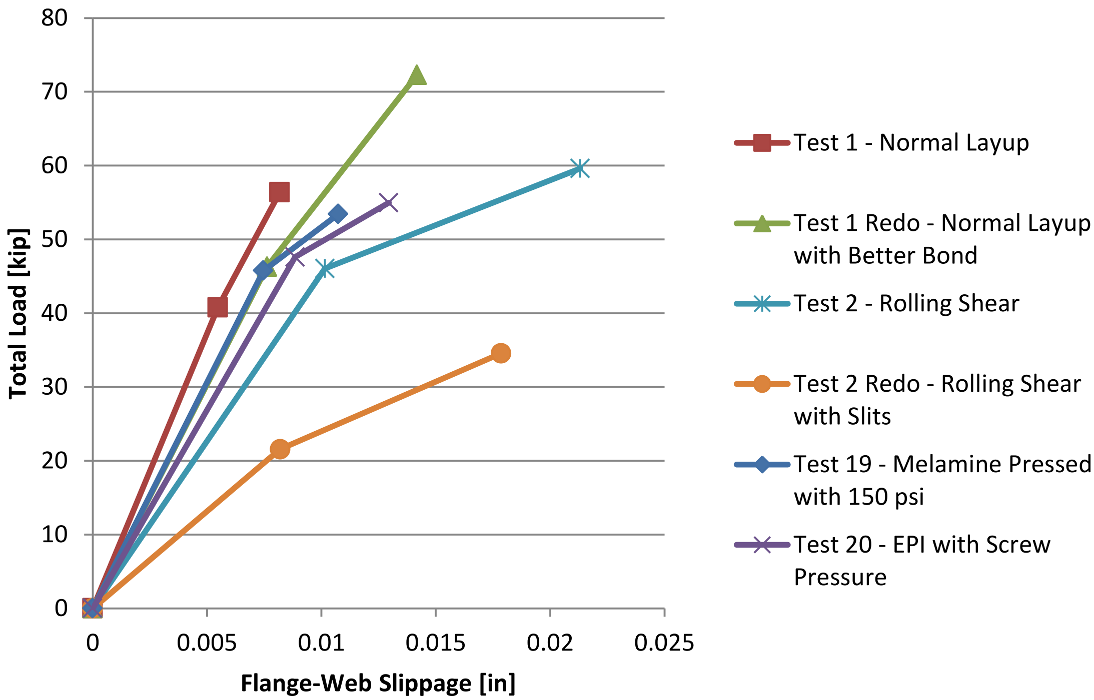

Figure 4.63: Comparison Between All Flat Glue Bonds

To conclude, the final options for the bottom connection will be discussed. These options are shown in Figure 4.64. From the test results EPI glue seems to preform very well for the structural bonds because it grabs the wood and it tends to be able to gap fill. Therefore only EPI bonds were chosen because of their performance. Since the brittle behavior of a glue bond is more unpredictable, it is desired to have a strong enough connection so that the failure would occur in the glulam. This would not change the brittle failure mode but this would max out the capacity of the connection and provide slightly better consistency by allowing the glulam to govern the capacity. The average strength of the glulams which were made of the Grade #2 Southern Pine lumber was 56.6 kips or 515 psi with the low being 44.3 kips or 369 psi and the average strength of the glulams made of MSR lumber was 82.3 kips or 748 psi with the low being 64.4 kips or 585 psi. If a connection was chosen that had an average capacity over 82.3 kips, it would consistently insure that the connection would be maxed out by the capacity of a glulam made of higher grade lumber. Since only Grade #2 lumber is planned to be used in a glulam of a HMT panel, a capacity of only 56.6 kips is needed . Tests #17 and #20 were chosen because their averages were around the targeted ultimate capacity.

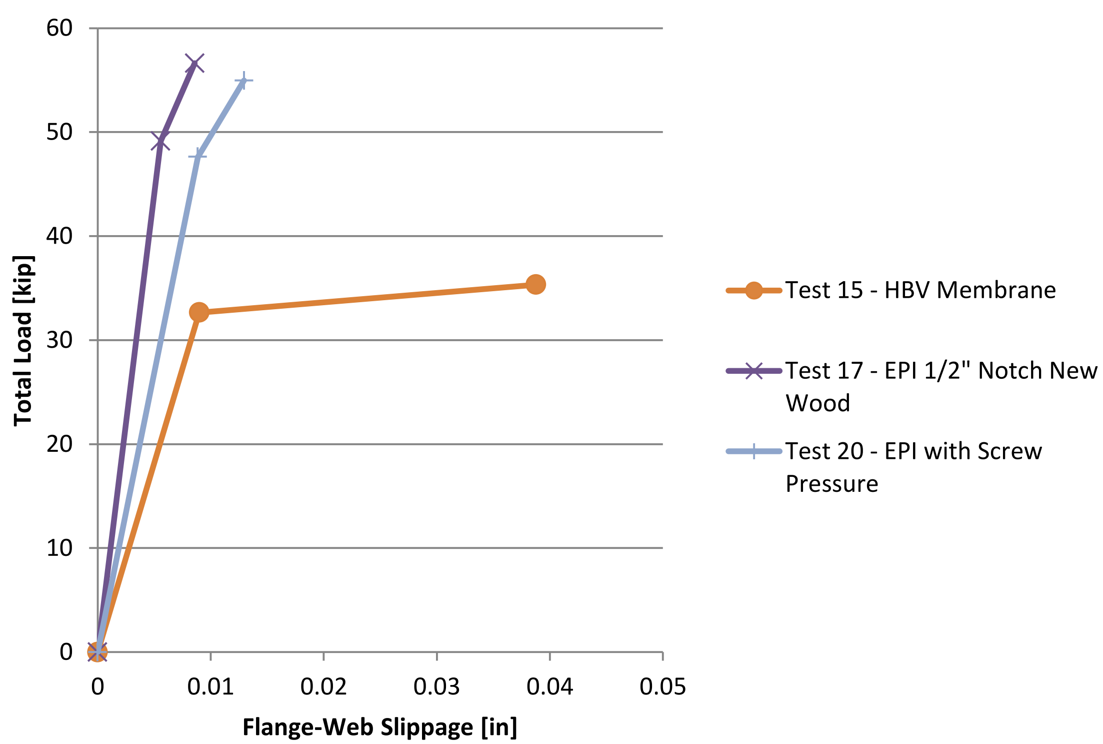

Figure 4.64: Final Options for Bottom Flange Connection

Because each specimen in Test #17 failed in the glulam, ½-inch notch is more than enough and might actually be too much. A glue bond with EPI glue clamped with screws should have enough strength for what is needed but it does not accomplish making the glulam the limiting factor. Also, a benefit of the flat glue bond would be that it is very easy to accomplish compared to a notch connection.

An HBV connection will not be chosen but could be an option to investigate more if a more ductile connection is desired. A glue connection was chosen over this connection, because of strength, stiffness and ease of construction.

What was determined as the best options for the top flange connector are shown in Figure 4.65 and will be compared so that final recommendations can be made. On the top panel connection, a mechanical connection was wanted so that the top panel could be attached in the field and the quality of the glue bond wouldn't be an issue. Similarly to how it is advantageous to shop weld and field bolt for steel, it is seen as advantageous to shop glue and field screw. It is believed that gluing is more of an art form and the quality of the bond effects the strength more. Because of this, the gluing for the flange to web connection will be limited to a controlled setting that is not on site. It is realized that if nothing is going to be put into the hollow void of the panel or if it wiring, piping or ducts were placed in the panel before shipping to the site then a glue connection for both top and bottom would be a good possibility. The glue connections are predicted to be strong, constructible and slightly cheaper than other more complex connections. With this being said, other performance aspects have to be considered like acoustics. If a membrane between the flange and the web is found to improve the acoustic characteristics of the floor, which is predicted, then it will be very advantageous to use inclined screws and if it is found not to be advantageous an all glue solution should be sought.

No other connector tested was predicted to gain any acoustical performance if the membrane was implemented in the connection. With glue connections, the two wood layers have to be in contact to form a bond, therefore a membrane can’t be placed in between. Also, with HBV was thought that it would be too stiff of a vertical connection and the vibration would transmit straight through the connector so that not as much damping would occur. These properties are believed to be different with a screw because 1) the connector can go through the membrane while still resisting shear loads and 2) a screw at 30 degrees has much less vertical stiffness than HBV and should be able to transmit less vibration through the floor. Depending on acoustic testing, a screw with a membrane should be chosen if the membrane helps acoustical properties of the panel and a glue connection the same as the recommended bottom connection should be chosen if the membrane does not help the acoustical performance. If the screw membrane connection is chosen then it is recommended that the screws be placed at 30 degrees because of the slight increase in strength and an increase in the stiffness when compared to being placed at a 45 degree angle.

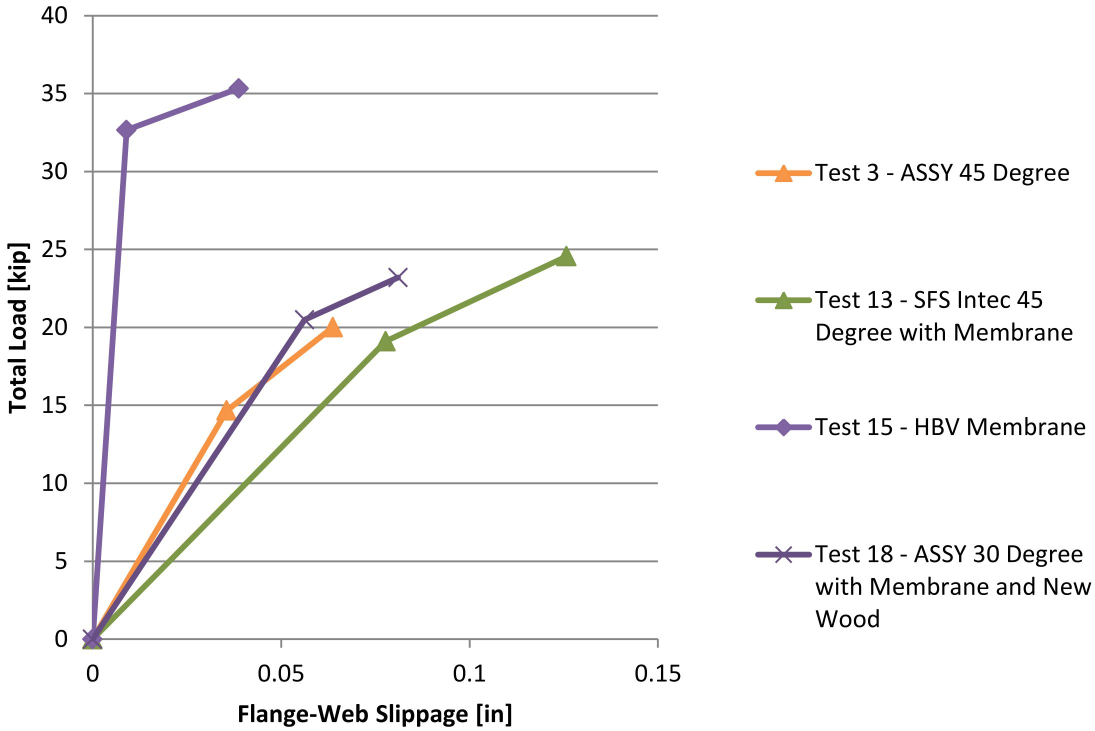

Figure 4.65: Final Options for Top Flange Connection

An HBV connection will not be chosen but could be an option to investigate more if a more ductile connection is desired. A screw connection was chosen over the HBV connection because it had acceptable strength and stiffness while being able to use a membrane for acoustical damping and be able to place MEP in the hollow void in the panel and close the panel in the field by attaching the top flange with screws. A glue connection could also be used as a top connection but this would take away some of the advantages that a screw connection has. A glue connection will be investigated in Chapter 5 to see the difference in performance.

Conclusions

The goal of this Chapter was to find the best solutions for what should be used as the web and what should be used as the flange to web connector. First, many wood products were investigated to see how well each would perform as a web member. What was looked for was a strong enough member so that the spacing of webs could be reasonably wide. Also, the web member was chosen based on constructability, fire performance, the ability to contain fire, and having enough area to make the flange to web connection. A 2.5-inch wide glulam was chosen to be spaced at 32” on-center for a total of three glulams per 8-foot wide panel.

Then a wide assortment of different connection techniques was investigated. Most of the connections could be broken down into glue or mechanical connections. Based on strength, stiffness, constructability and predicted improvement in acoustics, an inclined screw placed at 30 degrees through an acoustical membrane was chosen to be the best option for the top flange connection. Other possible solutions are HBV, glue and screws without membrane connections with a glue connection having the most advantages including stiffness, strength and cost, if the screw membrane was found to have no substantial acoustical improvement. For a bottom flange connection, an EPI glue bond was found to be the best. Either a ½-inch notch or a flat glue bond is recommended for this connection. A flat glue bond was assumed for the rest of this research because of its ease of construction, and it was thought that it would produce high enough strengths to resist design loads. After full-scale panels are tested, this can be confirmed or reevaluated. Performance of these connectors when placed in a full-scale model in

Chapter 5 will show more of how different stiffnesses and strengths affect the overall panel.

CHAPTER FIVE

MODELING OF A FULL-SCALE PANEL

Introduction

In order to form final conclusions on the panel configuration, connector type and connector configuration, modeling of a full-scale panel was used in the immediate absence of full-scale panel testing. The SAP2000 model that was presented in Chapter 3 with strength and stiffness values from different connectors provided in Chapter 4 was utilized in this analytical study. From this modeling, the effect on the overall performance of a panel from various connections could be evaluated and a recommendation for the selection of a connection to maximize the performance of a panel could be made. From final recommendations drawn from these studies, final predictions of the performance of a HMT panel were made. One additional topic that will be addressed in this section is fire design. By modeling of a char layer on the bottom surface of the panel, a fire resistance of the recommended final design was found.

Flange-to-Web Connection Model

The SAP2000 model used to model full-scale panels in this Chapter was modified from the model used in Chapter 3. Modifications included the replacement of linear link elements with non-linear links were needed to allow for the bi-linear behavior of different connectors in Chapter 4. Since there was a possibility that under ultimate loads the connector could be acting non-linear while under service loads the connectors could be acting linear, the model was run twice for both of these load cases. For strength calculations, model results of the panel subjected to the full ultimate load were used while for serviceability calculations, the loads on the model were lowered to service level loads and then the model was analyzed.

A validation of the model for the shear tests were done by plugging in shear tests loads into a model and checking if the correct displacement was obtained. The model of the shear test can be seen in Figure 5.1. When ultimate loads were applied and when the springs were set to an appropriate bi-linear model that corresponded to the stiffness of the screws, it was found that the tested value and model value matched very closely. As an example, Test 13 had a maximum deflection of 0.126” and when modeled a deflection of 0.125” was obtained which gives a 0.36 percent error. Therefore, the flange to web connection model was found accurate to use for the full-scale modeling.

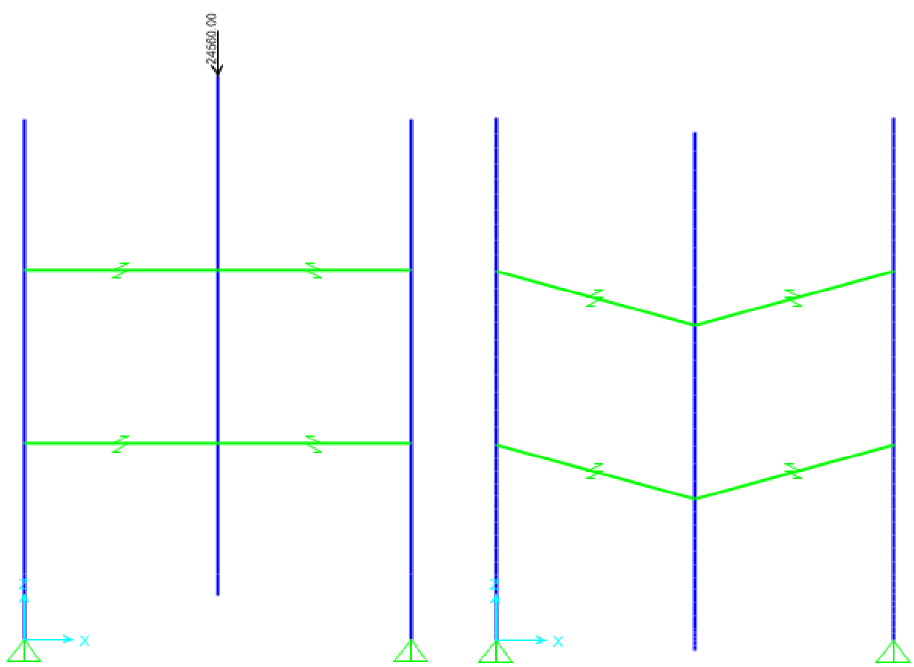

Figure 5.1: Applied Load and Deflection on Shear Test Verification Model

Refinement of Panel Design by Full Scale Modeling

This section investigates different parameters to make sure that the recommendations for a HMT panel were optimized. Different size screws at different angles were investigated along with other possibilities like screws which were not combined with membranes, using a #3 lumber top flange, different spacings of screws along the length of the panel, and a panel with all glue connections. By comparing the cost of connectors or the design span lengths obtained with identical cross sections, it could be seen how much change each variable made in the performance or cost of the panel. The results obtained were analyzed considering other non-structural advantages or disadvantages of the implemented change.

Before placing the screws in the full-scale model, the number of screws and screw spacing had to be designed to resist the shear flow from the web to the flange. Equation 2.1 taken from the CCMC evaluation report for SWG ASSY screws was chosen to be used to obtain the design strength of the screws because it was found to be most accurate to the true performance of the screws. To get the force that was needed to be resisted, total shear flow from the center span of the panel to the end was calculated. The total shear flow was then divided by the shear capacity of one screw and then rounded up in order to the amount of screws needed in half of one web of a panel. Since there are three web members in a panel, this number would be multiplied by six to get the total amount of screws in a panel. Since usually vibration controlled the design length of the panel, screws were designed for the vibration length and not for the full strength design span. The total amount of screws was then divided up into three regions, which in combination made up half of the span of a panel. These three regions were used to group an amount of screws to have an equal spacing in each region. A uniform, slightly weighted and heavily weighted arrangement of the screws were used for analysis. The uniform arrangement would have an equal spacing in all three regions with $^{1}/_{3}$ of the screws for a half of the panel in each region. The slightly weighted arrangement was proportional to the shear diagram that meant that more screws would be placed toward the outer ends of the panel. Each amount of screws in each region would be equal to the shear flow in that region. This would allow for $^{1}/_{2}$ of the screws to be in the outer end region, $^{1}/_{3}$ to be in the middle region, and $^{1}/_{6}$ of the screws in the central region closest to the center of the panel. The heavily weighted arrangement places more screws near the ends of the panel even more than the slightly weighted option did. The ratios were $^{2}/_{3}$ in the outer end region, $^{2}/_{9}$ in the middle, and $^{1}/_{9}$ near the center of the panel. This was thought to be the best option because it believed that the screws towards the ends of the panel were more effective because of the higher shear loads are there.

When the number of screws in a region was determined, the stiffness and strength of the total amount of screws in the region were divided up equally for each link in that region. These bi-linear models that represent the stiffness and strengths from the screws were combined in series with the properties of the wood in order to account for both shear deformations and the partial composite action of the panel. This was not only done for the screws but also for the glue connection. However, the glue connection was taken to be constant along the length of the panel because it was assumed that glue connection would be applied along the entire length of the panel and not at an intermittent spacing.

For modeling of the screw connection that used a rubber membrane, it was assumed that no extra depth was added to the panel because thickness of the membrane placed between the web and the top flange was minimal. It was hard to quantify this small amount; therefore, no extra gap was placed in the model. This was done in order to be conservative and also this was beneficial since all models were being compared at the same depth.

Since the optimal screw size, spacing and orientation was wanted to be found, a study was done which found the design lengths and cost of screws for the same cross section while varying the studied parameters. Therefore, design values of the screws were calculated based off Equation 2.1 and are shown in Table 5.1 along with a description of the test. For all these tested screw sizes and orientations, it can be assumed that a membrane and ASSY screws were always used unless stated otherwise.

Table 5.1: Screw Design Strengths

Model Number	Screw Size and Orientation	Factored Tensile [lb]	Factored Shear [lb]	Controlling Equation
1-1	$^{5}/_{16}$" x 11$^{3}/_{4}$" at 45^{o}	2965	2097	Withdrawal
1-2	$^{3}/_{8}$" x 11$^{3}/_{4}$" at 45^{o}	3654	2584	Withdrawal
1-3	$^{5}/_{16}$" x 11$^{3}/_{4}$" at 30^{o}	2804	2428	Withdrawal
1-4	$^{5}/_{16}$" x 13" at 30^{o}	3100	2685	Withdrawal
1-5	$^{5}/_{16}$" x 14$^{1}/_{8}$" at 30^{o}	3396	2941	Withdrawal
1-6	$^{5}/_{16}$" x 15" at 30^{o}	3593	3112	Withdrawal
1-7	$^{5}/_{16}$" x 17" at 30^{o}	3702	3206	Steel
1-8	$^{3}/_{8}$" x 11$^{3}/_{4}$" at 30^{o}	3455	2992	Withdrawal
1-9	$^{3}/_{8}$" x 12$^{5}/_{8}$" at 30^{o}	3702	3206	Withdrawal
1-10	$^{3}/_{8}$" x 13$^{3}/_{8}$" at 30^{o}	3949	3420	Withdrawal
1-11	$^{3}/_{8}$" x 14$^{1}/_{8}$" at 30^{o}	4196	3634	Withdrawal
1-12	$^{3}/_{8}$" x 15" at 30^{o}	4442	3847	Withdrawal
1-13	$^{3}/_{8}$" x 15$^{3}/_{4}$" at 30^{o}	4689	4061	Withdrawal
1-14	$^{3}/_{8}$" x 17" at 30^{o}	4700	4071	Steel

Also, the real stiffness and strengths had to be put into the model in order to predict performance. This was easily done for Model 1-3 because this was an exact replication of Test #18 found in Chapter 4. But since none of the others were specifically tested, values had to be extrapolated for them. This was done with help from other screw tests experimentally tested. Ratios from one test to another were found to account for things such as angle, diameter, use of membrane or not, and which manufacture made the screw. Screws of the same size and diameter and orientation, but have different lengths were assumed to all have the same strength and stiffness characteristics. From tests presented in Chapter 4, it was found that all of the screws tested ruptured in the steel, yet by design they were supposed to fail through withdrawal. This was even found true when tested with flanges made out of all Graded #3 lumber and with the screws had much shorter penetration depths than theoretically needed to obtain rupture of the steel. Therefore, it was determined that for the range of screws being investigated that extra penetration depth would not affect neither the strength nor stiffness.

From these input values, the design span length for each design requirement could be found along with the total screws needed for the vibrational design span since this governed the design. These values can be seen in Table 5. 2.

Table 5.2: Design Lengths with Varying Screw Diameter, Length and Orientation

Model Number	Total Screws [Screws/panel]	Vibration [ft]	Strength [ft]	Live [ft]	Long [ft]
1-1	168	39.91	44.56	48.99	48.99
1-2	138	39.71	44.43	48.79	48.85
1-3	150	40.34	44.87	49.38	49.20
1-4	138	40.11	44.72	49.18	49.07
1-5	120	39.89	44.55	48.98	48.91
1-6	114	39.77	44.48	48.85	48.89
1-7	108	39.65	44.40	48.82	48.87
1-8	126	40.16	44.72	49.21	49.02
1-9	108	39.90	44.59	49.07	48.99
1-10	102	39.77	44.51	48.93	48.99
1-11	96	39.64	44.43	48.78	48.98
1-12	90	39.48	44.27	48.63	48.74
1-13	90	39.44	44.19	48.51	48.51
1-14	90	39.44	44.19	48.51	48.51

From Table 5.2, it can be seen that vibration controls the design of the panel. For the standard cross section, which has an 11-inch deep glulam web to make a 19¼” deep panel, it can be seen that a design length of around 40 feet can be obtained. Strength tends to be approximately 5 feet longer than the vibration design span and both the long-term loading deflection equation and the live load deflection equation yield a design length approximately 9’ above the vibration length. Even though the goal was not reached of not having vibration and strength design be the same, improvement in efficiency of a massive timber panel has been made. Also, the results presented are just modeled values, and could change when full-scale experimental tests are done. Having higher strength values than needed and lower deflection values than needed could be a beneficial aspect of a panel design. Many designers might question using wood because of concerns about its strength and stiffness especially when relating to long-term deflections since wood is known to creep. Therefore, higher performances than code minimums might prove to be beneficial in the acceptance of the product by designers and might lead to higher satisfaction with occupants of a building using this system.

From small-scale modeling it was not known how much the full scale performance would change if different stiffness from different connectors were used. It was assumed in Chapter 4 that more small screws would perform better than using fewer large screws that did not have the same amount of stiffness gain as strength gain. This was proved to be true when modeling many variations of large and small screws in a panel, but it was found to only make a slight difference of just a couple of inches in the span of a panel. A larger difference was found in the cost comparing the larger and smaller screws. This can be seen in Figure 5.2, where the costs of the screws were normalized by span length so that they could be compared. It was observed that less larger screws tended to cost more than using a larger number of smaller screws. When comparing the best option of each, it was found that there was a $62.70 difference per panel. Also, it was found that there could have been a cost difference of $227.80 difference per panel when comparing the overall highest costing screw design to the lowest. These cost differences would be fairly small when compared to the full price of a panel but could be significant when thousands of screws in hundreds of panels are used. Also, the decision was made to continue to use the smaller screws because they were easier to place properly in the panel and also they would provide more redundancy.

Other observations from Table 5.2 were that it is advantageous to use screws at 30 degrees instead of 45 degrees. Also, it was found that the larger screws did not decrease the amount of screws required enough to be found to be advantageous for workers that could be putting a HMT panel together. By changing to a larger screw there was an approximate reduction of 25% of the screws. This was not found to be enough to have any advantages because of cost of labor to put in the screws mostly because the fact that the larger screws take longer to drive in. By comparing the costs, it was concluded that the ASSY VG $^{5}/_{16}$"x14$^{1}/_{8}$" screw placed at 30 degrees provided the best performance for the cost of the screws.

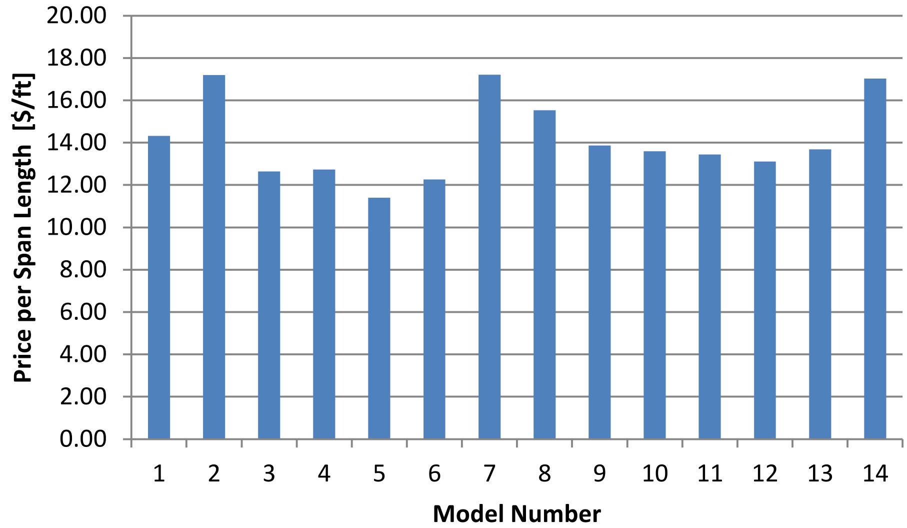

Figure 5.2: Normalized Cost of Screws

The next set of models run had a reference model (Model 2-1) that used the recommendations concluded so far. Model 2-1 used an 11-inch deep glulam with a flat EPI bond for the flange to web connection on the bottom and a top connection made with ASSY VG $^{5}/_{16}$"x14$^{1}/_{8}$" screw placed at 30 degrees through a membrane. The reference model had design spans that can be found in Table 5.2 under Model 1-5 and were used as a reference in Table 5.3.

Table 5.3: Difference in Design Span Lengths Compared to Reference

Model Number	Model Description	Total Screws [Screws/panel]	Vibration [ft]	Strength [ft]	Live [ft]	Long [ft]
2-1	Reference	120	0.00	0.00	0.00	0.00
2-2	#3 Lumber for Top Flange	120	-0.20	-0.17	-0.39	-0.28
2-3	Uniform Screw Spacing	120	-0.20	1.10	-0.07	0.04
2-4	Slightly Weighted	126	0.16	0.54	0.22	0.16
2-5	All Glue Connections	-	2.91	2.09	1.95	0.82
2-6	No Membrane a 30^{o}	126	1.05	0.63	0.77	0.39

From these results presented in Table 5.3, multiple conclusions can be drawn. Since there is only a small drop in design span when the top flange is made of all Grade #3 lumber, then it would be advantageous to use this cheaper wood to save on cost while getting the very similar performance. Because wood has higher design compressive strength values, #3 lumber can be used while not changing anything else about the design. Also, shown with Test # 18 in Chapter 4, using all Grade #3 lumber for the flange had no impact on the strength of the connector. Furthermore, since this panel like CLT is unsuitable to use as finished floor, it will have to be covered up. This would make sure that there would be no problem with the appearance of the wood.

From Model 2-2, it was seen that there was only a slight decrease when moving from a heavily weighted spacing to a uniform spacing. From Model 2-4, it was observed to have a slight increase in design length. Therefore, a slightly weighted arrangement of the screws is the best to get the largest design length. This was not as originally assumed but was confirmed by farther investigation into the model. It was found that the most effective area to place connectors if only a few are used are near the ends than in the middle, but if ample amount of connector are used then it is best to spread them out correspondingly to the shear diagram. From this it was concluded that a slightly weighted pattern be used.

Model 2-5 shows that almost 3 feet of design span can be gained if a glue connection is used for the top and bottom flange to web connection. This is a significant amount with a 7.3% increase in length. Because of this an all glued panel could be very advantageous if a screw membrane does not have significant improvement in acoustical performance or if acoustics isn't as important in the floor system. Overall, an all glue solution is recognized to be a completely viable solution no matter what the results are of future acoustical testing.

From Model 2-6 was seen to approximately have a 1 foot increase in design length if no membrane was used. Similar to Model 2-5, this is not enough to change recommendations, but it is recognized to be a viable solution that could be beneficial to the panel's performance if an acoustic membrane does not affect the acoustical performance of the panel as much as predicted.

Results and Predictions

From the previous sections, it is recommended that a HMT panel be made up of a flat EPI bond for the flange to web connection on the bottom and a top connection made with ASSY VG $^{5}/_{16}$"x14$^{1}/_{8}$" " screw placed at 30 degrees through a membrane and that the screws are only slightly weighted which corresponds to the shear diagram of the panel and an all Grade #3 lumber top flange should be used to meet similar performances with a significant cost reduction. Also, an all glue panel was found viable and even was able to achieve longer design lengths, but might be able to be greatly improved upon acoustical with the screw membrane. From this recommended panel, final designs for two representative building occupancies were modeled. These occupancies were a business and an assembly occupancy that had a live load of 50 and 100 psf, respectively. In addition to this occupancy live load, an additional 15 psf of live load was used to account for partitions.

Table 5.4: Final Designs for Business and Assembly Occupancies

Model Number	Model Description		Total Screws*	Vibration	Strength	Live	Long
	Occupancy	Full Depth	[Screws/panel]	[ft]	[ft]	[ft]	[ft]
3-1	Business	$13^3/_4$"	120	31.83	35.76	37.30	38.63
3-2		$16^1/_2$"	126	36.03	40.51	43.23	44.65
3-3		$19^1/_4$"	120	39.83	44.91	48.81	48.78
3-4		22 "	126	43.69	49.71	52.97	51.88
3-5		$24^3/_4$"	126	46.69	54.72	56.07	53.95
3-6		$27^1/_2$"	120	49.22	60.51	58.17	56.01
3-7	Assembly	$13^3/_4$"	156	32.70	28.89	29.95	36.00
3-8		$16^1/_2$"	168	36.90	32.39	34.82	41.72
3-9		$19^1/_4$"	180	40.75	35.97	39.49	46.47
3-10		22"	192	44.49	39.31	44.03	49.72
3-11		$24^3/_4$"	198	47.28	42.32	48.30	52.09
3-12		$27^1/_2$"	204	49.48	45.37	51.79	53.81

*The amount of total screws shown are based on vibration design for models 3-1 through 3-5 and live load deflection for models 3-7 through 3-12

As can be seen, a HMT panel has the ability to span from 30 – 50 feet by changing the depth of the panel and slightly farther if an all glue solution is used. Vibration controls the design of the floor systems with a business occupancy load applied, but strength controls the design of the floors with the assembly occupancy load applied. Since reference stresses for the flange were taken as f_t instead of f_b in order to be conservative, it might be very likely that strength actually does not control the design of the panel. Even if this is not the case, the strength of the panel could be increased by simply using a slightly higher grade lumber. Therefore, it was not considered that strength controlled the design of the panel subjected to assembly loadings. The next design requirement that controlled the design was the live load deflection limit, but this value was only slightly lower and sometimes higher than the strength design length. From this, it can be seen that the biggest difference between a business and assembly occupancy would be the amount of screws needed to be used.

Overall, the spans obtained did show that a HMT panel could fulfill the need for an all-wood long span solution. Span to depth ratios were found to be in the range of 21-28 with an average of around 25. With these ratios along with the fact that MEP can be placed inside the panel, lower floor-to-floor heights can be used and this can help reduce the overall cost of the building. Another way to quantify how well the recommended HMT panel performs is to compare the stiffness and strength relative to a fully composite panel. This was done by plotting the relative stiffness and strength for a wide range of connector stiffnesses by varying the actual connector stiffness values at the recommended distribution of connectors and then plotting where the recommended HMT panel was on this curve. This can be seen in Figure 5.3.

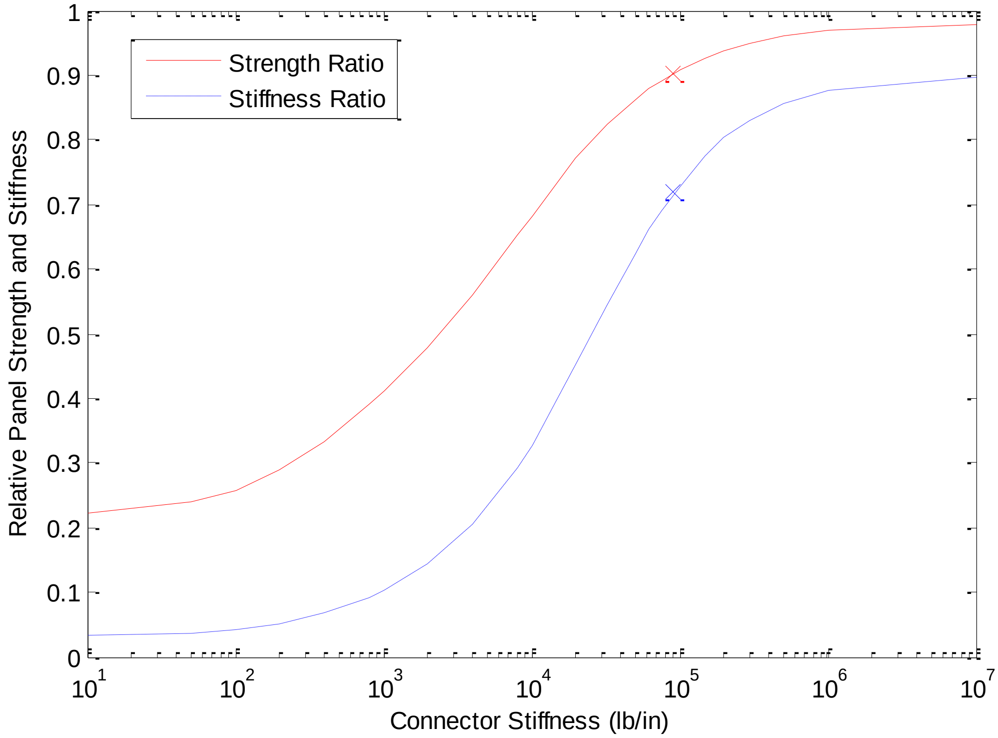

Figure 5.3: Partial Composite Action Curve

As can be seen, the recommended HMT panel is on the higher side of a composite action curve that is good, but does have potential to gain even more stiffness and strength before the curves level out because of unchangeable shear deformations in the wood. If a more fully composite panel is desired, a glue connection or a screw connection without a membrane was used for the top connector. This corresponds to Table 5.3 that showed that 3 feet of span length for a glue connection and 1 foot of span length for a screw connection could be gained if implemented. Also, from Figure 5.3 it was found that stiffness is reduced more than strength because of the connector stiffness, but it must be remembered that by making a hollow panel instead of a solid panel, stiffness gain was more than twice as much as strength gain. Furthermore, there is the potential that by using the screws with a membrane connection, more damping could be introduced in a HMT panel and therefore increase the vibrational performance and act against the negative effect the membrane has on stiffness.

Modeling of Fire Design

With a final recommended HMT panel, a predicted fire design wanted to be investigated to see the possible fire performance of the panel. Therefore, the NDS method for fire design was used which is a mechanics-based design method. This method is used for fire design calculations by using a char rate to reduce the cross section of the member for a given amount of time needed to be met for fire resistance. With the left over cross section, normal engineering strength calculations are performed, but an increase in strength values to the average is allowed.

It has been observed in CLT tests done by FPInnovations in Canada for development of fire resistance calculation methodology of CLT that an increase in char rate was caused by failure of the adhesive when a temperature of 550 ^{o}F was reached (Osborne, Dagenais, and Benichou). These tests were done with the widely used structural polyurethane adhesive (PUR) and therefore, the accelerated char rates could be related just to this type adhesive which was not used in this research project. For calculating the effective char rate, Chapter 8 Section 4.1.4.1 of the USA CLT Handbook was followed which used the accelerated (and conservative) char rate (Dagenais, White, and Sumathipala).

Figure 5.4 shows the results of a modeling the reduced cross section of the bottom flange in the SAP2000 model. From this, it was found that a HMT panel could achieve a fire resistance of 1.296 hours on its own, without any encapsulation from gypsum wallboard. Therefore, this could be improved even more by putting fire rated gypsum wallboard over it. Once the fire burns through the two parallel to span layers, there is a significant change in strength, because there would be no more flange to help resist the loads since the crosswise boards are not adding strength in the parallel to span direction. Therefore, a panel without a bottom flange was investigated to see if a HMT panel would have enough strength to not collapse after the point of losing its bottom flange. This could allow the panel to achieve a fire resistance of more than what was achieved with just the charring of the flange. Models shown in Figure 5.5 represent this situation and were used to determine if a HMT panel would have enough strength after the bottom flange was gone.

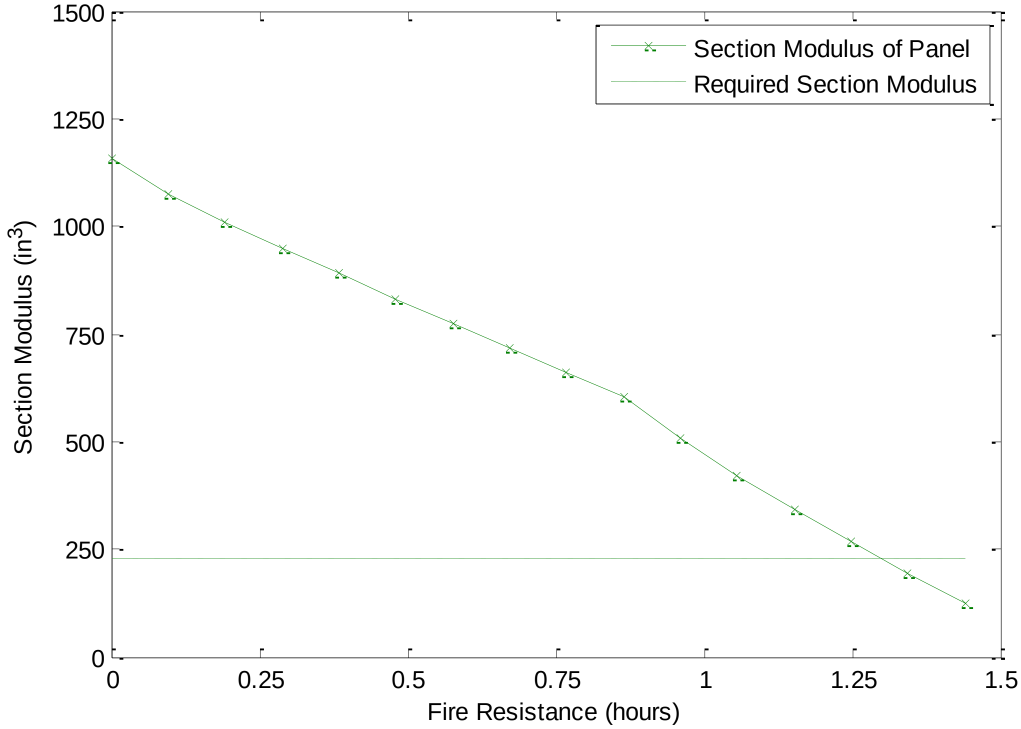

Figure 5.4: Design of Panel for Fire Resistance

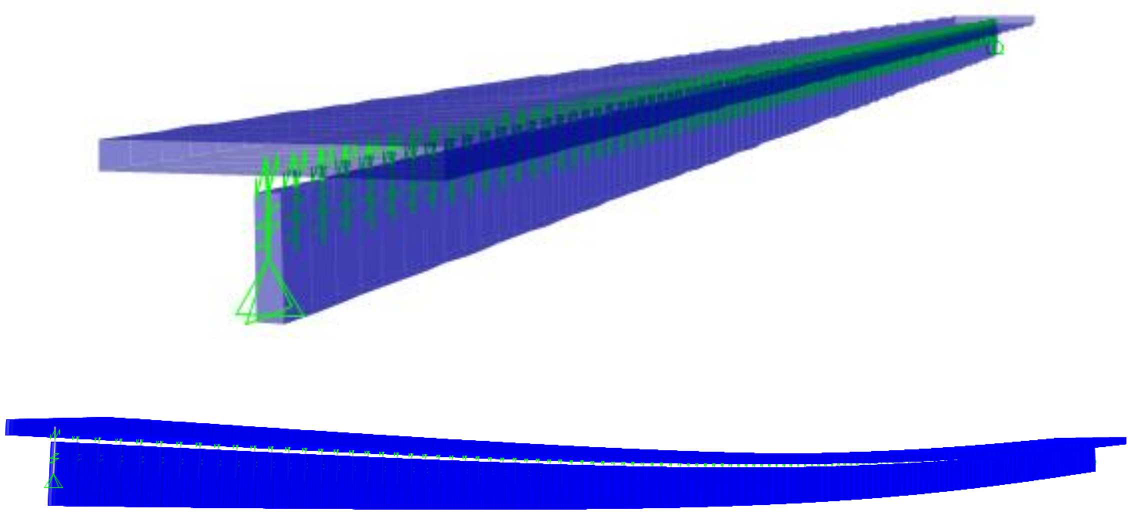

Figure 5.5: SAP2000 Model of Panel Remaining After Losing Bottom Flange

From this analysis, it was found that under ASD loads, a 40-foot long panel with the typical 11-inch deep by 2½" wide glulam would need to be able to handle a bending stress of 4832 psi when adjusted with the 2.85 factor that is allowed to adjust to average member stress value. This requirement would not be met with the all Grade #2 lumber glulam that has been assumed for as the web throughout this research, but by substituting a very common Southern Pine glulam, these values can be obtained. For example, a 24F-V4 glulam combination would be able to handle these stresses. The calculation can be simply done by multiplying 2400 psi by 2.85 for a product equal to 6840 psi, which is greater than 4832 psi. This calculation follows Chapter 16 of the NDS and assumes that the glulam is uncharred because of being protected by the bottom flange. For this calculation C_L was taken as unity because of meeting the requirement of NDS 3.3.3.3 that controlled over using the calculated C_V value that was larger than unity. Because it would take an additional 0.884 hours to burn completely through the bottom flange after the failure of the bottom flange. This would allow theoretically for 2.18 hours of fire resistance if only the glulam and the top flange can resist the loads. Further time for fire resistance could be reached if charring of the exposed web was considered. Therefore, from this calculation it could be practical to have an upgrade of a HMT panel to higher strength glulams, if a 2-hour fire resistance rating was required. Large-scale fire tests will need to be conducted to insure the ability to get to 2 hours because the bottom flange could fall off earlier than anticipated because fire could breach the remaining bottom flange before it is completely charred. This would allow fire to get into the hollow portion earlier and could cause a more rapid decline. This is great fire resistance of a wood product when considering that this is without any extra fire resistance added by encapsulating with gypsum wall board or other methods.

Conclusions

In this Chapter, the SAP2000 model used in Chapter 3 was combined with test data presented in Chapter 4 to predict full-scale performance of HMT panels. To do this, the model was upgraded to run a non-linear analysis because bi-linear models obtained from the connections were used. These methods used for the flange to web connection model were then checked by a model representing the shear tests and were validated for use for a full-scale panel.

A series of screws placed in the top connection using the recommended membrane with different size, orientation and length were placed in the model to find which screw resulted in the most effective and economic panel. From this study the recommended screw was found to be the ASSY VG $^{5}/_{16}$"x14$^{1}/_{8}$" screw placed at 30 degrees through a membrane.

Another study was done on several other variables and it was concluded that a top flange made of all Grade #3 lumber could provide similar performance while reducing costs significantly. Also, it was concluded that a slightly weighted spacing of the screws along the span of the beam was best because a higher effective stiffness was obtained which resulted in better performance of the panel. Furthermore, it was found that if a glue or a screw with no membrane was used as the top connection, an increase in design span could be gained, and could be considered viable solutions if tested acoustic performance of a screw through a membrane was not as high as expected.

Therefore, the final recommended panel uses a flat EPI bond for the flange to web connection on the bottom and a top connection made with ASSY VG $^{5}/_{16}$"x14$^{1}/_{8}$" screw placed at 30 degrees through a membrane with the screws placed in a slightly weighted arrangement which corresponds to the shear diagram of the panel and additionally uses an all Grade #3 lumber top flange which performs similarly buy could have a significant cost reduction. Also, it was found that an all glue panel could achieve slightly greater design lengths and would be recommended if acoustic testing of the screw membrane proves to not have much acoustical benefit or if the benefits of a screw connection at the top are not desired. Throughout the rest of this thesis, the screw membrane will be considered as the recommendation for simplicity sake.

The composite action curve for the panel was graphed and was found that the connectors recommended resulted in a high amount of composite action. Different depths were considered when using the final recommendations and it was found that a range of 30- to 50-foot spans could be obtained. With these results it was concluded that the recommended panel meets requirements that are needed for a long span alternate to CLT.

The fire performance of this panel was lastly investigated to predict possible fire resistances of a HMT panel. It was found that a 1-hour fire resistance can be achieved with no modifications to the panel and a 2-hour fire resistance is possible if the glulam web is upgraded to a standard Southern Pine glulam instead of a glulam that uses all Grade #2 lumber. These were viewed as very beneficial to since the fire performance of the structural elements is very important in buildings.

CHAPTER SIX

DESIGN IMPLICATIONS

Introduction

There are many other topics besides structural strength that affect the performance of a floor panel. These topics are of utmost importance and can have large effects on the success of the product. This Chapter outlines these topics and how they relate to the performance of the recommended HMT panel. These topics are: Fire, Vibration, Camber and Long-Term Deflections, Mechanical Electrical and Plumbing, Acoustics, Sustainability, Costs, Connection Details and Constructability. This chapter will address both the advantages and challenges associated with HMT panels in these various areas.

Fire Performance

During the preliminary stages of this research, discussions were held on whether a hollow massive timber system would be worth studying because of questions related to its fire performance. The concern stemmed from the combination of a combustible material and concealed spaces within the panel. Even if adequate fire performance of the panel could be proven, current building codes may still restrict its use and fail to capitalize on its full potential in construction. The industry viability of HMT panels will depend on a combination of fire-testing and building code advancement.

In the forthcoming 2015 version of the IBC, CLT, which is comparable to HMT panels in many ways, is able to be classified under three types of construction. It can be classified as Type III, Type IV Heavy Timber (HT) or Type V construction. Type IV HT is the most desired construction type for CLT because it has the largest allowable building heights and areas with the least fire rating restrictions. This type of construction takes into consideration the better performance obtained when larger timbers are placed in a fire. Large timbers form a char layer, which helps insulate the wood and slow the burning process. This delays the point of failure in a heavy timber building and improves the overall fire performance of the building. The heavy timber classification requires a certain minimum width, depth or thickness dimension of the structural members whether it is for walls, floors, or beams. In general there are no specified hour requirements except for the exterior wall, which has to have a 2-hour fire rating. There is also a requirement to have a 1-hour fire rating with any non-heavy timber partition walls. The exterior wall traditionally has to be built with a non-combustible material like CMU, but in the proposed 2015 IBC this requirement has changed to allow a 2-hour fire rated CLT assembly to be used as long as fire-retardant sheathing is placed on the exterior of the wall. Heavy timber also requires that there be no concealed spaces in the building. This is to prevent undetectable fire-spread. The HMT panels tested and presented in this document have internal voids that could be categorized as concealed spaces under the present codes and would therefore be prohibited for use in Type IV construction.

Type III construction allows for the use of CLT with certain fire rating requirements for wall, floor, frame and partition assemblies. Like Type IV Heavy Timber, the exterior wall has to be built with a non-combustible material however CLT is not allowed in the proposed 2015 IBC to be used to fulfill this requirement. When compared to Heavy Timber, Type III construction has slightly more restrictive height and area limitations as well as some other more restrictive fire-rating requirements. There is no restriction on concealed spaces within Type III construction, and therefore, HMT panels would be acceptable for use under this classification. That said, the height and area restrictions, combined with the requirements for non-combustible materials in exterior walls, would undermine the desire for large and versatile all-wood buildings.

Type V construction allows for any type of material and maintains either low fire-rating requirements or none. Because of this, Type V is very limited on allowable building heights and areas. Again, these size restrictions place practical limits on the potential of HMT and other massive timber panels.

From intuition, we are led to believe that the concealed spaces in a HMT panel will perform better than other situations involving concealed spaces. Some of the reasons include:

1) All of the elements surrounding the concealed spaces in HMT panels are made of massive timber, in this case CLT and glulam. These elements would char and resist catching fire in the first place.

2) Supposing that blocking were installed between webs at the open ends of each panel, the concealed spaces in HMT panels would be very well compartmentalized and therefore limit fire-spread, which is a main concern related to concealed spaces in heavy timber buildings. This blocking could be either CLT or glulam material.

3) Fire inside a concealed space within the panel may be self-extinguishing because of the limited air flow.

4) Even if fire were to breach the bottom panel around holes in openings for sprinklers or HVAC, the fire still could not spread out of the concealed space.

5) Because of the HMT panel's excess strength, fire will not have as much effect on the panel's ability to support necessary loads.

Redundancy is another positive quality provided by HMT panels and other massive timber products like CLT. There is a lot redundancy in both the panels and in the overall building. The reason for this is because CLT, whether it is by itself or used as flanges in a HMT panel, has strength in all directions. It can resist large in-plane loads along with having the ability for two-way action that can resist out-of-plane bending in both directions. If one part of the panel is burned, usually the load path can be changed in order that the loads can still be resisted. The same is applicable to the whole building. If one wall fails, the structure will still resist collapse because a wall above or below can act as a deep beam to resist the loads. This redundancy should always be considered when discussing fire performance of a massive timber building, because it will increase the performance of the system in an actual fire scenario.

It is believed that the proposed HMT panel addresses many of the traditional concerns with concealed spaces, because of the attributes described above, but there is an additional issue that could cause complications. Depending on the routing of the HVAC system, some duct penetrations may be unavoidable. If not handled properly, these penetrations could undermine the integrity of a concealed space compartmentalized with massive timber. Some suggested solutions include:

1) Group together most of the ductwork and other MEP into discrete areas in order to limit penetrations to certain spots,

2) Install fire blocks and dampers,

3) Fill all questionable voids with mineral wool insulation so that fire could not spread into or out of these spaces,

4) Specify simple, reliable and cost effective ways to seal around penetrations (such as expanding foam strips).

These are each ways to improve upon the fire performance and help ensure that fire risk can be minimized so that the system could perform equally or better than non-combustible construction.

In addition to these instinctive measures, a discussion with Dr. Robert White of the Forest Products Laboratory, provided three clear-cut options for how HMT panels could meet fire code. The first is to fill the entire hollow area with insulation so as not to qualify as a "space". Mineral wool would be a good option for this because of its extremely high heat resistance. The second option is to classify it as a Type III or Type V construction and accept the restrictions of those construction types. These first two options would allow for the immediate use of this system. The third option is to obtain approval through the building code. This could be on a case-by-case basis through local project review or through ICC evaluation reports, or it could happen through evolutions and changes in the building code itself (R. White). This third option would require extensive and recognized fire testing. Even the case-by-case approval of a local building official would need to be supported by some fire test data on the panels.

In summary, future fire testing will be crucial for definitively measuring the fire resistance rating and overall fire performance of a HMT panel. This is especially important for the effects of concealed spaces within HMT panels on the fire performance of a building. Tests may show that it is difficult for fire to begin inside the voids of the panel. That said, what happens if a fire does get inside the panel? Is the thickness of the timber components sufficient for restricting fire spread? Tests of different details could show which approaches achieve the best performance. Ideally, an interactive process would ensue and fire test results would lead to improvements and fire performances that are equivalent to code standards for 1-2 hour rated non-timber systems.

As mentioned previously, the 2015 proposed version of the IBC will recognize CLT within the Type IV Heavy Timber classification. This is helpful for advancing the acceptance of massive timber, however, it is not sufficient for taking full advantage of these systems. The heavy timber classification allows for greater building heights and areas because it ensures that structural elements have a certain minimum cross section, but the HT classification is not detailed enough at this point to account for the higher fire ratings that can now be obtained by CLT and other massive wood products like HMT panels. It is important to test and understand the full extent of the fire performance capabilities of massive timber panel because greater knowledge is the first step toward broader and deeper code acceptance.

This broader code acceptance will also need to contain a more nuanced approach to the topic of concealed spaces, which is not clearly defined in the 2012 IBC ("2012 IBC"). Ideally the codes would distinguish between specific types of concealed spaces along with their specific hazards so that consistent performance in all types of construction can be ensured and so that there is more flexibility for massive timber buildings in particular.

Vibration

Vibrations are known to limit the design of CLT and have been found through computer modeling in this research to control in most situations for which HMT panels would be designed. Since this is often a limiting factor, it is very important to quantify the panel's vibrational characteristics to ensure adequate performance to meet occupant's satisfaction. There are several critical assumptions that were made when calculating the vibration design length for HMT panels in this research. One of these assumptions was that a HMT panel could be described with the same equation as a CLT panel. This assumes that these two panels act the same with respect to vibrations, which might not be exactly correct. Some possible differences in the vibrational characteristics could stem from a greater damping in a HMT panel, the much longer spans used with HMT panels, or because of the hollow space, which results in a much smaller mass to stiffness ratio when compared to CLT.

Therefore, experimental vibrational testing is recommended to either confirm that the CLT vibrational equation accurately describes a HMT panel or, if not, to suggest the modifications that would need to be made for an equation specific to HMT. A potential test would involve people walking across different panels at different spans to see which design lengths prove satisfactory to building occupants (Hu). Other testing methods could include more detailed computer modeling and a vibrational analysis of the panel using accelerometers. It is also recommended to test HMT panels without the resilient acoustic membranes to see if this makes any impact on the vibrational characteristics of a panel.

Camber and Long-Term Deflections

It was predicted that long-term deflections would measure better than required minimums because they did not control the design of the panel. This is very beneficial because a higher performance in this area is especially desirable for a long span system in which as much as 2 inches of long-term deflection is allowed for a 40-foot panel span. Cambering of the panel could be an option to obtain even higher performances.

For cambering the panel the shoring would be set to a length that raised the middle of the panel by the desired amount of camber in reference to the level of the supports at the ends of the panel. This would arch the bottom section of the panel up by the desired amount while allowing for workers to be supported on top of the panel. The bottom section of the panel would consist of the bottom flange and the glulam web bonded together with the recommended EPI flat bond. By having just this bottom section, a cross section of lesser stiffness is produced, which allows for easier flexing of the panel to achieve a normal amount of camber. The camber of the completed panel would get locked-in when the top flange is placed and fastened with screws to the cambered webs and bottom flange. In comparison to normal flat panels, this construction procedure would be similar except for setting the required shoring higher to allow for the camber.

One other difference caused by cambering would be the slight pre-stresses resulting in the panel. For example, extra compressive force will be added to the top flange because of its resistance to the straightening of the panel. Also, since the bottom portion is bent, a tensile stress will be located at the top of the glulam and a compressive stress will occur at the extremity of the bottom flange.

Mechanical, Electrical and Plumbing Systems

Locating mechanical, electrical and plumbing (MEP) systems is a very important topic when dealing with residential and commercial buildings. Typically, MEP is designed to be hidden (for aesthetics and protection) within wall and ceiling systems. Since CLT is a solid element, this is more challenging than in traditional wood construction where joists or wood trusses are used and provide cavities for ductwork and piping. Potential solutions for CLT include:

1) Small channels for conduit or piping could be routed into the panel.
2) CLT walls can be furred out to accommodate MEP running behind the finished surfaces.
3) MEP could be routed through stick-framed partition walls used in the interior of the structure.
4) Drop ceilings could be used.
5) MEP systems could run through gaps left between the ends of adjacent wall panels.
6) Raised floors systems could be utilized above the CLT to hide or protect MEP, or even to act as a pressurized air plenum.

While these options do provide solutions for the placement of MEP, they also add constraints and complexities to installing MEP systems.

HMT panels improve upon CLT significantly in this area. The hollow voids in HMT panels allow for MEP systems to be easily routed within the thickness of the panel itself. This is beneficial because there is no extra material that needs to be added to cover up MEP systems and there is no additional height added to the building to accommodate drop ceilings or built-up floors. A HMT panel can accommodate electrical and communication wiring, plumbing and sprinkler pipes, as well as air ducts, all within the voids.

However, some complexities do arise when running these systems inside the panel. First, the size of the potential air ducts would be limited to the dimensions of the void spaces. The voids in the HMT panels measure 11”x32”. Larger air ducts would need to be rectangular in order to fit. The void size is thought to be large enough to fit ducts capable of servicing the area of multiple panels and therefore seems adequate.

There is also the possibility of running branching air ducts perpendicular to the span through the webs of the panels, but this would involve some restrictions. In this case, the duct would need to penetrate the center of the panel where the shear in the web is much less than near the ends. It is predicted that these areas would also have to be reinforced with self-tapping screws, especially if the holes were large. Since there is no equation given for this specific situation, tests would need to be performed to see how the size of the hole and the reinforcement affect the strength of the web.

Another related restriction is that any supply and return vents need to be placed away from the mid-span of the panel so that the bending strength is not decreased. Also, while vent holes located in the bottom flange of the panel can certainly be allowed, it would be better if vents were placed in the top flange and up through the floor.

A final related issue is maintenance. Because access to the hollow portions of the panel would be desirable for routine MEP maintenance or additional services, access openings will need to be strategically placed along the panel. These openings could be pre-installed or carefully prepared in the field. This is another advantage to working with a soft material like wood.

Acoustics

Acoustics are becoming a larger part of building design. CLT has been shown to meet acoustic demands, but it often requires various specific measures and assembly configurations. The HMT panel is likely to offer improved acoustic performance, which could be a substantial benefit when compared to a solid CLT panel.

There are a couple of main reasons that a HMT panel would offer better performance. The introduction of a hollow air space in the panel provides for better damping of air-borne sound. These sound waves would not be transmitted directly through a panel, but rather they would have to pass through the void spaces or through the limited area of the web members. Secondly, the screw connection that utilizes an acoustical rubber membrane is thought to dampen the impact noise vibrations through the panel by isolating the top flange from the rest of the panel. If code-required acoustical performance could be achieved by the simple integration of acoustical membrane strips then other measures such as raised floors or drop ceilings, which are associated with CLT construction, could be avoided, saving time and money. This solution would also preclude the use of concrete as a floor topping material, helping to keep ensure a lower carbon footprint. Direct acoustic testing of HMT panels is required to demonstrate code compliance with both air-borne noise (STC rating) and structure-borne noise (IIC rating).

Sustainability

One of main benefits of CLT is how sustainable it is compared to other common building materials. The same would be true for HMT panels as well. The key aspects that make massive timber panels sustainable are:

1) Wood is used as the construction material
    a. Wood takes less energy to harvest and manufacture to make a final product
    b. Wood is renewable and massive timber panels use fast growing, smaller diameter trees
    c. Wood naturally sequesters carbon from the environment
2) Massive timber uses environmentally friendly adhesives.
3) Massive timber can be a recycled.
4) Massive timber can help reduce the long-term energy consumption of a building by making an air tight building envelope (through fine tolerances) and by adding natural thermal resistance to the wall assembly itself.

These and other sustainable benefits of wood are discussed in detail in Chapter 2. That said, it is helpful to note presently some of the specific sustainability-related aspects of HMT panels that might be different than CLT and other similar products.

Two adhesives were used in this research. These were Melamine Formaldehyde (MF) and Emulsion Polymer Isocyanate (EPI). MF does included formaldehyde, which is known to be a carcinogen. However, tests have shown that the minimal levels of off-gassing from the product are not substantial enough to be harmful. The specific MF used in this research is Greenguard children & schools certified and indoor air quality certified ("Greenguard Certification"). Using non-formaldehyde glue like EPI has the benefit of being formaldehyde-free, but the Isocyanate in EPI is a hazardous chemical that does have to be handled properly before cured. With either of these adhesives there are no exposure issues over the service life of the structure.

In comparison to other wood products, HMT has some added benefits for sustainability. No concrete is needed for a HMT panel unlike a hybrid wood system (such as the HBV system) and unlike CLT, which might have to be topped with concrete for acoustical isolation. It is good that HMT does not use concrete because concrete would increase the carbon footprint of a wood panel significantly. Also, concrete would decrease efficiency in design because it would add unnecessary dead weight added to the panel. Moreover, in place of concrete, more wood is used. This allows for greater carbon sequestration. Another benefit compared to similar products would be the efficiency of the system. By using a more efficient design, long-span HMT panels get the equivalent strength of a solid panel but use substantially less material by volume.

Cost

Cost is an important topic because it plays a large role in the type of building system selected for a building. Even though massive timber buildings offer many benefits over other building types, cost competitiveness will still be a critical deciding factor. It has been shown that CLT can be cost competitive but it has also been shown to have some inefficiencies and may therefore have cost overruns compared to other methods. (Winter et al.). In research performed by Vienna University of Technology in Austria, it was shown that the main issue for optimizing costs in CLT buildings was choice of floor elements and their span lengths. It was suggested that costs could be reduced if rib slabs or more efficient floor structures were utilized instead of thicker CLT slabs (Winter et al.). This information suggests that a more efficient floor section like a HMT panel can have a large effect on whether a massive timber building is cost competitive.

HMT panels offer many benefits that support potentially lower building costs. One of these benefits is that it is an efficient, long-spanning system that requires less wood as a solid element of equal span. Also, HMT panels are even more efficient then a ribbed slab.

The excessive number of bearing walls in a standard CLT system is another source of cost inflation according to research performed in Austria (Winter et al.). Because HMT panels are a long-spanning solution, fewer structural CLT walls would be needed to support the same amount of area, increasing the efficiency of the entire system.

Finally, HMT panels could offer savings in the areas of acoustical treatment and MEP concealment. It is believed that both acoustical isolation and MEP routing would be handled within the HMT panels themselves and not require raised floors or drop ceilings.

Connection Details

Connection details are important for many reasons. One of these is that the connections have to be able to handle structural loads. Also, the connections have to ensure non-structural performance in many areas that have been discussed throughout this document. Moreover, connections for HMT panels need to have the flexibility to work with other massive timber systems, such as CLT and post and beam glulam configurations. The goal of the connection design was to keep the details simple and as similar to common CLT connections as possible.

Concerns arose when considering a platform framing scenario as is typical for CLT. The first concern related to the amount of shrinkage that would need to be allowed for panels that, at average size, would have a total wood depth of 19 ¼ inches, all loaded perpendicular to the grain. This would result in a magnification of an already considerable problem faced by wood some structures. Concerns over shrinkage would only increase when wood buildings are getting taller and taller.

The second issue would be the excessive number of connectors required to transmit shear through the assembly, starting from the wall to the top flange to the blocking to the bottom flange and then to the lower wall. Therefore, the platform framing system was modified to address these issues. This is illustrated in Figures 6.1 and 6.2.

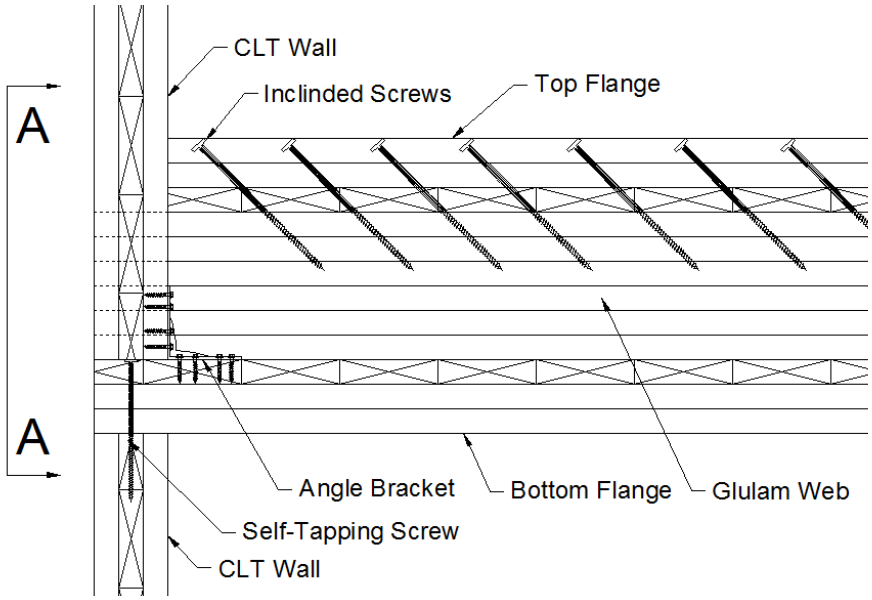

Figure 6.1: Modified Platform Framing Detail Side View

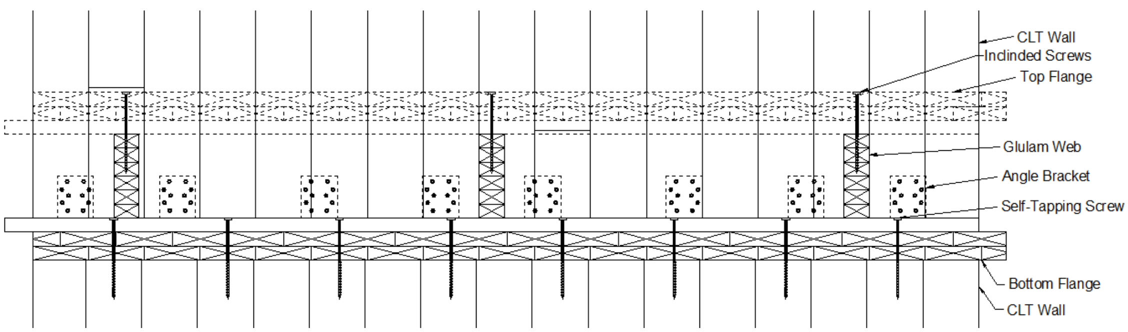

Figure 6.2: Elevation Section A-A

The modified platform framing detail shows that the top wall does not sit on the top flange of the HMT panel but runs past it and rests only on the bottom flange. Also, it shows that notches are cut into the upper CLT wall panel to fit around the web members. This configuration allows for the connection to be very similar to a simple CLT connection while solving the issues discussed above. Since the top and bottom wall are bearing on the bottom flange, there is only $4^1/_8$ inches of thickness of wood perpendicular to the grain that can shrink and affect the height of the overall building. Also, fewer connectors are needed because the shear loading on the building only has to be transmitted through the bottom flange. This configuration also allows for the shear in the glulam to bear onto the bottom flange, which is able to spread out the load before bearing on to the bottom wall. An additional advantage to this detail comes from being a bearing connection in which connectors are naturally concealed in a fire.

Another potential connection detail was investigated for a balloon framing system. This detail utilized a connector placed on the end of the glulam web member which was used to attach the panel to a continuous wall. Examples of the connectors that could be used for this detail include the Sherpa connector (see Figure 2.11), Knapp connector, or a set of three inclined screws similar to the screws used for the flange-to-web connection. The benefit to using a Sherpa or Knapp connector is that they are preinstalled on the wall and at the ends of the webs so that the two pieces can easily interlock when placing the floor element in the field. The field installation would not be quite as fast or seamless for the inclined screw connection, but the material costs would be less.

Some benefits that a balloon framing scheme would offer include the elimination of shrinkage because no load-bearing boards would be loaded perpendicular to the grain. Also, there would be no connectors needed to transmit shear loads from the floors through the panel at each floor level. This would result in a cleaner detail. Shear from the diaphragm would still need to be transferred into the walls, though, and therefore it is recommended to use something similar to the steel angle and lag screws used in Test 21 in Chapter 4 for connecting the bottom flange to the wall. One potential negative of this detail is that it is not a bearing connection, and the connector, therefore, has to be concealed from fire. Methods for doing this are provided by the manufacturers of the Sherpa or Knapp connectors. These connectors can be inset into the ends of the glulam beams and self-adhering fire-protective strips can also be incorporated. These strips expand in a fire to protect the connection. A similar treatment could be used along the bottom edge of the panel for further protection.

The connections described above could also be used to connect HMT panels to load-bearing glulam framing member. The HMT panels could bear on top of glulam beams connect into the side of beams. One benefit of connecting to the side is that the reduction of unnecessary structural depth which could allow for more sun light and potentially smaller floor-to-floor heights.

Constructability

Constructability was also a large consideration throughout this research. This includes both the manufacturing process and construction process. This section presents potential scenarios for both manufacturing and construction.

The manufacturing process would begin similarly to that of CLT panels. First, the flanges of the HMT panel would be made to the PRG-320 standard (ANSI/APA), just like CLT except that the layup of the flanges would have two adjacent parallel layers next to each other and one crosswise layer not placed in the middle. These panels would be cut by a CNC machine along with any pilot holes for fasteners or larger holes for MEP systems.

Glulams made of 2x6 lumber would be manufactured at a high-output plant and then cut in half lengthwise so that two glualms of at least 2½ inches in width are produced. These glulams are then adhered to the bottom flange via the recommended EPI flat glue connection, which is clamped with screws or other mechanical devices. It is important not to allow the bottom flange to sit for too long before being bonded with the glulam because if moisture is lost from the panel, the crosswise boards tend to curl and undermine the good flat surface required for bond. It is also suggested that blocking be used to align the glulams in the correct position and stabilize them while the adhesive cures. Or, in the best case, a consistent formwork would be built for this.

If prefabricated connectors such as Sherpa or Knapp are used, then the connectors would need to be pre-installed at the ends of the glulams before the panels are shipped to the construction site. The bottom flange and glulam assemblies can be transported by truck right-side-up so that they can simply be lifted right off without having to be flipped on site. To maximize shipping efficiency, a certain amount of top flanges should also be loaded onto a truck. If this was not done, the trucks with only bottom sections would be limited by volume of the panels and the trucks with the top sections would be limited by weight and this would result in substantial inefficiency. During erection, the top flanges loaded with the bottom sections would be used to place on top other panels already erected and ready to be completed at the construction site.

The construction process starts with the supporting walls or frame erected and ready for floor panels to be set. Also, shoring has to be provided to support the middle of the bottom sections of the panels that are laid first in the sequence. Once the shoring is ready, then the bottom flange and glulam assemblies can be then lifted from the truck and placed consecutively beside each other.

At this point the unique part of this erection process can take place, and the MEP systems are installed in the open panels. It would not be expected that the erectors and MEP subcontractors are very close to each other, but rather they would be separated by working on opposite sides of the building in order to limit interference. This will require the proper construction scheduling and sequencing to ensure efficiency. After all of the MEP systems are installed and inspected, the top flanges can be set and connected to the tops of the HMT webs.

Since, much like CLT, this is largely a prefabricated system, it is expected to have fast construction speeds. However, since installing HMT panels would be a two-step process, the time required to erect the structure might be longer, especially considering the integration of the MEP systems. On the other hand, because of larger panels being installed, more square footage could be installed quicker than using smaller CLT panels. The coordinated installation of MEP might also save time on the back end, when traditional installation would occur. It is also possible to pre-install some of the MEP systems off site in the factory in order to speed up on-site construction and provide better quality control.

Conclusions

Many properties of a successful floor system have been considered when developing this new HMT panel. Because of this, many problems have been solved simultaneously. This was critical because most of these properties interact with each other, and if they were not considered simultaneously, many areas might have been negatively impacted. HMT panels offer a variety of improvements over CLT. Some of these improvements include:

1) An optimized cross section designed to increase stiffness and decrease vibration enabling longer spans,

2) Integration of MEP into the void spaces of the floor panel,

3) Improvement of acoustical performance by the introduction of void spaces and through the integration of an acoustic membrane at the connection between the top flange and glulam web, and

4) Increased potential cost benefits by reducing the net volume of wood used for long spans, by reducing the amount of CLT bearing walls needed in the building, and by eliminating excess materials required for acoustical isolation or MEP concealment.

Emphasis was placed on retaining many of the other natural advantages of CLT. These included:

1) Retaining the fire performance of CLT panels by ensuring 1- and 2-hour fire ratings and suggesting solutions with respect to concealed spaces

2) Maintain the sustainability of an all wood system

3) Preserve the simplicity of construction details

4) Maintain the speed of on-site construction

HMT panels provide an all-in-one long-span system. This stems, in part, from the integration of acoustical measures and MEP systems. Therefore, these integrative benefits must be considered when assessing the feasibility of HMT because the structural costs by themselves could seem prohibitive. Overall, the HMT panels presented in this document have much potential if the measures described in this chapter are followed and further developed.

CHAPTER SEVEN

CONCLUSIONS

Since the development of CLT, there has been a large surge in interest in massive timber buildings. By discovering the potential of CLT, the design community is considering 30 or more-story tall timber buildings, proving that wood can compete with other building materials in the commercial building industry (MGB Architecture and Design et al.). Many other new products have been sparked by CLT over the last decade, and are helping to overcome some of the traditional performance limitations of wood. Along with searching for the potential of “wood” skyscrapers, there is another trend in research that can be identified – the need for a long-span massive timber floor systems. Unfortunately, solid CLT panels become structurally inefficient when spanning more than 25 feet due to the required thickness of the panel. With most commercial building owners desiring spans of 30 feet and greater, there is a need for an alternative wood-based long-span solution. An economical long-span solution could help to further penetrate the commercial building market with massive timber. The Hollow Massive Timber (HMT) panels presented in this document are conceived as a long-span timber system that could be a solution to this problem.

A literature review pertaining to massive timber products, connections and long-span systems was conducted. From this, a thorough knowledge was gained of topics that were found to have large influences on the success of floor panel design. The shortfalls of other massive timber products and long-span systems were also noted so that they could be improved upon. In particular, there was room for improvement when controlling vibration in floor panels because this is usually a limiting factor in the design of similar massive timber systems like CLT and HBV systems. Therefore, a panel configuration that maximized stiffness was sought. Similarly, fire concerns stemming from panels with internal voids were examined and potential solutions were investigated. In the end, many innovative new ideas and products were studied in hopes of finding potential solutions to the common problems identified.

The decision was made at the beginning of this research to make an offset panel flange. This solution would add to the volume of wood resisting fire through charring and produce a cross section with a greater moment of inertia. When looking at a preliminary analytical model of the hollow panel, it demonstrated larger stiffness gains than strength gains. This was significant because the need for stiffness was the foremost requirement in the design of the panel.

The next step was to perform a parametric study using MATLAB. This study assumed a fully composite cross section with no shear deformations. By testing many different variables for their effects on panel performance, it was determined that the combination of a thinner flange and a deeper overall member, helped narrow the difference between vibration-controlled design lengths and strength-controlled design lengths. It was also discovered that a lower strength board is better because the small reduction in stiffness does not change the design as much. These were the most important findings from the parametric MATLAB study but various others contributed to panel design recommendations as well. Ultimately, it was concluded that a cross section using two layers of Grade #2 Southern Pine lumber parallel to the span with one crosswise board was the most beneficial.

Next, a second parametric study was performed using SAP 2000 model. This time the study considered shear deformations and the stiffness of flange-to-web connectors. It was discovered that the connector stiffness, in comparison to other variables of a panel, had the greatest impact on the overall strength and stiffness. Since connector stiffness largely controlled how the panel would perform, an experimental study was chosen to examine different connectors that could be used for the connection between the flange and web.

The material of the web still had to be selected and various wood products were initially considered. A 2½" wide glulam proved optimal because of its high strength and capacity for compartmentalizing fire.

Using these recommended configurations, an experimental study was performed on the flange-to-web connection by conducting shear tests on the connection with compression loading. Different combinations of glue connections and mechanical fasteners were tested and compared to find the best solution. Stiffness and strength of the connectors were measured and bi-linear models were presented which could be plugged into a non-linear, full-scale HMT SAP2000 model. Ultimately, a 30-degree screw placed through an acoustical membrane was recommended for the top flange connection and a flat bond EPI glue bond for the bottom flange connection. Also, it was found that an all glue panel could achieve slightly greater design lengths and would be recommended if acoustic testing of the screw membrane proves to not have much acoustical benefit or if the benefits of a screw connection at the top are not desired.

The non-linear, full-scale HMT SAP2000 was used to see the difference that various connector combinations made on the overall performance. From these studies, it was found that a panel with Grade #3 lumber, and a slightly weighted screw arrangement should be used. Additionally, the screws used for the top flange connection were optimized and an ASSY VG $^{5}/_{16}$"x14$^{1}/_{8}$" screw at 30 degrees was chosen. The resulting panel achieved a relative stiffness and strength that were high on the graphed composite action curve (see Figure 5.3), and even better performance could be obtained with an all-glue connection or with screws without a membrane. Neither of these other options were ultimately selected due to other non-structural considerations. After all of the final recommendations for the panel were made, different depths were modeled and designed. HMT panels were found to have a predicted range of 30 to 50 feet. Analysis also suggested that panels for business and assembly occupancies could be identical except for the number of screws.

The final recommended panel was also designed for a fire resistance rating. It achieved a predicted 1-hour fire rating by allowing the bottom layer to char. Moreover, if the glulam webs were upgraded from all Grade #2 lumber to a 24F-V4 glulam combination, then a 2-hour rating would be possible. This is because the remaining un-charred cross section consisting of glulams and the top flange, was found to be able to resists the required loads. This was an important finding since these predicted ratings do not include any help from encapsulation of the panel.

In addition to these structural considerations, various non-structural aspects were also discussed. These included recommendations for how to handle fire codes and how to achieve proper fire performance. Other recommendations were made on the following topics: vibration, camber and long-term deflections, mechanical, electrical and plumbing systems, acoustics, sustainability, costs, connection details and constructability. After considering these factors, it was found that HMT panels improve upon many aspects of other massive timber systems. Various assumptions were made in regards to appropriate ultimate stresses, the vibration equation, fire performance, and acoustics and were found to heavily influence the recommendations of this research. These assumptions are described in each corresponding section of Chapter 6. Further testing will need to be conducted to verify these assumptions.

The purpose of this research was to determine whether a HMT panel could fulfill the role of a long-span massive timber floor system. Ultimately, it was discovered that the hollow massive timber panel holds tremendous potential to perform as a long-span solution within the rapidly expanding world of massive timber construction.

Recommendations for Future Work

This has only been a preliminary look at the feasibility of a HMT panel. Now significant testing has to take place in order to confirm, modify or deny assumptions made in this research. Here are the specific topics that need to be investigated in the future in order for HMT panels to advance and become accepted by the construction industry.

- Full-scale beam tests will need to be performed to study bending strength, shear strength and stiffness of the panel. These are critical for verifying the accuracy of the assumptions made in this research. One particular conservative assumption was that the ultimate stresses in the panel would correspond to the tensile capacity of the wood instead of the bending capacity. This may be overly conservative and full-scale testing would allow for panel design values to be modified to accurately describe performance and improve the rated capacity. Through this testing, the flange-to-web slippage would also be monitored so that results from the present connection tests can be validated.
- Fire testing will need to be performed. This is especially important for the effects of concealed spaces within HMT panels on the fire performance of a building. Tests may show that it is difficult for fire to begin inside the voids of the panel. That said, what happens if a fire does get inside the panel? Is the thickness of the timber components sufficient for restricting fire spread? Tests of different details could show which approaches achieve the best performance. Ideally, an interactive process would ensue and fire test results would lead to improvements and fire performances that are equivalent to code standards for 1-2 hour rated non-timber systems.
- Experimental vibration testing is also recommended. This would help to confirm whether the CLT vibrational equation accurately describes a HMT panel or if modifications need to be made. A potential test would involve people walking across different panels at different spans to see which design lengths prove

satisfactory to building occupants (Hu). Other testing methods could include more detailed computer modeling and a vibrational analysis of the panel using accelerometers. It is also recommended to test HMT panels without the resilient acoustic membranes to see if this makes any impact on the vibrational characteristics of a panel.

- Acoustic testing of the panel should be performed to verify the assumed dampening effects of the resilient acoustic membrane. In particular, testing and measuring the impact insulation class (IIC) will be critical.
- Passages for MEP systems will likely be desirable in certain parts of the panel. Therefore, testing should be performed to find recommendations for acceptable placement of these holes. It is predicted that large openings reinforced with self-tapping screws will be required to run air ducts perpendicular to the span through the center of the webs. Since, there is no equation given for this specific situation, tests will need to be performed to see how the size of the hole and reinforcement affect the strength of the web. Similarly, the sizes of holes in the top and bottom flanges should be tested.
- Finally, comprehensive cost studies should be performed which include costs of manufacturing the panel, transportation of the panel to the job site, installation of the panel, and maintenance of the panel over its service life. This is a critical step in understanding the market feasibility of this technology.

APPENDIX

The spreadsheet titled “Moisture Content and Density” contains the following:

- Raw moisture content of all CLT flanges, and glulam webs
- Weight of all CLT flanges and glulam webs
- Dimensions of all CLT flanges and glulam webs
- Calculations of average moisture content, density and dry density of each element

The spreadsheet titled “Shear Test Summary” contains the following:

- Strength and stiffness for each test specimen and their normalized values
- Failure mode of each test specimen
- Additional comments regarding wood failure, sensor errors, visual observation of the quality of the specimen connections, and additional observations

The spreadsheet titled “Shear Test Summary Graphs” contains the following:

- Connector stiffness graphs for each specimen that were plotted with raw data from each sensor
- Average connector stiffness graphed with bi-linear models to summarize the performance of each test
- Comparative summary bi-linear model graphs

REFERENCES

"2012 International Building Code." 2011: n. pag. Print.

"A Profile Of The Glulam Industry." *APA Wood*. N.p., 2014.

AF&PA. *National Design Specifications for Wood Construction*. Washington, DC: American Forest & Paper Association, 2012. Print.

Alex de Rijke. "Flatpack without Boredom." RIBA Research Symposium. Jarvis Hall. 2007.

ANSI/APA. *Standard for Performance-Rated Cross-Laminated Timber*. Tacoma, WA: APA – The Engineered Wood Association, 2012. Print.

ASTM. *Standard Practice for Establishing Clear Wood Strength Values*. West Conshohocken, PA: American Society of Testing and Materials (ASTM), 2011. Print.

---. *Standard Test Method for Strength Properties of Adhesive Bonds in Shear by Compression Loading*. West Conshohocken, PA: American Society of Testing and Materials (ASTM), 2009. Print.

---. *Standard Test Methods for Mechanical Fasteners in Wood*. West Conshohocken, PA: American Society of Testing and Materials (ASTM), 2012. Print.

Atkinson, Cathryn. "Pine Beetles Kill Forests, Builders Seek to Use the Wood." 29 Nov. 2012.

Bejtka, I., and H.J. Blass. "Joints with Inclined Screws." Kyoto, Japan: Universitat Karlsruhe, 2002. Print.

Blass, H.J., I. Bejtka, and T. Uibel. *Tragfähigkeit von Verbin Dungen Mit Selbstbohrenden Holzschrauben Mit Vollgewinde*. Universitat Karlsruhe, 2006. Print.

Chen, Yue. "Structural Performance of Box Based Cross Laminated Timber System Used in Floor Applications." University of British Columbia, 2011. Print.

Closen, Max. ASSY Screw Testing. Sept. 2013.

---. "Self-Tapping Screw Assemblies Under Monotonic and Reverse Cycle Load." University of British Columbia, 2012. Print.

Comite Europeen de Normalisation. "Eurocode 5: Design of Timber Structures." 2004: n. pag. Print.

Crespell, Pablo, and Sylvain Gagnon. "Cross Laminated Timber: A Primer." 2010.

Dagenais, Christian, Robert White, and Kuma Sumathipala. “Chapter 8 Fire.” *CLT Handbook*. US Edition. FPInnovations and Binational Softwood Lumber Council, 2013. Print. SP-529E.

Equilibrium Consulting Inc. “UBC’s Earth Sciences Building (ESB).” N.p., 4 May 2014.

Foit, Jana. “Innovation in Wood Construction.” 2014.

“Greenguard Certification.” *AkzoNobel*. N.p., 2012. Web. 20 July 2014.

Hu, Lin. Cross Laminated Timber. 28 June 2013. E-mail.

Hu, Lin, and David Adams. “Chapter 9 Sound.” *CLT Handbook*. US Edition. FPInnovations and Binational Softwood Lumber Council, 2013. Print. SP-529E.

Hu, Lin, and Ying-Hei Chui. “Chapter 7 Vibration.” *CLT Handbook*. US Edition. FPInnovations and Binational Softwood Lumber Council, 2013. Print. SP-529E.

Kretschmann, David. *Chapter 5 Mechanical Properties of Wood*. Madison, WI: Forest Products Laboratory, 2010. Print. Wood Handbook, Wood as an Engineering Material.

“Mass Timber Products.” *Mass Timber*. N.p., 2013.

Mestek, Peter, and Stefan Winter. “Cross Laminated Timber (CLT) - Reinforcements with Self-Tapping Screws.” Trentino, Italy: N.p., 2010. Print.

MGB Architecture and Design et al. *The Case for Tall Wood Buildings*. N.p., 2012. Web. 4 May 2014.

Osborne, L., C. Dagenais, and N Benichou. *Preliminary CLT Fire Resistance Testing Report*. Ottawa, ON: FPInnovations, 2012. Print.

Pang, Weichiang. “Introduction to Wood Characteristics and Engineered Wood Products.” 2014.

Pirvu, Ciprian, Bradford Douglas, and Borjen Yeh. “Chapter 6 DOL and Creep.” *CLT Handbook*. US Edition. FPInnovations and Binational Softwood Lumber Council, 2013. Print. SP-529E.

Quenneville, Pierre, and Hugh Morris. “Japan Kobe Earthquake Shake Table Simulation.” *NZ Timber Design Journal* 15.4 n. pag.

Rhomberg Group. “Cree Systems.” *CREE*. N.p., n.d. Web. 4 May 2014.

Rizo, Michael. *Cross-Laminated Timber and Gypsum Board Wall Assembly (Load Bearing)*. Leesburg, VA: NGC Testing Services, 2012. Print.

Ross, Loren, Sylvain Gagnon, and Edward Keith. “Chapter 3 Structural.” *CLT Handbook*. US Edition. FPInnovations and Binational Softwood Lumber Council, 2013. Print. SP-529E.

Schraubenwerk Gaisbach GmbH. “Innovative Application for ASSY plus VG Wood-Concrete Composite Slab.”

SFS Intec, Inc. “WT Fastening System Brochure.”

Skidmore, Owings and Merrill, LLP. *Timber Tower Research Project*. N.p., 2013. Web. 4 May 2014.

“Specification for Structural Steel Buildings.” 22 June 2010: n. pag. Print.

Strand, Renee. “The Evolution of Structural Composite Lumber.” *STRUCTURE Magazine* Aug. 2007.

“Structural Composite Lumber (SCL).” *Mass Timber*. N.p., 2013.

SWG ASSY® VG Plus and SWG ASSY 3.0 Self-Tapping Wood Screws. Canadian Construction Materials Centre, 2013. Print.

TimberFirst. *Prefabricated CLT Panel*. N.p., 2012. Web. 4 May 2014.

Vinzenz Harrer GmbH. “Sherpa Connection Manual.”

Wallwork, Tristan. “Tristan Wallwork Structural Engineer.”

Waugh Thistleton. *Murray Grove Unfinished Interior*. N.p. Web. 4 May 2014.

White, Gavin. City Academy. 9 June 2013. E-mail.

White, Robert. Cross Laminated Timber Fire Design. 19 June 2013.

Winter, Wolfgang et al. “Strategies to Increase Use of Timber in Multi-Storey Buildings - Case Studies.” Auckland, New Zealand: N.p., 2012. Print.