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Enhancing the performance of dowel type fasteners and a case study of timber truss failure

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This document will outline the findings of three separate and independent studies:

Study 1: In or around 1972, an experimental building was constructed. One of the intents of the construction project was to demonstrate advancements in wood building construction design. It was value-engineered throughout. That is, its materials and systems were intended to function at or near design capacity. In 2019, part of the roof of the structure collapsed. This case study investigates two potential factors that led to the failure: stress concentration in excess of the 12 allowable stress for 2 × 4 web members and insufficient plywood sheathing to support live loads 13 caused by large rain events.

Study 2: As a building material, cross laminated timber (CLT) has exponentially grown in popularity recently. Although performing superior to numerous other popular building materials, a consistent issue presented in wood construction is the effect of moisture on performance. This study looks to investigate the effect of moisture content on the performance of a 2-way dowel type fastener system loaded in shear perpendicular to the major strength axis. It was found that the peak load capacity of the specimens was not affected by the moisture content of the CLT. However, yield strength increased as the moisture content decreased. Lastly it was

found that the failure mode changed from ductile to brittle as specimens became drier than 12% moisture content by mass.

Study 3: Inherently, the weak point of any structure is the connection system. This phenomenon is particularly apparent in wooden structures as dowel type fasteners place tremendous amounts of stress perpendicular to the grain of the wood, as well as shear stress under the bolt. In hopes of mitigating this behavior, fiberglass reinforcement of these samples is examined to see if both failure mode as well as overall performance of these fasteners could be improved with reinforcement. It was found that fiberglass significantly reduced the standard deviation of failure strength of fasteners, significantly increased the overall strength of the fasteners, increased the efficiency of the fasteners, and finally increased the probability of bearing failure opposed to block shear failure.

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CHAPTER I

TECHNICAL NOTE: LIGHT FRAME WOOD TRUSS COLLAPSE IN MISSISSIPPI, A CASE STUDY

1.1 Introduction

Building and or roof collapses are not entirely uncommon. Regardless of building materials and roofing systems, failures can occur. Often the postfailure analyses (postmortem) are protected and become discovery for insurance claims and civil, or sometimes criminal, litigation. In the case of public construction however, related documents from the time of building inception are both preserved and available. In this instance, in or around 1972, a 557 m^2 (6000 square foot), two-story, light-wood frame construction building was conceived and erected. The rectangular building was framed on a slab 8.5 m (28 feet) wide and 29 m (96 feet) long. Its purpose was for use as both office and laboratory space at the Mississippi Forest Products Utilization Laboratory, hereafter MFPUL. At that time, the MFPUL was both a separately funded state agency, with a research and service driven mission, and an academic entity within Mississippi State University. Since construction, this structure has been known as "Building 4" as it was the fourth building to be constructed as part of the MFPUL.

The first written notion of constructing this building is excerpted below by MFPUL Director, Dr. Warren S. Thompson. "As part of a new research project on timber construction, the Mississippi Forest Products Utilization Laboratory plans to erect a building that will incorporate several innovations in construction techniques. Data collected during and subsequent to the construction of this building will be of value in evaluating the applicability of the techniques used to residential and other types of light-frame construction" (Thompson 1972). This manuscript is intended in support of the building's documented intent toward collection and dissemination of data subsequent to construction. As a public building, its conception, construction, service life, and repair history are well recorded. Of particular interest and focus herein is the roof structure. It was framed with dimension lumber-based flat roof trusses.

1.2 Light Frame Building Construction

Roof framing, sheathing, and surface: The roof's framing system is light frame dimension lumber flat trusses. The trusses are placed 1.2-m (4-foot) on center. The live load design for the trusses is 0.96 kPa (20 pounds per square foot (psf)) (Thompson 1972). The trusses have a depth of 1.8 m (six feet) and a length of 8.53 m (28 feet). The exterior stud walls are 14-cm (5.5-inch) thick resulting in a clear span of 8.25 m (27 feet 1-inch). Atop the trusses is 1.9-cm (¾-inch) thick plywood sheathing. Atop the plywood sheathing is layered rigid cellulosic insulation, approximately 10-cm (4-inches) thick at mid span. Part of the intent of this insulation is to give the roof surface a small amount of pitch, on the order of 2.5-cm (1-inch) rise across 3 m (10 feet) of length. The roofing material is built up of heavy asphalt impregnated felt paper, with tar and gravel—approximately 1-cm (3/8-inch) washed aggregate—on the surface. Dead loads are estimated in Table 1tbl1. A rendition of the truss design is provided in Fig 1.1. The top chord is 2 × 6 southern pine dimension lumber, the bottom chord and webs are 2 × 4 southern pine dimension lumber (Clearspan Components, Inc. 1972). The web material is specified as Number 2 grade, kiln dried.

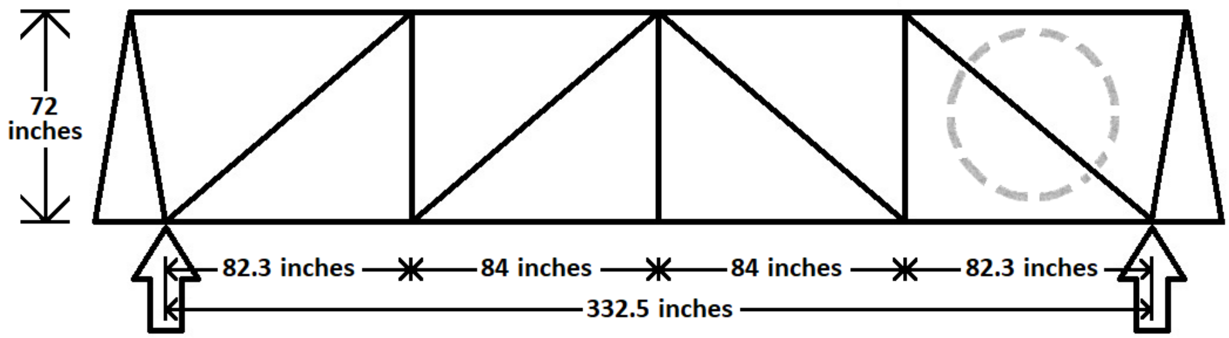

Figure 1.1 Truss diagram. The support reactions are shown at the centerline of the exterior support walls. The dotted circle indicates the web member that failed on multiple trusses and appears to have initiated the collapse.

1.3 Intermediate Repair

In or around 2005, after approximately 33 yr in service, some truss web members failed. These failures were audibly noted by building occupants who reported periodically hearing “wood snapping” and “wood cracking” above their heads, inside and above the suspended ceiling. Inspection of the roof framing revealed that the web members that failed were the ones that ran upward, diagonally, from the exterior southeast wall toward top chord, toward the interior of the building. The failure was due to columnar buckling in nonbraced compression web members. Figure 1 illustrates the web critical web member on the original truss blueprint. Neither decay nor overloading seemed to be factors in the failure.

Exterior inspection revealed water ponding, on the roof, over approximately half of the span of three of the trusses. That is, the ponding extended across several trusses. The trusses were shored up by reducing the span and locating supporting beams underneath the top chords of both the failing trusses as well as the trusses immediately adjacent thereto. This repair transferred a portion of the loading through an interior wall, the floor system, and down to the building’s reinforced concrete slab. This repair effectively lifted up the roof to the point of being level for approximately 6.1 m (20 feet) of length near the middle of the building's 29.3-m (96-foot) long dimension, eliminated the ponding, and increased the service life of the roof system. At that time, no additional analyses were conducted and no building-wide roof repairs were considered.

1.4 Final Collapse

During the winter of 2019, occupants again noted audible wood failure inside the building's ceiling, this time toward the building's southwest end. The area of the building at issue was vacant at the time. The area was cordoned off, locked, and then immediately inspected. The suspended ceiling tile system had come down as much as 12.25 cm (6 inches) under some of the trusses in the area of failure. Inspection of the roof truss system, by ladder from inside the building, indicated that several trusses were failing. Ultimately, these failures should likely be classified as deflection induced, that is, long and unbraced columns deflecting, which led to ponding, which exacerbated the deflection, and so on up until catastrophic failure. Common to these failures were the unbraced compression web members immediately atop the southeast wall (similar to the web members that broke and were repaired, in another area of the building, approximately 15 yr prior). Inspection of the roof showed ponding, that is, additional dead weight on the system. The pond's maximum depth was estimated at 7.5-10 cm (3-4 inches) and its area was estimated at approximately 6.7 × 6.7 m (22 × 22 feet) (Fig 1.2). At that stage, it appeared that catastrophic roof failure was likely imminent and there would not be time to propose and/or enact a repair. As such, the area at issue under the roof was blocked off to avoid any potential injury. That evening, during a heavy rain and wind event, approximately 8 h after the initial reports of "wood breaking" were received, the roof collapsed (Fig 1.3).

Figure 1.2 Ponding on roof, over an estimated 6.7 x 6.7 meter (22 x 22 foot) area, hours before a heavy rain and windstorm event during which the roof collapsed.

Figure 1.3 Roof section, approximately 7.3 x 8.5 meters (24 x 28 feet), (7 trusses each 28 feet long) immediately following collapse.

1.5 Analysis

After the failure, the truss plans and roof system were analyzed. Two findings are of particular interest. One finding relates to the roof's plywood sheathing and the other to the compression stress in one of the repetitive truss's web members.

It is estimated that the dead load of the roof system is at least 0.67 kPa (14 psf). Combined with a 0.96 kPa (20 psf) live load (by design), the total design load for the structure is 1.63 kPa (34 psf). It is noted that a 7.6-cm (3-inch) deep pond equates to approximately 0.77 kPa (16 psf) live load—in perpetuity. As such, it seems reasonable to assume that, at least in some portion of the roof's area, it experienced this loading more or less in perpetuity. In contemporary allowable stress design, it is anticipated that members and or structures will only experience full design load for 10 yr, cumulatively, throughout the life of the structure. Permanent or perpetual loading requires an adjustment factor of 0.9 to bending strength.

Though stretched to its potential maximum, the 122-cm (48-inch) on-center roof spacing under 1.9-cm (¾-inch) thick plywood sheathing is acceptable. However, to achieve 122-cm (48-inch) on-center spacing, it is recommended that the plywood be edge-blocked (Hoyle 1978). That is, 48/24-span rated plywood needs H-clips, tongue and groove, lumber/block supports, or some other manner of sharing the load between adjacent sheets to achieve the 122-cm (48-inch) rating. The time of the construction may predate H-clips and tongue and groove sheathing but the need for load sharing among panels, as noted by lumber blocking or edge blocking was noted. Absent this edge support, the maximum allowable on-center spacing for roof framing beneath the plywood is 91.5 cm (36 inches). In this case, no edge supports were noted. As such, based on the guidance from Hoyle (1978), the plywood supports were likely too far apart. That factor would seemingly create or facilitate ponding in a scalloped fashion, between adjacent trusses. This situation may have contributed to between-truss ponding and initial sagging of the sheathing and or trusses.

Post-collapse inspection of the remainder of the building's non-collapsed roof framing, revealed significant compression-induced deflection in the web member of interest. Fig 1.4 illustrates a series of trusses with their unbraced web members as they sit atop the exterior wall. An exemplar web member (Fig 1.5) was photographed in an effort to measure the deflection against a stringline. At the time of the photograph in Fig 1.4, there was no standing water on the roof. Thus the only loading on that truss at that time is the approximate 0.67 kPa (14 psf) dead load. At 1.63 kPa (34 psf) combined load, 9.22 m (30.25 foot) top chord length, and 1.22-m (4-foot) spacing, each truss is intended to carry 1866 kg (4114 pounds). That level equates to 202 kg per lineal meter (136 pounds per lineal foot), uniformly distributed along the symmetric truss length. As such, the estimated reaction at each exterior wall is 933 kg (2057 pounds). The load from the outermost half of the outermost 243.8-cm (96-inch) section of top chord is subtracted from this value to determine the reaction force that is pushing on the web member of interest. The top chord span above the exterior wall is approximately 243.8 cm (96 inches) (Fig 1.6). The loading associated with outer most 1.22 m (4 feet) (half of the chord) is transferred to the reaction wall through the shorter web member that extends from the exterior wall, slightly outward, to the top corner of the roof. That vertical reaction load value associated with that web member is estimated as 246.75 kg (544 pounds) (1.22 lineal meters [4 lineal feet] times 202.4 kg per lineal meter [136 pounds per lineal foot]). The balance of the reaction force (933 kg − 247 kg = 686 kg [2057 pounds − 544 pounds = 1513 pounds]) is transferred vertically through the web member of interest.

Figure 1.4 Series of repetitive flat roof trusses sitting on the exterior roof wall. Of particular interest is the unbraced 2x4 compression web member running diagonally up from the exterior wall.

Figure 1.5 Exemplar web member showing 15.5 centimeters (6-1/8 inches) of deflection. The member is 2x4, #2 as called for in the truss diagram. Its unbraced length is 256.6 centimeters (101 inches). The truss had a little or no live loading at the time of the photograph.

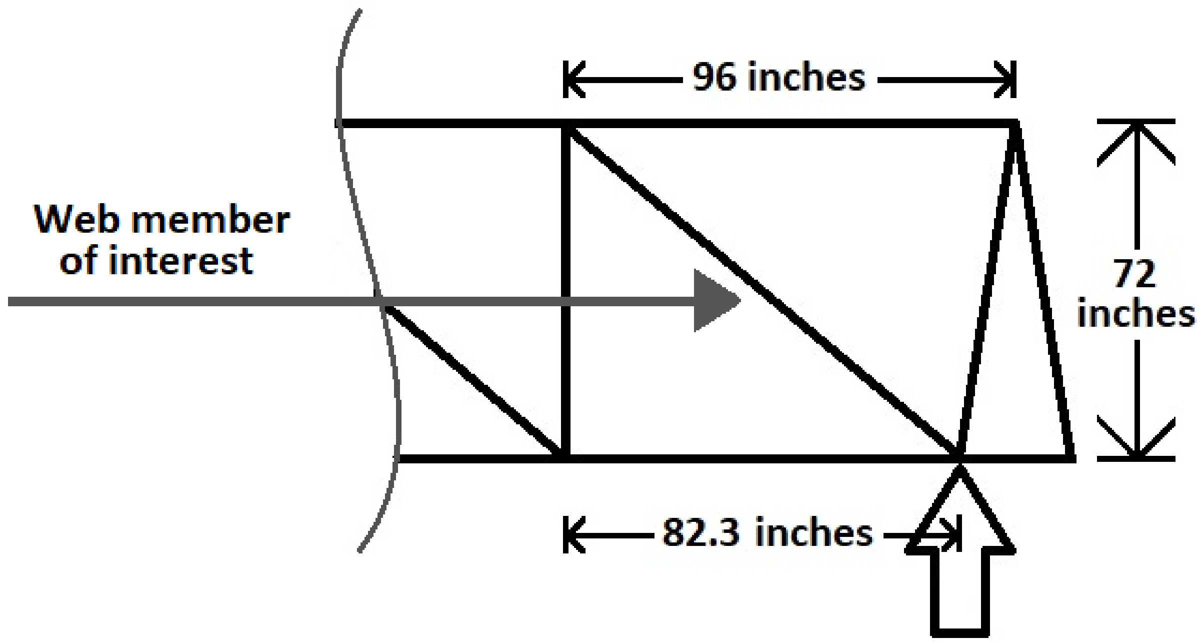

Figure 1.6 Determination of reaction force transferred to the web member of interest. The total vertical reaction is estimated as 933 kilograms (2057 pounds). The load from the outer 1.22 lineal meters (4 lineal feet) (246.75 kilograms or 544 pounds) of the top chord is transferred to the support wall by the outer compression web member. The balance, 686.3 kilograms (1513 pounds is the vertical reaction force that the web member of interest resists.

As measured, the unsupported length of the web member of interest is 256.5 cm (101 inches). Its vertical rise is 160 cm (63 inches), that is, 182.9-cm (72-inch) total truss height less the 8.9-cm (3.5-inch) deep bottom chord and 14-cm (5.5-inch) deep top chord. As such, with respect to the horizontal, the web member's angle theta is 38.6 degrees (sin 38.6 degrees = 63/101). At that angle, the 686 kg (1513 pound) vertical reaction force equates to 1100 kg (2426 pounds) of compressive force to the web member of interest. With a cross-sectional area of 33.9 cm^2 (3.8 × 8.9 cm) (5.25 $inches^2$ [1.5 × 3.5 inches]), the web member of interest is estimated to be routinely experiencing 3185 kPa (1100 kg/33.87 cm^2) (462 psi [2426 pounds/5.25 in^2]) of columnar compression due to total loading.

With a 256.5-cm (101-inch) unsupported length and a least cross-sectional dimension of 3.8 cm (1.5 inches), the slenderness ratio of this web member of interest is 67.3 (ie 101/1.5). With that relatively large slenderness ratio, this web member can be approximated as a long column; that is, its allowable strength is a function of its stiffness, not tabulated parallel to grain compressive strength. Per the design values of that time period, the allowable bending stress (single member) and modulus of elasticity for this web material are 9308 and 1.03×10^7 kPa (1350 and 1.5×10^6 psi), respectively (Southern Pine Manual of Standard Wood Construction 1975). The Southern Pine Manual of Standard Wood Construction (1975) also provided guidance regarding column design. In the solid column section, it provides a formula for calculating allowable column design stress. The column design formula is:

$$\mathrm{Fc}' = \frac{0.3E}{(\frac{l}{d})^2} Fc' = \frac{(0.30E)}{\left(\frac{l}{d}\right)^2} \tag{1.1}$$

$$Fc^{'} = \frac{(0.30 \times 1{,}500{,}000psi)}{\left(\frac{256.5}{3.81}\right)} = 99.3psi \ = \ 685kPa$$

A further reduction of this allowable compressive stress should be taken as the loading is more or less permanent or perpetual. The correction factor for permanent loading in the mid- to early 1970s was 0.9 (Southern Pine Manual of Standard Wood Construction 1975). The same factor remains in effect today. If applied, this reduces the Fc’ value to 616 kPa (89.4 psi). The Southern Pine Manual’s section on columns also states “for a simple solid column, the ‘l/d’ ratio should not exceed 50.” As such it appears that as-constructed the slenderness ratio of this web member significantly exceeds the maximum allowable slenderness ratio and that its loading is excessive.

Another way of visualizing or estimating the bending stress in the compression web members is by calculating the stress required to achieve the observed level of deflection. The deflection in the web member of interest stems from both loading and creep, however the potential creep deflection is unknown. Creep aside, the analogous uniform load (pounds per inch) on the member can be calculated by the following equation (Douglas Fir Use Book 1958):

$$w = \frac{384\delta_m EI}{5l^4} = \left(\frac{384\delta_m E}{5l^4}\right)\left(\frac{bh^3}{12}\right) w = \frac{384\delta_m EI}{5l^4} = \left(\frac{384\delta_m E}{5l^4}\right)\left(\frac{bh^3}{12}\right) \tag{1.2}$$

where

w = the uniform load on a member

δ_m = the maximum deflection of 15.6 cm (6.125 inches)

E = modulus of elasticity at 1.03 × 10^7 kPa (1500,000 psi)

b = the width of the member at 8.9 cm (3.5 inches)

h = the height of the member at 3.8 cm (1.5 inches)

I = the moment of inertia at 40.96 cm^4 (0.984 in^4)

384 and 5 are constants based on load configurations and assume a simply supported uniformly loaded beam.

l^4 = the length to the fourth power, which is 4.33 × 10^9 cm (1.04 × 10^8 inches)

Thus, the analogous uniform load on the member required to create that level of midspan deflection is 1.19 kg per cm (6.67 pounds per inch). The maximum moment associated with this load configuration is calculated as:

$$M_{max} = \frac{wl^2}{8} M_{max} = \frac{wl^2}{8} \tag{1.3}$$

Where:

M_{max} = maximum moment at mid span

w = the unit load per length = 1.19 kg per cm (6.67 pounds per inch)

l^2 = unsupported span length squared = 101^2 = 6,5812 cm^2 (1,0201 in^2)

Thus, M_{max} = 961 N meters (8505-pound inches). By the flexure formula, one can then calculate the bending stress associated with this bending moment.

$$s = \frac{Mc}{I} \tag{1.4}$$

where

s = observed bending stress (kPa (psi)) = 4,4692 kPa (6482 psi)

M = Maximum moment = 961 N meters (8505-pound inches)

c = ½ the beam's depth = 0.5 * 1.5 inches = 1.9 cm (0.75 inches).

I = Moment of inertia = 40.96 cm^4 (0.984 in^4)

Thus, the estimated bending stress (es), absent any creep, to create the level deflection as observed is 4,4692 kPa (6482 psi) in a series of compression web members whose Fb value is 9308 kPa (1350 psi) (Southern Pine Manual of Standard Wood Construction 1975).

1.6 Conclusions

As constructed, the plywood sheathing appears to have inadequate framing underneath it. Without edge supports (H clips, tongue and groove, lumber blocking, etc.) as a means of sharing the load, the maximum recommended able on-center spacing for roof framing is 91 cm (36 inches). As built, the on-center spacing of the framing is 122 cm (48 inches), without edge blocking, and is therefore beyond that recommended. Given that deflection is a cubic function of length, one can expect that the deflection associated with 122-cm (48-inch) spacing (118-cm

[46.5-inch] actual span) is 2.4 times more than that associated with 91-cm (36-inch) on-center spacing (34.5-inch actual span). This situation likely led to initial water ponding on the roof between adjacent trusses.

As designed, and as constructed, the web member of interest behaves as a long column. In this type of situation, its allowable columnar stress is limited by its modulus of elasticity. The truss design calls for a (nonstructural) “1 × 4 continuous lateral bracing by others (typical)” such that each column (compression web member) would be braced to the analogous members in the trusses immediately adjacent thereto. If that bracing had been installed, and if it had acted in some type of structural capacity, in the best-case scenario, the web member of interest’s slenderness ratio could have been reduced to approximately 33.7. In that case, the bracing may have raised the allowable design capacity, corrected for duration of load, to approximately 2468 kPa (358 psi). In all likelihood, this action would have prevented and mitigated the long-term deflection and avoided the premature catastrophic failure.

As built, however, the columns (compression web members of these trusses) are entirely unbraced. Thus, their slenderness ratio (67.3) exceeds that allowable per guidance of that period (50), per the Southern Pine Manual of Standard Wood Construction (1975). By calculation, the Fc’ for these web members in service is approximately 620 kPa (90 psi). As the actual columnar stress on each of these web members appears to be on the order of 3185 kPa (462 psi). As such, in service, these web members appear to be overloaded by a factor of approximately 5.

The excessive deflection in this web member of interest facilitates significant sagging in the top chord and thereby permits excessive ponding. This ponding, and its associated perpetual loading likely contributed to the collapse. At the time of the inspection, many of the roof trusses that remain in service in the building appear significantly overloaded. Based on this finding, it will be recommended that the roof system be completely redesigned and rebuilt prior to reoccupancy or that the structure be razed and rebuilt depending on the associated costs of each alternative.

CHAPTER II

THE EFFECT OF MOISTURE CONTENT ON 2-WAY WALL-TO-FOUNDATION DOWEL TYPE CONNECTION SYSTEM

2.1 Introduction

The interest in cross laminated timber (CLT) as a building material has increased dramatically over the last decade. With superior performance in strength, construction speed, and utilization of lower grade wood, CLT is favorable as compared to other wood construction options for mid- to high-rise buildings and other large construction projects. Simultaneously, CLT provides better thermal, acoustic, and seismic performance while providing a more carbon neutral alternative than steel and concrete. These benefits make CLT an intriguing option for future buildings.

One of the more common utilizations of CLT is for shear walls. The wall to foundation or wall to floor connections of CLT shear walls experience loading that fall into two basic categories: uplifting forces and lateral forces. Traditionally hold-down brackets withstand uplifting forces while angle brackets resist lateral forces. In other words, 2 separate wall to foundation connection systems are used to effectively resist each force (Gavric et al, 2015) (Ceccotti et al, 2013).

This study looks into using a single fastener that would extend the full length of the CLT shear wall. Utilizing a single connection system that resists both uplifting and lateral forces will increase design simplicity and speed up installation. For the purposes of this preliminary study, lateral movement resistance strength of this connection system was the primary focus. Lateral movement resistance was focused on due to the fact that the majority of large CLT wood construction buildings constructed in the southeastern United States are constructed with a larger on ground square footage rather than tall buildings. Buildings constructed in the southeastern United States are also subjected to high levels of humidity. With this in mind, moisture content was also considered in this study.

One issue that remains constant with any wood product is the deleterious effect of moisture on the mechanical performance and longevity of wooden members in service. As the moisture content of wood increases, it is well documented that the performance of dowel type fasteners decreases. Rammer and Winistorfer (1999) found an inverse linear relationship between dowel bearing strength for clear specimens of southern pine and Douglas fir / larch and moisture content. Their study found that moisture content was an independent determining factor in dowel bearing strength separate from species, specific gravity, and fastener type.

Studies undertaken by Lepage et al. (2017) demonstrated that CLT exposed to exterior conditions is extremely susceptible to reaching moisture contents that are unacceptably high. In their study, water resistant coatings were tested to prevent moisture from penetrating panels during the construction phase. That study found that untreated panels can reach moisture contents over 20 percent in the center laminations in a matter of months of outdoor exposure. Furthermore, that study found that treating panels with water protective coatings can be beneficial to the longevity and performance of the panels in exterior conditions.

Keeping in mind that the focus of this study is for wooden structures constructed in the southeastern United States, using fresh CLT panels would not accurately represent the in situ performance of a CLT shear wall. The climate of the southeast United States presents high temperature and humidity for extended periods of time and flooding conditions which result in large amounts of moisture penetration into the wood. For this reason, panels were recycled from a previous study (Weaver et al, 2018) that exposed the untreated panels to the harsh climate of Florida for 1 full year. Utilizing these panels will better represent the long-term performance of this connection and show the response to changing moisture conditions.

This study examines the effect of moisture content on the performance of a new CLT shear wall to foundation connection system. The shear wall connection is the region of a building system that is often closest to the ground, resulting in the harshest conditions for wood with respect to mechanical performance. In this experiment, 3/8 in diameter lag screws connected ¼-in steel angle to 5-Ply V1 Douglas fir CLT specimens (ANSI/APA, 2018). Steel connections were overdesigned to focus on wood performance under these conditions. These specimens were tested in short span 3-point loading in order to determine the effect of moisture content on the wood performance in CLT shear walls connected using this dowel type fastener.

2.2 Materials and Methods

Specimens for this study were made from a single 15-feet by 7-feet 5-ply V1 CLT panel (ANSI/APA, 2018). The panel was cut down into specimens measuring 36-in long and 10-in deep. The panel was an unused remnant from a previous study conducted by the U.S. Department of Agriculture Forest Products Laboratory. The untreated panel had been exposed to exterior conditions for 1 year. For full details of the previous utilization of the CLT panel used in this study see Weaver et al. (2018).

Panels were cut into 10-inches deep by 36-inches long specimens. The panel was oriented so that the major strength axis was the 36-inches dimension. A total of 25 specimens were made for this study. A typical specimen is shown in Fig. 2.1. Once specimens were cut, they were drilled according to the American Wood Council National Design Specification for Wood Construction section 12.1.4 in order to accommodate 6-inch lag screws with ¼-in thick steel plates. Dimensions for the A36 galvanized steel bracket can be seen in Fig. 2.2**.** For installation, 3/8-inch diameter holes were drilled to a depth of 1 ½ -inches from the wood surface, and ¼-inch diameter holes were drilled to a depth of 5 ¾-inches A drilling pattern was made in order to ensure that the concentric holes were centered on top of each other. This same drilling pattern was used when fabricating the steel angle brackets in order to ensure a perfect fit that would not induce additional stresses from the fasteners (Fig 2.3).

Figure 2.1 Typical example of 5-Ply CLT sample

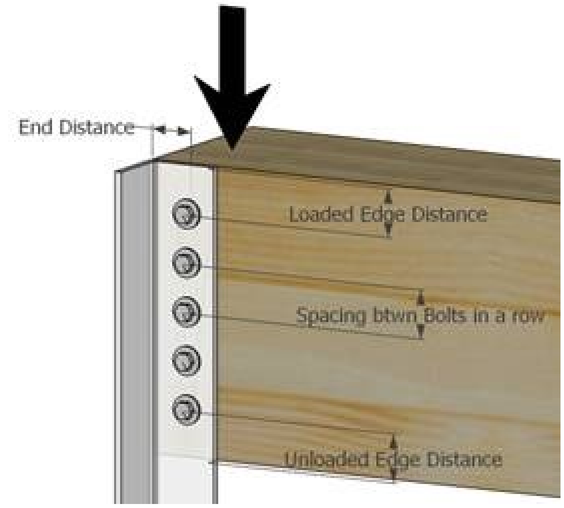

Figure 2.2 Reference for bolt spacing. Loaded and unloaded edge distance is 2 inches. Spacing between bolts is 1.5 inches.

Figure 2.3 Jig utilized for uniform predrilling of CLT specimens.

Once the specimens were cut and drilled, they were divided into 7 separate groups: Control (Dry), 30%, 27%, 22%, 19%, 17%, and 15% (percent meaning the dry-basis moisture content of the outermost layer in the failure plane of the specimen). The Dry group was comprised of 3 specimens. These specimens were placed in a temperature and humidity-controlled room for approximately 2 months. The room was set to 89.9˚F and 25% relative humidity. The goal of placing specimens in these conditions was to make them as dry as possible during the time of the study without exposing them to potentially damaging elevated temperatures. The remaining 18 specimens were placed in a soaking basin for a duration of 11 days. Specimens were placed in the basins in 2 layers separated with dunnage. Weights were placed on top of the specimens to keep them submerged. Upon completion of the 11-day wetting cycle, specimens were removed from the water. Immediately upon removal, plastic was wrapped around the end grain in order prevent rapid moisture loss. Additionally, wrapping the specimens in plastic forced moisture movements to simulate that of an entire wall section. Water was forced to move through the thickness of the specimens rather than laterally from the specimen. In a wall section, moisture moves through the thickness of a specimen due to the assembly of the building. This wrapping better simulated in situ conditions of a CLT structure. Specimens were then stored in a temperature and humidity-controlled room set to 68.3˚F and 60.0% relative humidity. The wetted and plastic wrapped specimens can be seen in **Figure 2.4**. Surface moisture contents exceeded 30% after soaking, however moisture did not penetrate deeply into inner layers. Internal moisture contents varied after soaking from 16%-22%. It was evident that soaking did not alter these values appreciably, if at all. The assumption was made for this study that the greatest amount of stress concentration would be on the surface layers of the specimens creating the largest effect on mechanical performance.

Figure 2.4 Samples end grain wrapped in plastic to limit moisture movement and imitate conditions of whole wall more closely.

The moisture content at the center of each layer was measured and monitored throughout the conditioning process. Insulated screws with conductive tips were driven to the center of each layer as seen in **Figure 2.5**. Moisture content was measured using a Delmhorst J-2000 moisture meter. When the outermost laminations reached the appropriate moisture (where moisture content was approximately equal to one of the target percentages mentioned above) content they were brought to the test floor where they were prepped for testing. A total of three specimens was used in each group. Specimens were weighed, moisture content rechecked, and measured for length, width, and depth. After measurements were recorded, two 13-inches long, ¼-inch thick galvanized steel L-Brackets were attached to the specimens using 10 total 6-inch galvanized steel lag screws (5 per L-Bracket). The holes were drilled to a tight fight dimension. Upon completion of these steps, specimens were brought onto the test floor in order to begin testing. The test set up was confirmed using 4 specimens. One specimen was used for test set up and demonstration, the remaining 3 were tested in order to determine loading rate and to ensure the testing procedure was replicable, efficient, and effective. A rendering of a completed specimen can be seen in **Figure 2.6**.

Figure 2.5 Insulated screws used to check moisture content of specific layer in CLT.

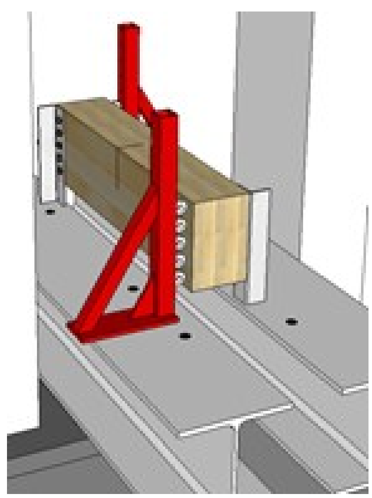

Figure 2.6 Rendering of test set up

Specimens were tested using a floor model Instron universal testing machine with a 60,000-lb load cell. The test configuration can be seen in **Figure 2.7**, **Figure 2.8**, and **Figure 2.9**. Specimens were loaded at a rate of 0.075 inches/Minute per ASTM D1761. This standard stated a load rate of 0.035 inches/Minute or maximum load in 5-20 minutes. This loading rate achieved maximum load in approximately 15 minutes. A linear variable differential transformer (LVDT) was placed at the center of each connection grouping. The LVDT measured the deflection of the joint over time. A specimen was considered failed when it deflected 0.6-inches at one of the joints or when the capacity over time began to decrease per ASTM D1761. Once the specimen was placed in the machine and the test had started, the plastic wrap was removed and discarded.

The testing setup used in this study tested specimens in double shear. This means that both joints were tested simultaneously, yielding results that are the average of the two joints. Testing in double shear allowed for testing to be performed more efficiently while having negligible impact to the overall results of the study.

Figure 2.7 Full test set up

Figure 2.8 LVDT utilized to track movement of bolts under loading.

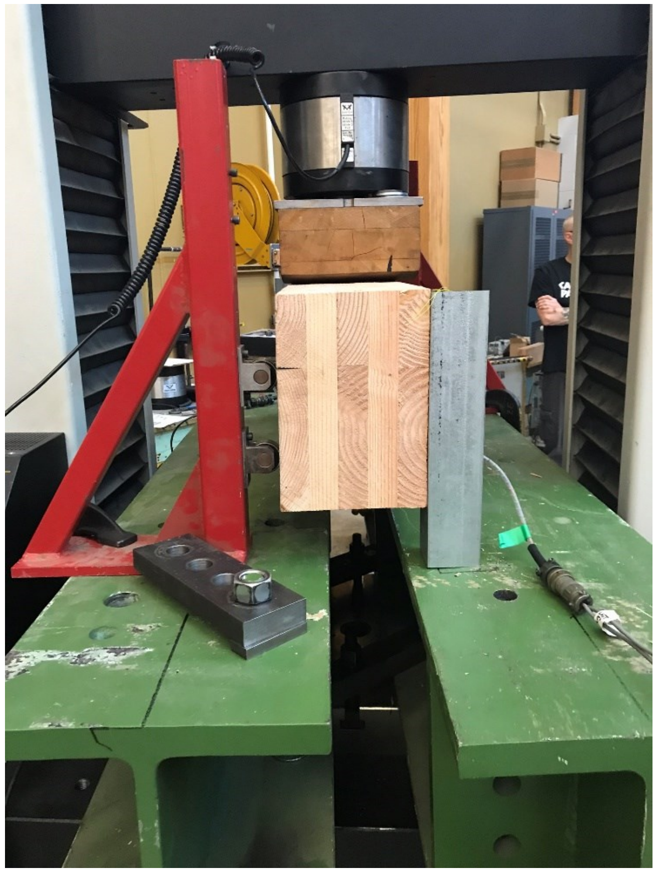

Figure 2.9 End view of test set up showing that samples were kept in plane during loading.

2.3 Results and Discussion

Upon conclusion of bending tests, data was analyzed for three separate values: max load, yield load, and failure mode. Max load was defined as the greater of the load at the deflection limit or the peak load of the deflection curve. These values were compared to the moisture content of the outermost layer. As stated previously, it was assumed that the outermost layer would affect performance the most, as in situ conditions would place the highest stress concentrations on this layer.

It should be noted that assumptions were made in this study. The first assumption is that the specimen would remain rigid and perpendicular to the connections as it was loaded (or parallel to the ground). Furthermore, it was assumed that the load was shared equally by each bracket. This assumption reinforces that in a double shear test set up, meaning the registered peak load on the Instron testing machine would be equally distributed to each bracket. Additionally, it was assumed that the connections moved uniformly about the center point of the connection. Another assumption that was made was that the measured moisture content remained constant throughout the lamina. The final assumption that was made was that wood was the failure point of each of the specimens as opposed to the steel connections, hence the use of the oversized steel components.

The yield load was determined utilizing the Yasumura and Kawai (Y & K) Method. This method calculates initial stiffness based on the slope of the line drawn between 10% and 40% of the peak load held by each bracket. A straight line is also drawn between the 40% and 90% of peak load. From this second line, a third line parallel to the 40% to 90% line is drawn tangent to the load versus displacement curve. This third line represents the immediate post elastic zone prior to reaching peak loading. The intersection of the post elastic zone line and initial stiffness line represent the yield load and displacement. An example of how this method is carried out on a typical load versus displacement graph can be seen in **Figure 2.10**.

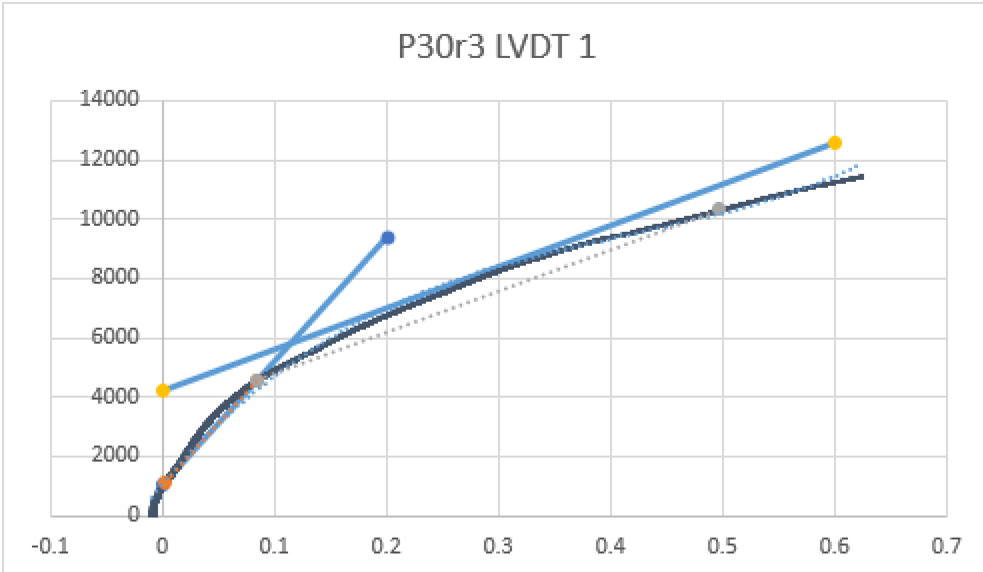

Figure 2.10 Exemplar load versus displacement curve with the Y & K method applied to determine yield load.

Analysis of variance as well as regression analysis was utilized to determine if the strength properties of CLT dowel type connections were significantly affected by moisture content. It was determined, based on this analysis, that moisture content in the outermost lamina did not significantly affect the connection strength of CLT with respect to the maximum capacity of the specimens. A significance level of 0.05 was used in order to draw this conclusion. **Table 2.1** lists the summary statistics regarding the peak load with respect to moisture content. A best fit plot of maximum load versus moisture content can be seen in **Figure** 2.11.

Table 2.1 Summary statistics for maximum load versus moisture content

Analysis Variable : Max_Load					
MC	N Obs	Sum	Mean	Corrected SS	Variance
5	6	69598.000	11599.667	3635417.33	727083.467
15	8	90808.000	11351.000	5687860.00	812551.429
17	4	46206.000	11551.500	3538161.00	1179387.00
19	6	64256.000	10709.333	10686465.3	2137293.07
22	5	55628.000	11125.600	563665.200	140916.300
27	6	66398.000	11066.333	3221369.33	644273.867
30	5	57706.000	11541.200	558154.800	139538.700

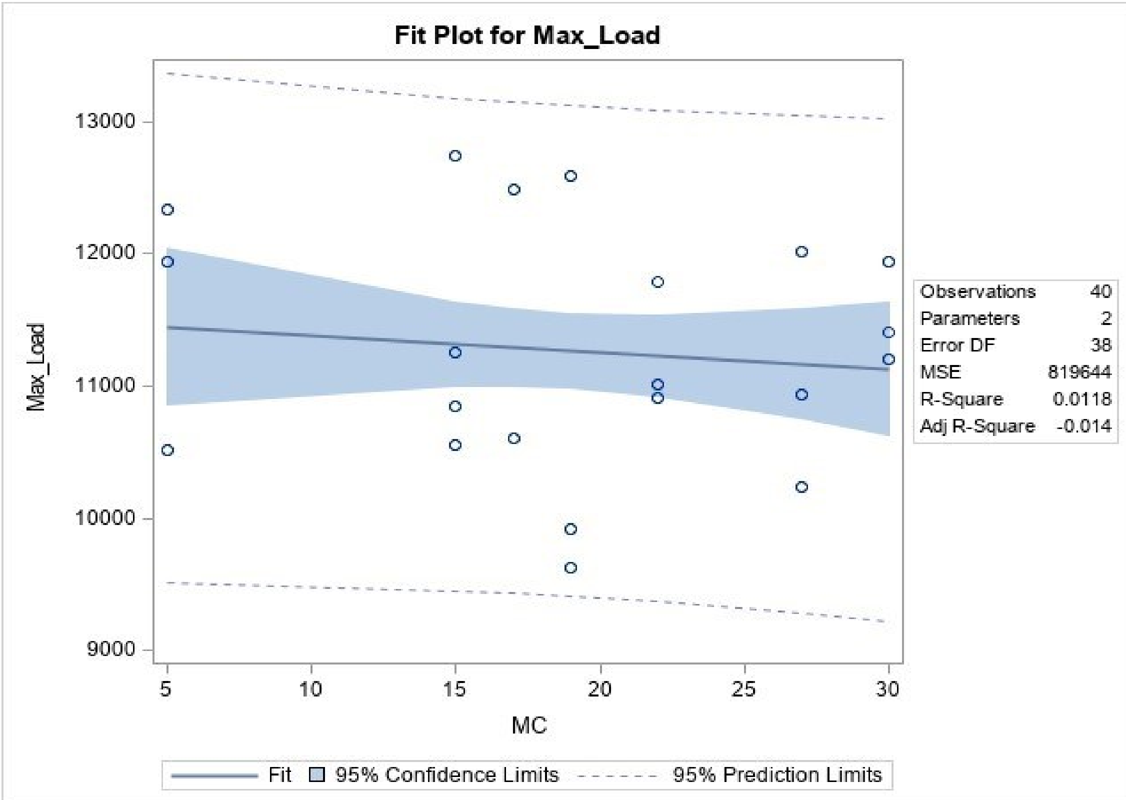

Figure 2.11 Fit plot for maximum load versus moisture content with 95% confidence limits.

Analysis of variance as well as regression analysis of yield point found significant results for load at yielding, however did not yield significant results for the deflection at yielding. Utilizing a significance level of 0.05, this means that the yield load does significantly decrease as moisture content increases. However, the deflection at which yielding occurs does not significantly change as moisture content increases. **Table 2.2** shows the results of the analysis of variance for yield load. Table 3 shows the result of the analysis of variance for deflection at the yield point. The best fit plot with 95% confidence intervals can be seen in **Figures 2.12** and **2.13** for yield deflection and yield load respectively. **Figure 2.14** and **Figure 2.15** show the distribution plots for yield deflection and yield load respectively.

Table 2.2 Analysis of variance results for yield deflection versus moisture content

Analysis of Variance					
Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	1	0.00208	0.00208	2.65	0.1120
Error	38	0.02985	0.00078549		
Corrected Total	39	0.03193			

Table 2.3 Analysis of variance results for yield load versus moisture content

Analysis of Variance					
Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	1	15877985	15877985	12.63	0.0010
Error	38	47761009	1256869		
Corrected Total	39	63638994			

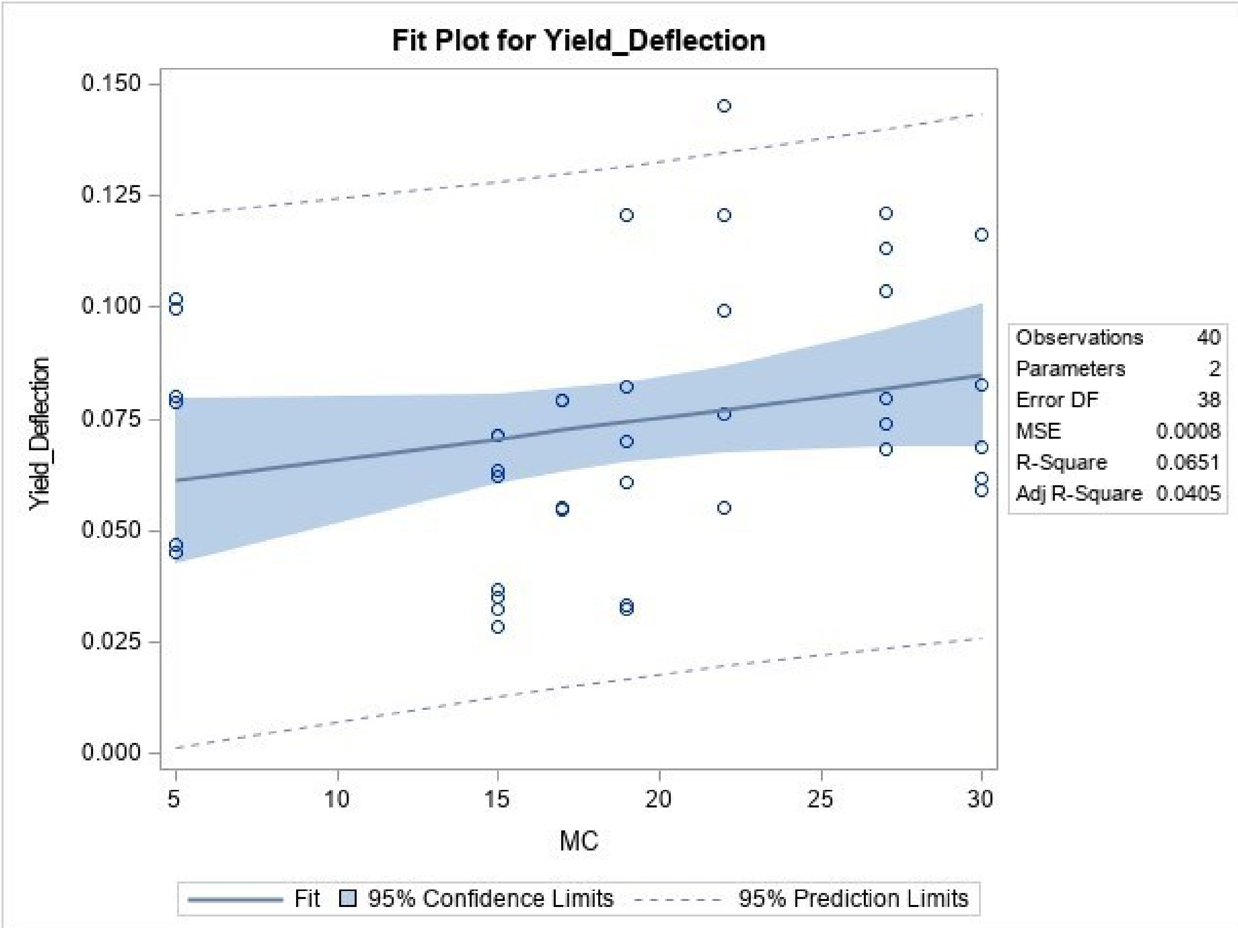

Figure 2.12 Fit plot for yield deflection versus moisture content with 95% confidence limits

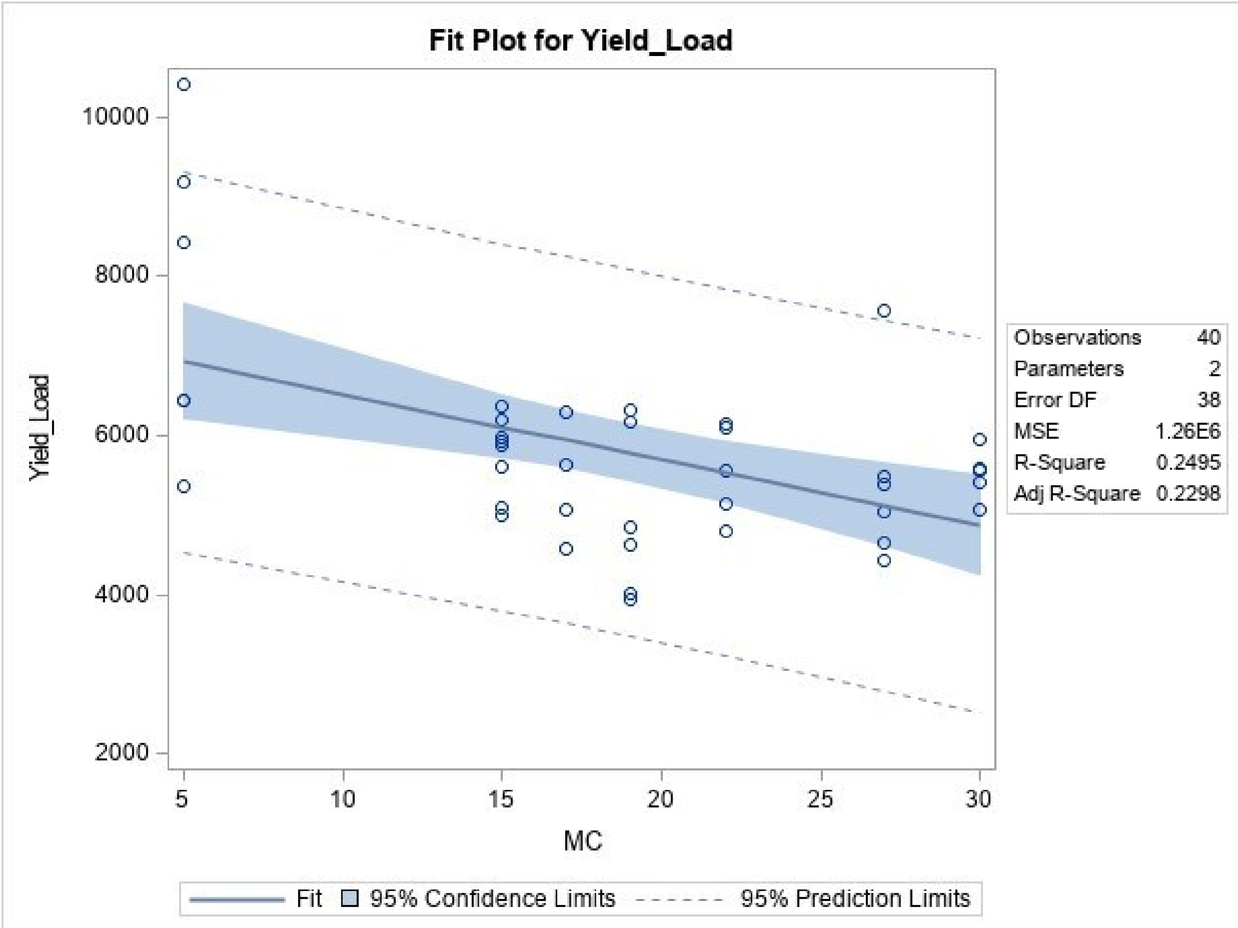

Figure 2.13 Fit plot for yield load versus moisture content with 95% confidence limits.

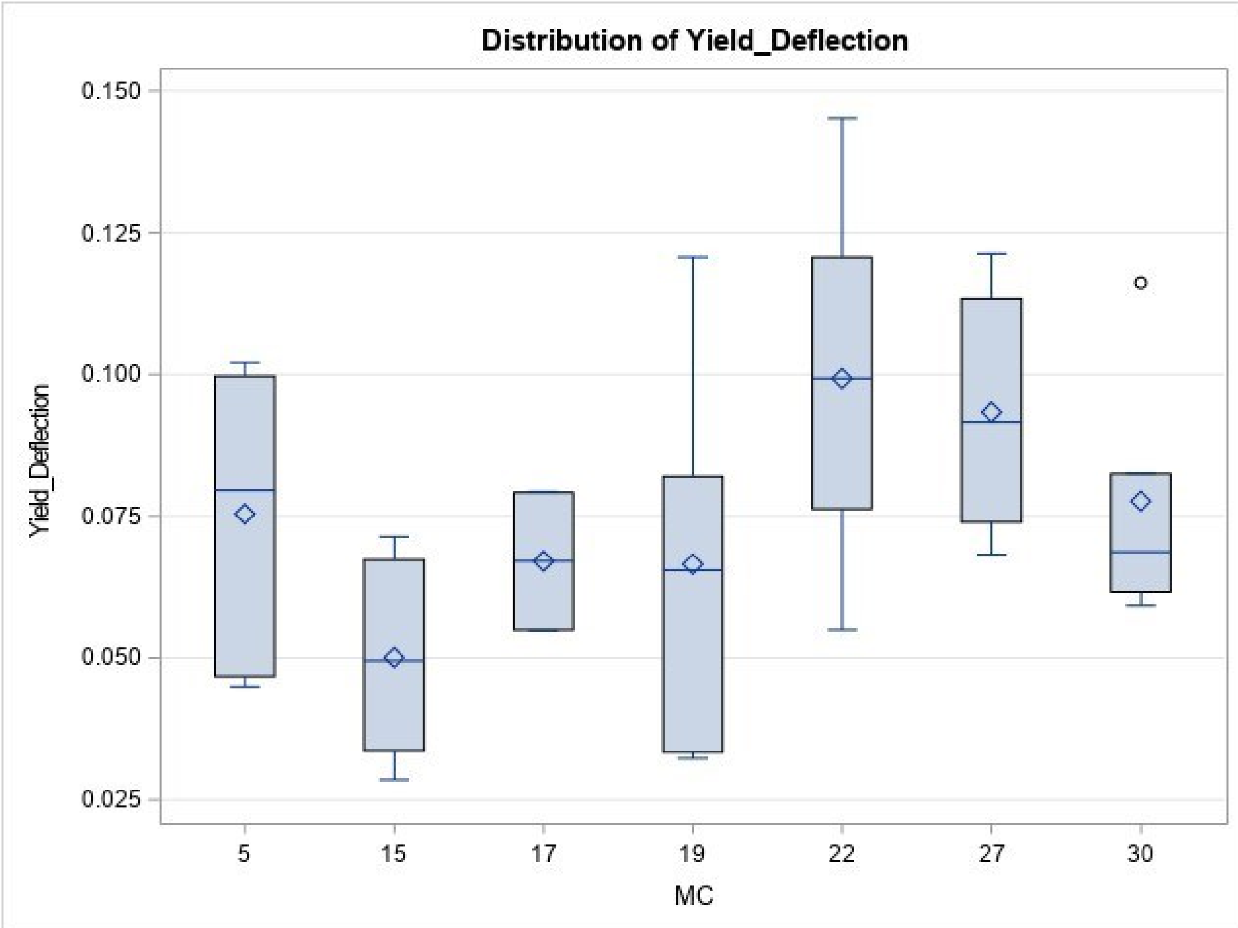

Figure 2.14 Distribution plot for yield deflection versus moisture content.

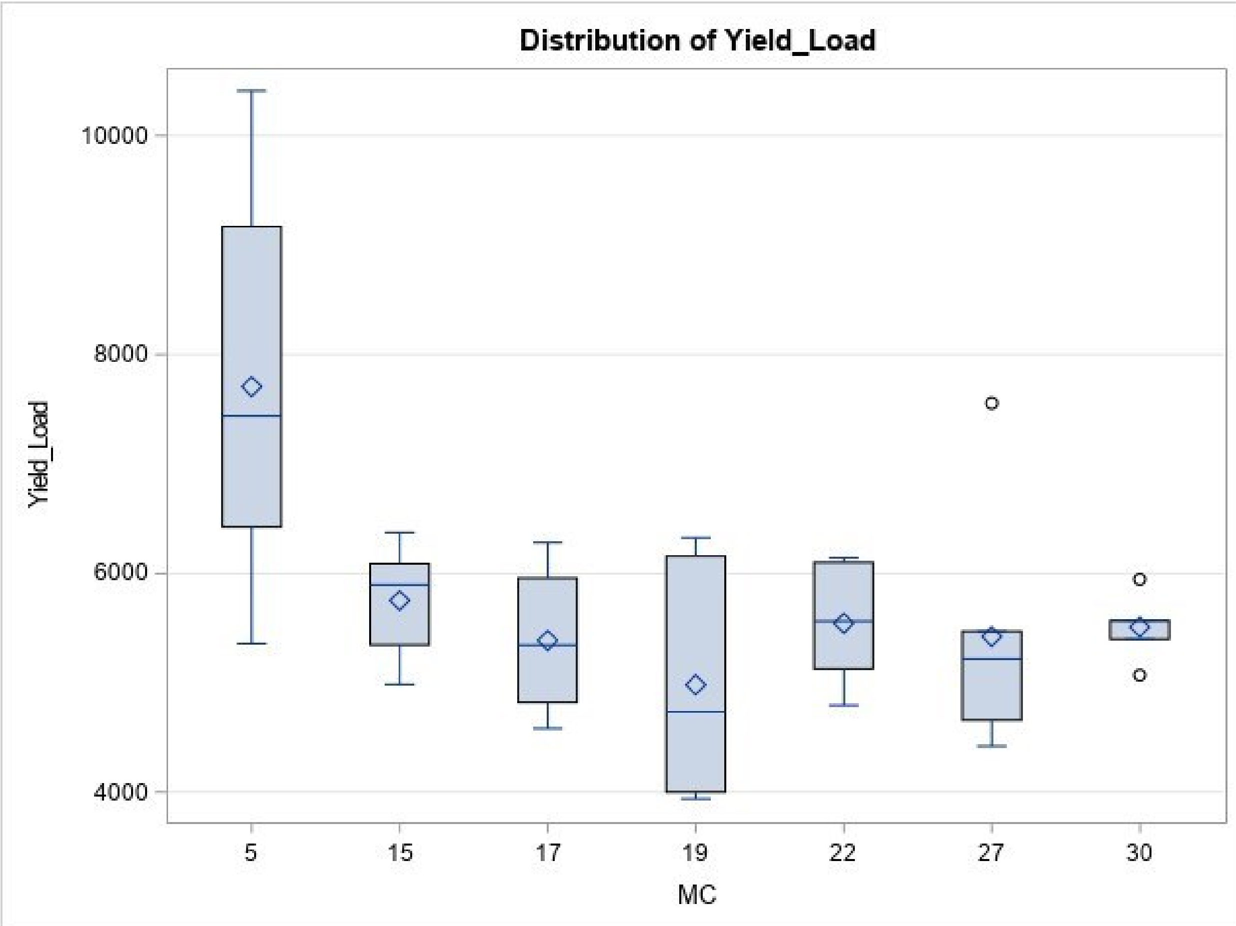

Figure 2.15 Distribution plot for yield load versus moisture content

Lastly, failure mode was monitored as a function of moisture content. Per ASTM D1761, the specimens were considered failed if a displacement of 0.6-inches was measured at the connections. This displacement was reached at all of the specimens with the exception of the dry (5% MC by mass) specimens. With the exception of the dry specimen group, the specimens continued to deflect and carry more or at least maintain the load as the 0.6-inches of deflection mark was reached. Dry specimens however all failed in a brittle manor at approximately 0.25-inches of deflection on average, well before the 0.6-inches target.

2.4 Conclusions

The use of recycled specimens that were exposed to a Floridian exterior environment may have increased the variation in this study. Despite this variability, two out of three aspects of this study found significant results. The lack of statistically significant results can be largely explained by the previous history of the panel. Overall this study yielded promising results that should be further examined in future studies.

This connection system showed consistent performance when loaded in shear. Peak loads and yielding points were consistent with current design standards. The design strength of the connection system exceeded expected failure loads based on existing design standards. Overall, this system has been shown to be a viable option for shear loading in CLT buildings.

The moisture content of CLT does significantly decrease the yield strength of dowel type connectors. That being said the deflection at yielding is not significantly changed by moisture content. Implications of this finding mean that displacement parameters can be implemented no matter the conditions of the structure to ensure loading does not exceed safe loading conditions. Additionally, monitoring displacement of joints can alert one to potential failure.

The effect of moisture content on wood strength is a well-researched and well documented science. All sources of information indicate that wood is weaker and more ductile while it is wet, and stronger and more brittle when it is dry. This study agreed with past research that the stiffness of the elastic range of the wood was higher as the CLT specimens dried. Additionally, the shift in failure mode from ductile to brittle as progressively drier specimens were tested was consistent with expectations. The unexpected result, however, was the lack of effect on overall load capacity. Unaccounted variables that were the product of using a small sample size coupled with the organic nature of wood as a building material were likely the cause of this phenomenon. Further studies should be carried out limiting these variables in order to determine just how much effect moisture has on the overall capacity of CLT dowel type connections.

Overall this test showed promising results. Significant results were found for both yield load and failure mode with respect to the effect of moisture content on these variables. Larger sample sizes in future studies will likely show that maximum load will also be affected negatively by increased moisture content. Water is one of the largest areas of concern for wood and it is imperative that its effects are fully understood before valuable money and resources are poured into buildings that are not suited to withstand the environment to which they may be subjected.

CHAPTER III

FIBERGLASS REINFORCEMENT OF DOWEL TYPE FASTENERS

3.1 Introduction

Connections in engineered structures are often the weak point of the entire system. The systems used to connect members together reduce the cross-sectional area and introduce points of stress concentration, thus making connections the location of failure for a large number of structures. Furthermore, connections in wood structures frequently fail in double shear and tension perpendicular to the grain. These modes of failure are catastrophic when they occur with rapid strength loss and minimal deflection to indicate the imminent failure. Reinforcement of these joints could ensure that joints perform more consistently and have a more favorable failure mode such as bolt bearing. This type of failure would allow for issues to be seen prior to failure and ensure wood remains competitive with other engineering materials that perform more predictably.

In addition to being inherent weak points, connections are inefficient. In wood structural design, a load reducing coefficient is applied to all connections reducing the design capacity of each fastener. The more fasteners present in the connection, the further the capacity of each individual bolt is reduced. This phenomenon is due to the fact that in an idyllic and simplified scenario, approximately half of the total load is carried by the outermost bolts. Reinforcement could help distribute loads more efficiently, which would decrease the size and number of fasteners required to maintain adequate load capacity.

In a study by Windorski et al (1997), a total of 80 single bolt connections were tested in double shear with 0, 1, 2 and 3 layers of biaxial fiberglass reinforcement. The results showed that the largest amount of strength increase was seen when going from 0 to 1 layer of reinforcement. Significant increases in strength were also seen when going from 1 to 2 layers of reinforcement and when going from 2 to 3 layers of reinforcement. However, when more layers of reinforcement were added to the specimens, the strength gained decreased. Thus, it was concluded that the most economical and efficient means of reinforcement was to only utilize one layer of reinforcing fiberglass. Additionally, it was found that approximately half of the single layer reinforcement specimens and all of the specimens with multiple layers of reinforcement failed in bearing while all of the unreinforced as well as the remaining single layer reinforced samples failed catastrophically in splitting below the bolts.

In a review study by Bulleit (1984) it was found that unidirectional fiberglass was the preferred reinforcing material up until that point in time, however the review found that there was no consensus on whether to utilize a woven or nonwoven fabric, strand or mat system. Additionally, it was found that there was no preferred resin. Epoxy was most popular, however appropriate results were also found with phenolic, polyester, and phenol-resorcinol formaldehyde resins. Finally, the study concluded that reinforcement of wood was technically feasible for improving strength and stiffness properties, however it also determined that doing so was economically infeasible.

3.2 Objectives

The objectives of this study are to determine if fiberglass presents a feasible reinforcement method to increase the efficiency of connection systems in laminated wood composites. The study will explore if the load reducing coefficient can be eliminated or significantly decreased by adding fiberglass reinforcement into the connection system. Finally, this study will explore if failure mode is affected by the addition of fiberglass.

3.3 Materials and Methods

In order to determine if fiberglass reinforcement truly impacts group action efficiency in dowel type fastener performance, a sample had to be manufactured where fiberglass was fully interfaced with the bolt. Laminated veneer lumber with fiberglass in between layers would create this interface, however laying fiberglass between layers of veneer with epoxy would be costly, would require intricate pressing techniques and would be exceptionally time consuming. Glulam beams would be far more cost effective and timely, however the glue line does not interact with each bolt in a glulam beam. With this in mind, a scaled-up hybrid of laminated veneer lumber (LVL)and glulam was thought to be the best case for this total interaction between bolt, wood, and fiberglass. The engineered wood specimen would be oriented with the glue line perpendicular to the fastener similar to the interaction of LVL. Dimensional lumber, not veneers, would be used in order to ease time requirements, cost requirements, and pressing complexity.

A pallet of kiln dried (KD19) 2 – inches x 8 – inches x 8 – feet. (nominal) grade 1 southern yellow pine lumber was purchased from Shuqualak Lumber in Shuqualak, Mississippi. Boards were planed 1/8 of an inch in order to ensure a clean and uniform bonding surface in accordance with ANSI 117 – 2020 (The Engineered Wood Association, 2020). Once resurfacing was completed, the boards were cut into 24 – inch lengths. Careful consideration was taken at this point. In the event that a knot or significant defect was located outside the middle 1/3 of the 24 – inch section, the board was removed from production in order to remove a source of variability. A total of 180 2 – feet. sections of the planed 2 – inches x 8 – inches were taken to the cold press for gluing.

Specimens were glued using the West System "105 epoxy resin" with the "206 Slow Hardener". This system is designed specifically to bond fiberglass to wood. The fiberglass used for this study was the woven, biaxial "Episize" glass fabric from West System. This fabric was designed for use in building composite laminates. Fiberglass was cut into sections slightly larger than the specimen size in order to ensure fresh, uniform fabric was bonded into the samples rather that the inconsistent fabric damaged from cutting. Excess fiberglass was cut from samples after curing.

A 5:1 ratio of resin to hardener was used per the recommendation from the epoxy manufacturer. Resin was applied liberally to the face of the board and a total of 30 3-ply samples were made with no reinforcement. An additional 30 samples were constructed with fiberglass reinforcement along the 2 glue lines. Samples were placed in a cold press for 15-24 hours to allow for full curing prior to removal. Cure time per manufacturer's guidelines was 10-15 hours. Assembly of samples can be seen in **Figure 1.**

Figure 3.1 Assembly of samples, laying fiberglass on wood and epoxy application.

Specimens were then weighed and arranged in ascending order of density. Moisture contents were consistent across all samples as they had been stored in identical indoor atmospheric conditions for over 2 months. Specimens were then selected in order to maintain similar distribution of density in each of the treatment groups. Specimens were the drilled to accommodate bolt holes. Each specimen was anchored by a pair of 1 – inch diameter bolts. The testing region was drilled with 1, 2 or 3 bolt holes to accommodate a 9/16 -- inch diameter bolt depending on which treatment group the specimen was assigned to.

Bolts were medium carbon alloy steel that had been quenched and tempered with a minimum yield strength of 130 x 10^3 psi (grade 8 bolts). Utilizing grade 8 bolts ensured a rigid connection that would test the strength properties of the wood rather than the fasteners themselves. Bolts were long enough that the threads were excluded from the loading region. For each test a new set of bolts was used to ensure any warping or fatigue from previous tests would not impact the results of the test. The bolt spacing adhered to the guidelines set in the National Design Specification for Wood Construction.

Specimens were pulled in tension longitudinally at a rate of 0.03 inches per minute per ASTM D5652. Failure was to occur in 5 to 15 minutes per this standard. Failure was defined as one of the following:

- Specimen split under the bolt
- Load resistance of the connection decreased steadily
- Displacement of the Connection exceeded 0.3 – inches.

At the conclusion of the test, failure regions were carefully inspected, and failure mode was noted. It was predicted that the unreinforced samples would fail in double shear while reinforced samples would fail in bearing. The shear failure would be more rapid and catastrophic while the bearing failure would be gradual displacement type failure. Test set up can be seen in **Figure 2.**

Figure 3.2 Test set up with LVDT attached to monitor movement of fastener.

3.4 Results and Discussion

Testing of each specimen yielded a load versus displacement curve. A representative example of these curves can be seen in **Figure 3**. It can be noted that each curve exhibited a shakiness. This shakiness can be attributed to the instability of the slightly larger diameter of the bolt hole in the wood compared to the diameter of the bolt. This discrepancy in diameter created an instability that resulted in the wobbly load versus displacement curve. This instability, while noteworthy, can be largely disregarded as it was consistent across all samples. Additionally, the overall shape of the load versus displacement curve was as expected, exhibiting an elastic region, yielding, and a region of deflection prior to failure.

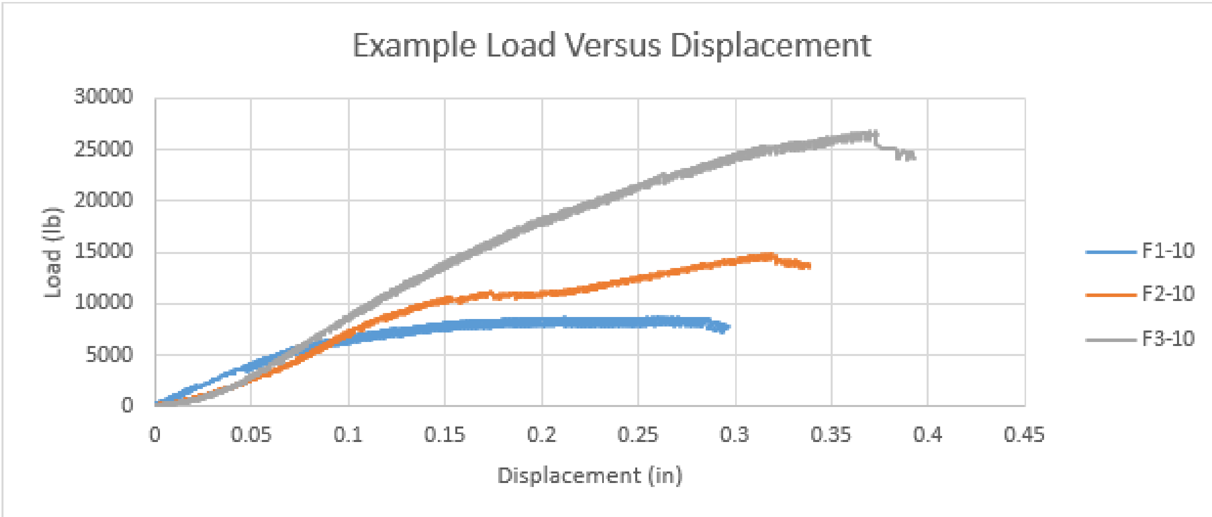

Figure 3.3 Exemplar load versus displacement curves for each bolting combination.

At the conclusion of testing, analysis of variance tests were carried out in order to determine if the fiberglass significantly increased the strength of the connection. Analysis of failure mode was also carried out to determine if fiberglass significantly increased the chances of bearing failure instead of block shear. **Table 1** summarizes results of the testing. It can be seen that fiberglass increased the average strength of each fastener group as well as the frequency of bearing failure. It should also be noted that the standard deviation for each group containing fiberglass was lower, meaning that the results were more consistent in the fiberglass group. Statistical analysis confirmed that the increases in strength and bearing failure were statistically significant utilizing a 95 percent confidence level.

Table 3.1 Summary of testing results

Fiberglass	Number of Bolts	Average Max Load	Percent Bearing Failure	STD Deviation	Efficiency
YES	1	7788.022	90	856.7410539	1
NO		7736.882	70	1231.525188	1
YES	2	13710.072	90	3085.40021	0.8802
NO		10494.812	10	4884.909896	0.6782
YES	3	22446.067	80	3016.423008	0.9607
NO		19981.402	40	3724.229339	0.8609

When comparing the results of 2 fasteners versus 3 fasteners, it was noted that the 3-fastener bolting pattern performed more efficiently than the 2-fastener bolting pattern. This phenomenon can be explained due to the fact that the same fixture was utilized for all 3 bolting combinations. Although the NDS specifications for edge and bolt spacing were satisfied, the non-uniform spacing of the 2-bolt pattern created greater stress concentration on the outer region of the samples compared to the 3-bolt and 1-bolt pattern where stresses were evenly distributed throughout the sample. For this reason, the 2-bolt pattern was inherently less efficient than the other bolting patterns. Had a bolting pattern that exhibited an equal edge distance and the center to center spacing, it can be speculated that it would have performed more consistently with the other two bolting patterns. Despite the discrepancy, the 2-bolt pattern actually exaggerated the effects of the fiberglass because of the increased stress concentration. The effects of fiberglass were most apparent when looking at the 2-bolt connection.

Efficiency of the fastener system was the key objective from this study. Assuming the average 1 bolt connection strength as a base unit, the value was multiplied by 2 and 3 to compare to the average strength of the fiberglass and non-fiberglass samples containing 2 and 3 bolts. It was found that the fiberglass was significantly more efficient for both the 2-bolt and 3-bolt configurations.

3.5 Conclusions

The most important takeaway from this study is that utilizing fiberglass in dowel type connections makes buildings safer. The first way that fiberglass reinforcement makes buildings safer is in its ability to induce bearing failure. Bearing type failure can typically be found prior to a catastrophic failure. Contrarily, shear tear out failures occur rapidly and without warning. When shear failure is the expected failure mode, larger factors of safety are placed on the design in order to ensure that failure does not occur. This leads to increased building costs and structures that are far less efficient than structures made of a competing material.

Fiberglass reinforcement increases the efficiency of a structure in multiple ways. A tendency for bearing failure decreases the need for extensive factors of safety. This reduction in necessary material for safe construction will reduce material and shipping costs leading to an overall more efficient structure. Not only does a bearing failure mode require a lower factor of safety, but a smaller standard deviation increases design values of the fastener systems. Within wood construction, a 5th percentile strength value is utilized for design strength of a member or system. When the standard deviation is decreased, the 5th percentile value is significantly increased as the distribution of data is more tightly compacted.

Efficiency is also increased when one considers the overall efficiency per bolt of a connection system. As seen in the results, the efficiency going from 1 to 2 bolts and 1 to 3 bolts increased significantly when adding fiberglass. This means that fewer bolts or smaller bolts could be utilized while achieving the same strength as an unreinforced system. Additionally, a smaller load reducing coefficient could be utilized on fiberglass reinforced members, which would additionally increase the potential efficacy of the system.

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