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Bending Response of Timber Mortise and Tenon Joints Reinforced with Filler-Modules and FRP Gussets

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Bending Response of Timber Mortise and Tenon Joints Reinforced with Filler-Modules and FRP Gussets

Andrew Robert Pacifico

Problem Report submitted to the
College of Engineering and Mineral Resources at
West Virginia University in
partial fulfillment of the requirements
for the degree of

**Master of Science
in
Civil Engineering**

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Morgantown, West Virginia
2019

Keywords: Fiber Reinforced Polymers, composites, gussets, filler-modules, timber framing, mortise and tenon joints, retrofit technique

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ABSTRACT

Bending Response of Timber Mortise and Tenon Joints Reinforced with Filler-Modules and FRP Gussets

Andrew Robert Pacifico

In 2013, the California Bay Area (CBA) passed a set of ordinances to ensure that their 10,000 plus timber soft-story buildings were prepared for seismic events, through nondestructive evaluation methods. Many property owners are searching for an affordable retrofitting system that will also meet CBA's new laws focusing on installations by the mandated deadlines in 2020. Over the past three to four decades, Fiber Reinforced Polymer (FRP) composites have found their way into the civil infrastructure sector for rehabilitation. The objective of this study is to evaluate the bending behavior retrofitted mortise and tenon timber joints reinforced with engineered wood filler-modules and FRP composite gussets. In addition failure modes with filler-modules and/or gussets are evaluated for future studies to develop design methodologies. The above efforts are focused on a West Virginia University (WVU) patented joint performance enhancement system.

A total of ten specimens with varying retrofitting schemes were studied by using a static bending test. A 6" by 6" timber post joining system with a mortise and tenon connection style were used throughout this study. These bending tests are conducted and reported herein to determine the predominate failure modes based off of the inclusion of filler-modules and gussets. The increase in load capacity and energy absorption after retrofitting the joints subjected to bending and shear, and also the limit states on deflections and deformations of timber joints after incorporating the proposed retrofit schemes, are evaluated from the test data and reported herein.

The results from the tests indicate that a joint with filler-modules and a FRP gusset with an enhanced stiffness will perform better than a conventional joint system. The control specimens, i.e. without retrofit schemes, averaged a maximum load of ~2,900 lbs. A retrofitted specimen was able to reach a maximum load of ~16,000 lbs. Conventional beam-column mortise and tenon joints made of 6" x 6" size could only deflect 0.29" at the peak load with a 18" bending span, while a retrofitted joint with filler-modules and gussets was able to reach 0.96" at peak load. Joints retrofit with the proposed scheme also show up to 500% increase in energy absorption.

Mortise and tenon joints have a shear failure on tenon when the dowel is made of a harder wood than the beam and column but will have a dowel bending/shear failure if it is softer than the bearing material. Joints retrofitted with filler-modules were able to prevent the tenon failure but failed once the filler-module on the tension side would debond. To delay the debond, FRP gussets were installed which resulted in FRP rupture failure at the tip of the joint area or beam-column interface.

The retrofit system proposed in this paper displayed an increase in strength, ductility, and energy absorption by factors of about 3 to 5. The data herein proves that the filler-module and gusset combination is an effective retrofitting scheme. The system is an easy to learn installation process which can be implemented cost-effectively in a short amount of time and would save millions, in both the rehabilitation and retrofit costs.

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CHAPTER 1 INTRODUCTION

1.1 Background

Natural disasters such as earthquakes and hurricanes induce extreme lateral forces onto structures and can cause catastrophic failures in structures that are not designed to resist dynamic forces. In the San Francisco area, a highly seismic zone, there are over 10,000 timber soft story buildings (ground floor weaker than above) that are highly vulnerable to failure under seismic forces (The City and County of San Francisco n.d.). An example of a soft story structure can be seen in Figure 1-1. Absence of framing is ideal for open spaces such as apartments, parking or commercial areas; but it results in weakening the lateral resistance of a structure, commonly referred to as a ‘soft story’ structure. Over 115,000 San Francisco residents live in buildings that are classified as a soft story structure. On record the United States averages 1.75 hurricanes every year, and 3 hurricanes that are classified as Category 4 or 5 every five years. Major structural damage, which can be seen in Figure 1-2, renders building uninhabitable until rehabilitation or reconstruction is completed, which can take months to years. The U.S. spent a record \$306 billion in damages from natural disasters in 2017 (Ferris 2018), and \$91 billion in 2018 (Chappell 2019). The high population density of soft story buildings and the high cost of repair after disasters made retrofitting a high priority, as legislated by many communities and municipalities in the Bay – LA area.

Figure 1-1: Example of a Soft Story Building (Rañoa et. al 2015)

Figure 1-2: Soft Story Building Damage from Hurricane Katrina (Pan 2014)

In 2013, the City of San Francisco approved Ordinance 66-13 which mandated many wood-frame buildings to be retrofitted to resist seismic forces and minimize failures such as the ones in Figure 1-3. The ordinance covers soft story structures covering two or more stories over a basement, carrying a minimum of residential units, such as Type V wood framed building, and any building that had the application of permit for original construction prior to January 1, 1978 (The City and County of San Francisco n.d.). The city plans on complete retrofit of deficient structures by late 2020. With many cities and counties following San Francisco's footsteps, the need for a faster, safer, and more economical retrofit solution are needed than the current ones in practice. This void can be filled by using engineered composite wood as filler-modules and also

fiber reinforced polymer (FRP) composite gussets; thus this study focuses on retrofitting joints with engineered wood filler-modules and advanced FRP composite gussets under static loading conditions which can be used as a design equivalent load for quasi-dynamic response of framed structures.

Figure 1-3: Soft Story Building Failure due to a Seismic Event (Arroyo 2015)

The last thirty years have been a boom for FRP composites in the structural sector. FRPs are an attractive alternative to conventional retrofitting techniques due to their extraordinary properties such as lightweight, high strength-to-weight ratio, corrosion resistance, high durability, and high workability (Hollaway & Teng 2008). Designers and contractors have been somewhat reluctant towards using FRPs since it is a relatively new material and has a learning curve to fully appreciate the correct use of FRPs in structural systems. A system is needed using FRP composites that is easy to design with, easy to install, and will improve the performance and safety of a structure.

At West Virginia University (WVU) a team of researchers developed a method of strengthening structural joints that is durable, fire resistant, energy absorbent, cost-effective, and

safe in resisting service forces generated from seismic, hurricane, and blast actions. United States Patents 9,611,667 (GangaRao and Majjigapu 2017), 10/100,542 (GangaRao and Majjigapu 2018), and 16/133,337 (GangaRao and Majjigapu 2019) were award to the team based on a three-part invention using FRP wraps or gussets, reinforcing dowels, and filler-modules. Figure 1-4 illustrates a joint strengthening system with details for potential installation as a retrofit device in a building system requiring seismic force resistance. Filler-modules are blocks of high strength materials that are installed to help reduce the stress concentrations in a joints corner (re-entrant corner) (Majjigapu 2019). Stress concentration is a location at the re-entrant corner of a joint where stress is concentrated, resulting in micro-cracking before a beam or column fails under present design guides. Figure 1-5 shows how stress is concentrated in the corner of a conventional joint, but when filler-modules are installed, the stress is distributed across the entire joint more evenly, as shown in the middle and right side of the finite element simulations of a joint under transverse loading. Also note that filler-module shape impacts the stress distribution. A wedge-shaped filler-module still has stress concentrations at the tip of the modules, but at a much lesser magnitude than a conventional joint with abrupt change in angle of 90° . A curve-shaped filler-module can distribute the stress better than a wedge since there is a smooth transition from beam to column. Significant advances in the WVU inventions are needed before field implementing them as affordable retrofit systems in building or bridge codes across the world.

Figure 1-4: Proposed Retrofit System Installed on a Building System (Majjigapu 2019)

Figure 1-5: Finite Element Simulations Showing Principle Stress (Top Row) and Shear Stress (Bottom Row) Concentration Effect on Varying Filler-Module Shapes Due to Transverse Loading

Throughout the entirety of this study varying joining schemes are used to study their effect on the bending behavior of the timber joints retrofitted with filler-modules and gussets. The shape of the filler-module will be examined to determine if a curve or a wedge can improve on distributing stresses and modify the failure to improve load carrying capacity and energy absorption. A gusset installation is studied in terms of ease of field installation, when compared to a FRP wrap installation. FRP gusset material and configuration (fiber architecture) will be modified during this study to determine which composite configuration (as described in

CHAPTER 3) provides the largest increase in strength, ductility, and energy absorption. Lastly reinforcing dowels will be installed on one joint to see if the dowels are able to contribute to a better force transfer.

1.2 Objectives

The main objective of this study is to complete static bending test on timber mortise and tenon joints, with and without the proposed retrofit scheme, and determine the behavior and failure modes to set up future research in this field.

- Determine what configuration of the proposed retrofit scheme gives the largest increase in strength, ductility, and energy absorption
- Review the current practices of timber framing and the available retrofitting techniques
- Evaluate the behavior and failure modes of the retrofitted joints

1.3 Scope

- Static bending test on mortise and tenon timber joints with and without the proposed retrofit system/scheme
 - 2 control specimens
 - 1 specimen with only filler-modules
 - 2 specimens with different gusset materials
 - 5 specimens with varying combinations of filler-modules and gussets
- Analysis of joint behavior with varying retrofit schemes
- To minimize catastrophic failures. discussion on failure modes with the proposed retrofit schemes is provided with emphasis on failure initiation, failure location, failure type, ductility limit states including peak load resistance and energy absorption

- Make recommendations for future research using the WVU retrofit scheme

1.4 Report Organization

- Chapter 1 introduces the current need for moment resisting retrofit techniques and the WVU retrofit scheme
- Chapter 2 reviews current literature
- Chapter 3 discusses the materials used and explains the testing methodology
- Chapter 4 provides an analysis of the experimental data as a part of this test program collected and an in-depth look into failure modes
- Chapter 5 discusses conclusions and future work

CHAPTER 2 LITERATURE REVIEW

This chapter focuses on a review of timber construction. Furthermore, this chapter provides background information pertaining to the selected jointing systems and their response to loading.

2.1 Timber Construction Introduction

Wood-based products are excellent as construction materials, because they have a high strength to weight ratio and they are green, i.e. renewable source. The use of timber quality is not as high as it used to be due to the depletion of virgin forests and accelerated growth mechanisms of southern yellow pine trees. Timber does not have the attributes in regard to reliability, serviceability, and durability that a designer will get from steel, concrete, and even masonry. Designing with timber takes someone with a significant amount of expertise to work with the highly complex transversely isotropic material (Köhler 2007).

2.1.1 Potential Damages

Structural timber systems are subjected to fire, wind, decay due to moisture and fungi attacks and many other reasons. A few of these adverse effects of reaction of timber to nature's vagaries are given below.

2.1.1.1 Fire

Fire can be a catastrophic event to happen to any building material due to the prolonged exposure to high temperatures. Timber degrades when exposed to the elevated temperatures that fire can produce. At temperatures below 212 °F, the immediate effects of temperature on wood properties are reversible. Under prolonged exposure to temperatures above 150 °F, timber will undergo permanent losses in strength properties (Ross et. al 2005).

Timber does have some natural fire resistance. Wood members can retain some load capacity if the charring effect does not reach a critical depth. Charred wood does not have any residual load capacity; therefore, the new capacity is dependent on the wood beneath the char layer. Figure 2-1 shows how charring effects the outer layers of a wooden member but can still have a solid core.

Figure 2-1: Depiction of Fire Damage on a Timber Cross Section (Ross et. al 2005)

2.1.1.2 Wind

High winds such as hurricanes and tornados cause billions of dollars in damage and hundreds of lives lost each year in the United States. Most damages are on structures that are non-engineered due to dry-rot, termite attacks and others. These buildings are common residential dwellings or office or school buildings. Normally it takes winds over 100 mph to cause widespread damage to buildings, but there have been many instances where structures have been damaged or destroyed by winds of lower speeds due to improper or inadequate joining schemes. Texas Tech University has conducted a large number of investigations due to wind

damage over the last forty years and have made significant impacts on how buildings are designed (Liu et. al 1989). Design codes should be followed to avoid premature failure due to wind damage, like the building in Figure 2-2.

Figure 2-2: High Winds Causing a Building to Tilt (FEMA 2006)

2.1.1.3 Decay

Timber is an organic material, which makes it a great source of energy for microorganisms. Many organisms that can be found in buildings such as mold, fungi, bacteria, and insects are always searching for food. It is easier for these living creatures to survive on timber members when the wood is especially exposed to moisture. The factor that will keep a building from decaying is keeping the timber dry (Singh and White 1997). Decay can be prevented by allowing the timber members to be in a well-ventilated area with low humidity and making sure that the member is protected from leaks (Houspect WA 2015). Chemical treatment of wood with products such as pesticides will also further delay decay actions.

Structural members that are afflicted by decay must be treated immediately or run the risk of needing replacement or a building collapse. As seen in Figure 2-3 a structural member will lose cross sectional area due to micro-organisms eating away at the periphery of wood members.

Once a member reaches its critical load because of the reduced area due to decay damage, then the structure supported that member can collapse, if it can't find alternate load path.

Figure 2-3: Example of Timber Decay on Beam Members (Houspect WA 2015)

2.2 *Timber Moment Connections*

Since typical timber construction is used on non-engineered structures, i.e. houses, schools, and offices, there are standards to connect beams to columns to roof trusses. A typical timber building can have several connection types, which are illustrated in Figure 2-4. The joint that is considered to be vital in any structure is the beam-column joint. The beam-column joint is important, because it is responsible for transferring loads from the flooring system to the column. Figure 2-5 displays a few typical beam-column connections in timber framing systems. Structures that will encounter a large amount of lateral forces will need to be constructed with moment connections, and therefore will be the focus of this study. A rigid joint, or moment connection, must have the ability to fully transfer bending moments and shear forces. The ability to transfer bending moment through a timber connection has been a difficult task due to their fiber orientation. Characteristics of timber such as volume change, low tensile strength

perpendicular to the grain, and low fastener stiffness make timber connections easier to design as pinned supports. Moment connection designs are available (Moses et. al 2016) for timber structures but many modern solutions to timber joint design under seismic and other unusual loads are not code approved currently and can be difficult to acquire.

Figure 2-4: Typical Timber Connection Methods on a Residential Building (FEMA 2006)

Figure 2-5: Typical Timber Beam/Girder Connections to Columns: (a) Girder to Steel Column; (b) Girder to Timber Column; (c) Beam to Pipe Column; (d) Beam to Wood Column, with Steel Strap Welded to Steel Side Plates; (e) Beam to Wood Column, with a T-Plate; (f) Beam-Column Connection with Spiral Dowel and Shear Plates (Merritt and Ricketts 2001)

2.2.1 Dowel-Type Fasteners

Dowel type fastener is a general term that covers nails, screws, dowels, and bolts that transfer load perpendicular to the longitudinal axis. These connections transfer load through connected members by a combination of flexure and shear in the fasteners and shear and bearing in the timber (Gocál 2014). Connections must be looked at both the local and global levels. At the local level, a single fastener must have a sufficient load carrying capacity with the timber it is embedded into. At the global level, spacing and distance between fasteners must be enough to withstand the forces being transferred (Pedersen 2002).

Figure 2-6: Examples of Dowel Type Fasteners: Nails (Top Left) (Encyclopedia Britannica n.d.), Wooden Dowels (Top Right) (Craftsmanspace n.d.), and Screws/Bolts (Bottom) (DIY Extra n.d.)

2.2.1.1 Behavior

The ductile behavior of dowel-type fasteners is based on the strength and stiffness of the connection type and its ability to transfer forces without getting suddenly loosened under repetitive loads, or impact loads resulting in jerking actions. The strength of the connection is modeled based on the dowel effect (the contact pressure between the fastener and the members), friction between connected members, and axial tension force in the fastener. The ultimate carrying capacity will be the summation of those three factors. The stiffness can be determined by many different types of tests, since there is no standard on calculating stiffness in a fastener. Research from many different tests has provided an empirical equation that is only valid for finding the initial stiffness per shear plane for fasteners, which can be seen in Equation 2-1. Another common approach for finding stiffness is an analytical method considering the fastener as a beam on a foundation. Bernoulli's theory (Haukaas 2014) can be applied assuming that the foundation is linear elastic. Kuenzi (Kollmann et. al 1975) applied the elastic foundation theory

on laterally loaded dowel-type fastener connections. Kuenzi determined that the stiffness based off of this theory is depended on the modulus of the timber, the fastener diameter and the force transfer length of the fastener. The stress length is difficult to determine and must be done independently for each test specimen. Van der Put (van der Put and Leijten 2000) used Holographic Interferometry technique to prove that the beam on an elastic foundation method is a potential method to determine the stiffness of the moment connection (Fokkens 2017).

$$K_{ser} = \rho_m^{1.5} * \frac{d}{23}$$

With:

K_{ser} , instantaneous slip modulus [N/mm];

ρ_m , mean density of the timber [kg/m³];

d , fastener diameter [mm].

Equation 2-1

2.2.1.2 Failure Modes

Determining the failure mode for dowel-type fasteners can be done by using Johansen's yield theory (Johansen 1962). According to Johansen the load carrying capacity is dependent on the geometry of the connection, the bending resistance of the fastener, and the embedding strength of the timber material. The design equations used for dowel type fasteners are based on a rigid-plastic behavior of both the fastener under bending moments and the wood under embedding stresses, while also taking into account the plastic moment capacity of the fastener (Aicher and Reinhardt 2000).

There are several possible failure modes, outlined in

Figure 2-7, that happen in a timber joints depending on the connection type and materials used. Mode I is a wood bearing failure that happens in the timber member; mode II is a fastener

rotation without bending; mode III is a combination of bearing failure with a plastic hinge formation; and mode IV is the same as mode III but with more plastic hinges forming. The modes are true for the assumption that the fastener and wood are both perfectly plastic. Axial force on the fastener and friction between members are ignored for these models, since they can significantly increase the capacity. Design of dowel-type fasteners is to be treated as the failure mode that results in the lowest load, i.e. which failure mode would occur first when loading is applied (Rammer 2016).

Figure 2-7: Failure Modes for Dowel-Type Fasteners in (a) Two Member Connections and (b) Three Member Connections (Rammer 2016)

2.2.2 Mortise and Tenon

Mortise and tenon connections have been used for centuries around the world in timber framing. The connections are made entirely out of wood, and therefore were a common joining method before metal fasteners were introduced in timber construction, which can be seen in Figure 2-8. The mortise (grove end) is secured to the tenon (tongue end) with the use of a dowel, as seen in Figure 2-9. Without the dowels, the tenon will slip right out of the mortise and not transfer any load. The two dowel connections will help transfer bending moment, because the tenon will be restricted from rotation by the second dowel (Schmidt and Daniels 1999); thus partially converting bending forces into axial and/or shear forces in the dowels.

Figure 2-8: Mortise and Tenon Connection in a Building System (Vermont Timber Works n.d.)

Figure 2-9: Beam Column Mortise and Tenon Connection (Schmidt and Daniels 1999)

2.2.2.1 Behavior

The behavior of a mortise and tenon connection is once again based on the strength and stiffness of the materials. The connection will behave as a semi-rigid joint, until a bearing or dowel failure initiates, at which point some of the rigidity is lost. The connection will lose all rigidity when the ultimate failure occurs, which can be a ductile or brittle behavior depending on the controlling failure mode (Schmidt and Daniels 1999).

2.2.2.2 Failure Modes

Mortise and tenon connections can experience three different failure modes. The first being a mortise split. A mortise can split if the wall thickness is inadequate to restrict bending/tensile force transfer from column to beam because the dowel material is stiffer than the bearing of the timber. The second mode that could occur is a tenon split. Tenons can split when there is not a sufficient edge distance between the dowel and the end of the beam. The dowel bending or shearing is the third failure mode. This behavior is the same action that is described in section 2.2.1.2 (Schmidt and Daniels 1999).

Figure 2-10: Mortise and Tenon Split (Schmidt and Daniels 1999)

2.2.3 Available Retrofit Methods

Current retrofitting techniques focus on the entire structure, rather than addressing the weak link of beams and columns which is the joint. The practice that many retrofit engineers use to fix weak structures include installing shear walls and adding moment resisting steel frames (Partner Engineering and Science, Inc. n.d.). Figures 2-11 and 2-12 illustrate how invasive a global

retrofit is. The steel frame and shear wall installation are effective retrofit schemes, because they add a higher strength material or more material to transfer lateral loads. FRP rehabilitation is becoming a promising alternative to conventional methods due to the lower cost of repair, ~10% cheaper. A new material proves to be difficult not only for designers to use, but also for the workers installing the system. A new retrofit system that is able to be low cost while maintaining high structural efficiency that is also an easy installation process would integrate into current practices rapidly.

Figure 2-11: Installation of a Steel Moment Resisting Frame on a Soft Story Building (City of Santa Monica n.d.)

Figure 2-12: Shear Wall Layout on a Building (Civilengineer 2017)

CHAPTER 3 EXPERIMENTAL MATERIALS AND METHODOLOGY

3.1 Testing Materials

This section will go over the materials used and the size of throughout the testing. Test specimen, filler-module, and gusset fabrication will be outlined within this section as well. Standard building timber will be used for the joints to simulate a real building system. Engineered wood will be used for the filler modules, due to the higher strength and controlled properties. The FRP gussets will be made out of glass FRP (GFRP) due to the lower cost and high strength-to-weight ratio.

3.1.1 Joint Materials

Testing samples were made using readily available building materials. All timber (yellow pine) were obtained from the local lumber company (Lowe's). Severe Weather Ground Contact pressure treated 6" x 6" posts were used to create the joint seen in Figure 3-1. Note that 6" x 6" is the nominal size of the southern yellow pine (SYP) timber post, while the actual post dimensions are 5.5" x 5.5". The pressure treated wood was made of #2 grade SYP. Dimensions for the test specimens are laid out on Figure 3-1. The length of the beam was limited due to the restrictions of the loading bay. To easily install the test specimens into the loading frame a 20" long beam was used. Mortise and tenons were cut to a very tight tolerance for a very tight fit mortise and tenon joint and were joined together using wood glue. Using Schmidt and Daniels' paper for recommended dowel spacing, Figure 3-2 shows the layout used on all test samples. Dowels were 3/4" diameter oak rods that were cut to 6" length then glued and hammered into a slightly

undersized hole. Timber material properties used for this study were gotten from Southern Pine Reference Design Value and are reported in Table 3-1.

Figure 3-1: Mortise and Tenon Timber Joint Dimensions

Figure 3-2: Mortise and Tenon Dowel Layout

Table 3-1: Design Values for #2 Southern Yellow Pine (Southern Forest Products Association 2013)

Flexural Strength F_b (psi)	Tension Strength Parallel to Grain F_t (psi)	Shear Strength Parallel to Grain F_v (psi)	Compression Strength Perpendicular to Grain $F_{c\perp}$	Compression Strength Parallel to Grain F_c (psi)	Modulus of Elasticity E (psi)	Minimum Modulus of Elasticity E_{min} (psi)
850	550	165	375	525	1,200,000	440,000

3.1.2 Filler-Module Materials

Filler-modules were cut out of 40" x 25" x 5" parallel strand lumber (PSL) modules from Trus Joist Corporation. On testing samples that required a FRP gusset, a 1/4" wood veneer was glued onto each side of the filler-module to close the gap between the gusset and joint, as seen in Figure 3-3. The veneer fillers are important, because the gussets will not be able to bond to the joint if the 1/4" remained, and would lead to a premature failure. Modules are attached to the joint by applying Sikadur®-31 Hi-Mod Gel to the contact areas and using screws as a clamping force. A select amount of test specimens had 1/2" GFRP dowels to secure the filler-module to the timber. The material properties for the PSL module and Sikadur®-31 can be found in Table 3-2 and Table 3-3 respectively.

Figure 3-3: Before and After Attaching Wood Veneer to Filler-Module

Table 3-2: Design Values for PSL (Trus Joist 2018)

Flexural Strength F_b (psi)	Shear Modulus of Elasticity G (psi)	Shear Strength Parallel to Grain F_v (psi)	Compression Strength Perpendicular to Grain $F_{c\perp}$	Compression Strength Parallel to Grain F_c (psi)	Modulus of Elasticity E (psi)	Minimum Modulus of Elasticity E_{min} (psi)
2,900	137,500	290	625	2,900	2,200,000	1,118,190

Table 3-3: Design Values for Sikadur®-31 (Resin) (Sika 2018)

Modulus of Elasticity in Compression E_c (psi)	Flexural Strength F_b (psi)	Modulus of Elasticity in Flexure E_f (psi)	Tensile Strength F_u (psi)	Elongation at Break	Shear Strength F_v (psi)	Pull-off Strength (psi)
795,000	6,100	1,670,000	3,300	0.9%	4,600	420

3.1.3 FRP Gusset Materials

Three different fabrics were used to manufacture gussets. The first set of gussets were made using SikaWrap® Hex-106 G bi-directional fabric while the second set were made using SikaWrap® Pre-saturated 430G. The final set of gussets were made of SikaWrap® Hex-100 G. The fabrics were all wet with Sikadur®-300 epoxy resin. Before curing started, weight was applied on the fabric to ensure that the gusset would be flat while squeezing out extra resin. Gussets were bonded to the test specimens using Sikadur®-330 and Sikadur®-300. #8 x 1” long self-tapping wood screws were used to apply a clamping force onto the gusset. Screws were patterned out such that the gussets were in full contact with the joint. Material properties for the fabrics and resins can be found in Table 3-4 and Table 3-5, respectively.

Table 3-4: Design Values for Glass Fabric (Sika 2018) (Sika n.d.) (Sika 2018)

Fabric Name	Area Density (oz/yd ²)	Tensile Strength F _u (psi)	Modulus of Elasticity E (psi)	Elongation at Break
SikaWrap® Hex-106 G	9.6	65,600	4,240,000	1.45%
SikaWrap® Pre-saturated 430G	13	51,328	4,357,548	1.4%
SikaWrap® Hex-100 G	27	78,400	3,970,000	N/A

Table 3-5: Design Values for Sikadur® Resins (Sika 2018) (Sika 2018)

Resin Name	Flexure Strength F _b (psi)	Modulus of Elasticity in Flexure E _f (psi)	Tensile Strength F _u (psi)	Modulus of Elasticity E (psi)	Elongation at Break
Sikadur®-300	11,500	500,000	8,000	250,000	3%
Sikadur®-330	8,800	506,00	4,900	N/A	1.2%

3.2 *Filler-Module/Gusset Retrofit Schemes*

Simplicity is key when retrofitting existing buildings. Real world applications were taken into consideration when deciding what schemes would be tested.

3.2.1 Filler-Module Schemes

Filler-modules are designed based on the beam depth at the joint which is being reinforced for improved energy absorption and enhanced load capacity. The length of a beam that is most susceptible to shear failure is equal to the depth of the beam away from the face of the column. Since the nominal depth of the timber beam is six inches, the minimum filler-module leg length should be equal to that depth. Seen in Figure 3-4, the filler-module dimensions for both the curve and wedge are provided. Note for the curve module that there is a 1/8th inch lip, which is provided for added stiffness to the tip. Fiber orientation of the PSL was considered for this study, i.e., if the module would be rotated to a sub-optimal direction the module will split along the glue lines which is the weak plane of failure. It was determined that placing the fibers on the 45° would be the optimal orientation. Figure 3-5 shows an engineered wood wedge module cut to size and rotated to the correct angle. For specimens that included dowels, one 5/8" hole is drilled in the center of each face of the filler-module, as seen in Figure 3-6. Note that the holes do overlap on the long side of the module, but the reinforcing dowels are cut 1/4" shorter than the entire length of the hole, to ensure that once the first reinforcing dowel is installed it would not interfere with the installation of the second dowel.

Figure 3-4: Filler-Module Dimensions

Figure 3-5: Wedge Shaped Filler-Module (45°) – Engineered Wood

Figure 3-6: FRP Dowel Layout on Filler-Module

3.2.2 FRP Gusset Schemes

The gussets were made in three stages, making improvements after every stage. The first stage consisted of two layers of SikaWrap® Hex-106 G placed in the $\pm 45^\circ$ orientation rather than the $0^\circ/90^\circ$. The second set of gussets were made using SikaWrap® Pre-saturated 430G. Needing fibers in more directions, the second set consisted of a layer in the $0^\circ/90^\circ/45^\circ/-45^\circ$, making a quadriaxial lamina. The third set of gussets had the same fiber architecture as gusset set 2, but differed due to the use of SikaWrap® Hex-100 G fabric. Figure 3-7 shows an enlarged layout of the fiber architecture for all the gussets. Gussets were cut to 3" past the tip of the joint area. Figure 3-8 shows the dimensions for all gusset shapes.

Figure 3-7: FRP Gusset Fiber Orientation

Figure 3-8: Dimensions of all Gusset Sizes

Additional layers of FRP were added depending on the testing scheme. Anchors are an important addition onto this system, because they provide confinement of the gusset to structural members (columns and beams) while also providing extra protection from gusset debond. Every

specimen that had a gusset bonded to it, also had a beam anchor attached, as seen in Figure 3-9. Beam anchors are a 3” strip of FRP that wrap around the beam twice at the tip of the filler-module overlapping the beam element and the composite. Figure 3-10 shows the layout when filler-module anchors are added. The filler-module anchor is a piece of FRP cut to width of the arc length/hypotenuse of the module and wrapped around the back. There is one layer of fabric on both the top and bottom modules. Two layers of longitudinal wrap can be applied on joints starting at an end of the column working its way over the filler-module and ending on the beam, as seen in Figure 3-11. Two layers of FRP are used as longitudinal wraps and are applied to both the top and bottom faces of the beam. When a longitudinal wrap is bonded onto a specimen, column anchors are also attached to the top and bottom 3” of the column. Column anchors are similar to beam anchors, because they both wrap around their member twice. The hand layup of FRP glass fabric layers are made with the same materials as the gussets, and orientated in the 0° direction for the highest confinement effect.

Figure 3-9: Beam Anchor Setup

Figure 3-10: Filler-Module Wrap Setup

Figure 3-11: Longitudinal FRP Wrap Setup

3.3 Testing Specimens

The testing specimens will have varying retrofitting schemes. Table 3-6 outlines the 10 testing samples, and why they are being tested.

Table 3-6: Testing Specimen Breakdown

Specimen Number	Filler-Module (Shape)	FRP Gusset (Fabric)	Reason for Testing
1	No	No	Control
2	No	No	Control
3	Yes (Curve)	No	Filler-Module Effect
4	No	Yes (SikaWrap® Hex-106 G)	Gusset Effect
5	Yes (Curve)	Yes (SikaWrap® Hex-106 G)	Filler-Module and Gusset Effect
6	Yes (Curve)	Yes (SikaWrap® Hex-106 G)	Filler-Module, Gusset, and Filler-Module Anchor Effect
7	Yes (Wedge)	Yes (SikaWrap® Pre-saturated 430G)	Filler-Module and Improved Gusset Effect
8	No	Yes (SikaWrap® Hex-100 G)	Improved Gusset Effect
9	Yes (Wedge)	Yes (SikaWrap® Hex-100 G)	Filler-Module, Improved, and Longitudinal Wrap Gusset Effect
10	Yes (Wedge)	Yes (SikaWrap® Hex-100 G)	Filler-Module, Improved, Longitudinal Wrap Gusset, and Filler-Module Dowels Effect

3.4 Test Setup

The joint specimens were placed into a steel testing frame that is attached to a strong floor of WVU's Major Units Lab, as seen in Figure 3-12. Two hydraulic hand pump actuators applied the load onto the beam and column. A forty and ten kip load cells were placed on the beam and column respectively under their actuator. The beam load was applied 18" from the face of the column, or 2" from the end of the beam. 18" was selected as the loading distance, because that is about the distance to an inflection point on an 8' beam.

Figure 3-12: Beam-Column Joint Testing Setup

3.5 Instrumentation

A combination of strain gages and linear variable differential transformers (LVDT) were used throughout this study. Strain gages, obtained from Micro-Measurements, were applied on the FRP gusset and on the filler-module. Three $\frac{1}{4}$ " strain gages were applied on to the FRP gusset in the $0^\circ/45^\circ/90^\circ$ directions (rosette setup) on the tip of the joint area, as seen in Figure 3-13, to obtain the principal strains at that location. The tip of the joint area is an important location to study, because of its high stress concentration from the applied loads. For test specimens that had filler-modules, a 1" strain gage was put on the center of each module as seen in Figure 3-14. For specimens that had an FRP layer on top of the filler-module a quarter inch gage was used, while a one inch long gage was used when it was applied directly onto the PSL surface. The strain gage on the filler-module will help indicate when the filler-module debonds and how it is behaving before debonding occurs. A 4" LVDT was placed directly under the beam

at or near the vertical load application to measure vertical deflections. For joints that had no FRP gussets attached, two 1" spring loaded LVDTs were attached onto the timber surface, as shown in Figure 3-15.

Figure 3-13: Strain Gage Location and Orientation on the FRP Gusset

Figure 3-14: Strain Gage Location on the Filler-Modules

Figure 3-15: LVDT Locations

3.6 Loading

A two-step loading process was used for the duration of this study. The first step consisted of applying a load to the column. The column load is important, because it simulates the real-world action of load transfer from other stories. Seven thousand pounds of load was applied onto the column to simulate the transfer of load through a building system. The column load was found by taking ~10% of the failure load of SYP. The second loading step was the beam load. Loading is completed when the joint fails. Load rate was not at a set speed, because loading was applied thru hand pump and it is difficult to achieve a constant rate of loading.

CHAPTER 4 TEST RESULTS AND EVALUATION

4.1 Introduction

This chapter presents the test data in terms of load versus strains or deflections, and discussion of data in terms of strength, ductility, and energy absorption. Since this is a preliminary study, a detailed evaluation of failure modes will be the emphasis of the limited testing conducted as a part of this study to help future research in establishing more accurate design and ultimate joint failure values resisting beam and column loads. However, preliminary analyses is carried out to find theoretical failure load values. Appendix A contains load vs strain and load vs deflection plots for each test specimen.

4.2 Test Specimens

A total of ten specimens have been tested within this study. Two joints were un-retrofitted (base line) to compile control results. The remaining eight joints have varying configurations of filler-module and FRP gusset schemes. The dimensions and layout of retrofitting schemes are detailed in CHAPTER 3. A detailed breakdown of test specimens is presented in Table 4-1.

Table 4-1: Testing Specimen Breakdown

Specimen Number	Filler-Module (Shape)	FRP Gusset (Fabric)	Additional Information
1	No	No	N/A
2	No	No	N/A
3	Yes (Curve)	No	N/A
4	No	Yes (SikaWrap® Hex-106 G)	Beam anchor using SikaWrap® Hex-106 G
5	Yes (Curve)	Yes (SikaWrap® Hex-106 G)	Beam anchor using SikaWrap® Hex-106 G
6	Yes (Curve)	Yes (SikaWrap® Hex-106 G)	Beam and filler-module anchors using SikaWrap® Hex-106 G
7	Yes (Wedge)	Yes (SikaWrap® Pre-saturated 430G)	Beam anchor using SikaWrap® Hex-106 G
8	No	Yes (SikaWrap® Hex-100 G)	Beam anchor using SikaWrap® Hex-100 G
9	Yes (Wedge)	Yes (SikaWrap® Hex-100 G)	Longitudinal wrap, beam anchor, and column anchors using SikaWrap® Hex-100 G
10	Yes (Wedge)	Yes (SikaWrap® Hex-100 G)	Longitudinal wrap, beam anchor, and column anchors using SikaWrap® Hex-100 G. GFRP dowels used

4.3 Results

The results from the loading process, outlined in Section 3.6, are presented in Table 4-2. The loading was stopped once the joint failed. Failure was defined as an increase in deflection while gradual decrease or plateau in the applied beam load. Comparison plots for load vs deflection, joint strains (from the rosette strain gages that were described in CHAPTER 3), and filler-module strains can be seen below in Figure 4-1 through Figure 4-7. Principal strain was

calculated using Equation 4-1 (Garmoll n.d.) and the collected data from the rosette strain gages. Principal strain was found to study the combined effect of loading. For the first specimen, the LVDT (deflection monitor) malfunctioned and therefore the deflection readings have been left out of this report.

Table 4-2: Joint Test Results – Failure Load vs Deflection

Specimen Number	Max Beam Load (lbs)	Beam Deflection at Max Load (in)
1	2,935.18	N/A
2	2,820.01	0.29
3	7,420.20	0.58
4	2,796.97	0.36
5	8,094.77	0.48
6	7,663.71	0.51
7	11,691.35	0.69
8	4,708.79	0.64
9	15,166.15	0.96
10	15,932.87	0.96

Figure 4-1: Load vs Beam Deflection under Beam Loading

Figure 4-2: Load vs Principal Strain (Tension) for Specimens 4, 5, 6, and 8

Figure 4-3: Load vs Principal Strain (Tension) for Specimens 7, 9, and 10

Figure 4-4: Load vs Principal Strain (Compression) for Specimens 4, 5, 6, and 8

Figure 4-5: Load vs Principal Strain (Compression) for Specimens 7, 9, and 10

$$\varepsilon_{1,2} = \frac{\varepsilon_0 + \varepsilon_{90}}{2} \pm \frac{1}{\sqrt{2}} \sqrt{(\varepsilon_0 - \varepsilon_{45})^2 + (\varepsilon_{45} - \varepsilon_{90})^2}$$

Equation 4-1 (Garmoll n.d.)

Figure 4-6: Load vs Top Filler-Module Strain

Figure 4-7: Load vs Bottom Filler-Module Strain

4.4 Discussions

This section discusses the joint behavior based on strength, ductility, and energy absorption. A comparison to the control specimens will show improvements made by the retrofitting technique. Strength, ductility, and energy absorption are all important factors when designing and retrofitting a structure, and therefore the proposed system should show improvements in all three categories.

4.4.1 Strength

Strength is an important factor when designing structures. A designer needs to know how much force a structural system can take before it fails to transfer any more load. Table 4-3 shows the strength increase for all specimens compared to the average control strength.

Table 4-3: Strength Comparison for All Specimens

Specimen Number	Max Beam Load (lbs)	Percent Increase (%)
1	2,935.18	N/A
2	2,820.01	N/A
3	7,420.20	158
4	2,796.97	-3
5	8,094.77	181
6	7,663.71	166
7	11,691.35	306
8	4,708.79	64
9	15,166.15	427
10	15,932.87	454

The percent increase in strength varied greatly during the testing. Samples with the SikaWrap® Hex-106 G gusset did not show a large increase in strength compared to the other two gusset materials. Specimen 4 is within 3% of the max load compared to the control specimen, and specimens 5 and 6 are within 8% of specimen 3. Figure 4-1 shows how the specimens with the SikaWrap® Hex-106 G gusset do not gain strength, when compared to

similar retrofit schemes without gussets. The FRP of these plates rupture at the high stress concentration area of the joint's tip (re-entrant corner location), but no delay in failure (or increase in failure load) was achieved by adding these thin plates. Such minimal strength increase led to conclude that the SikaWrap® Hex-106 G gussets did not contribute enough reinforcement to the joint to justify adding a very thin FRP gusset; therefore a stiffer gusset would be needed to improve joint strength.

Specimen 7 showed an increase of 306% over the control specimens. The stiffer gusset worked better than the gusset in Specimens 5 and 6, but the increase was still lower than expected, so another adjustment was made. Specimens 9 and 10 were adjusted from the strength-stiffness response in the earlier samples; thus another major increase of strength was obtained with increase in gusset stiffness. The maximum increase in strength obtained during this study was 453%. Specimen 7, 9, and 10 displayed the largest increase in strength, and it is evident from Figure 4-1 where their respective load-deflection responses exhibit maximum loads. The failure mode seen in these specimens is the same as the ones with thinner gussets. Using the same gussets that gave the highest strength increase, a sample without filler-modules was tested to see how best gusset alone would help. Specimen 8 shows a 64% increase in strength over the control specimen, which is a great step up from the control samples. The stiffer gusset also was able to change the failure mode and help delay the joint from failing. The failure still occurred at the tip of the joint area, which is to be expected since that is a high stress concentration area.

The reinforcing dowels inserted on Specimen 10 did not add strength to the system. There was only a 5% increase of strength from Specimen 9 to Specimen 10, which shared the same retrofit scheme except for the inclusion of reinforcing dowels. The shape of the filler-

module also did not add much strength to the system, because the failure of the specimens was not on the filler-module.

After the testing was concluded, we can clearly see that filler-modules contributed to the increase of strength. The addition of FRP gussets works best when in tandem with the modules. Figure 4-1 shows the specimens with filler-modules had a taller curve than those without modules, but the tallest curves were those of specimens with both stiff gussets and filler-modules.

4.4.2 Ductility

Ductility is an important factor to study, because a ductile failure does not lead to a sudden collapse. To find the ductility of the test specimens, deflection at the maximum deformation capacity for a minimum residual load resistance. Specimens were not tested to their maximum deformation capacity, so Table 4-4 was put together for deflection 25% past the peak load.

Table 4-4: Ductility Comparison for All Specimens (*Did Not Reach 25% Past Peak Load)

Specimen Number	Ductility (in)	Percent Increase (%)
1	N/A	N/A
2	0.36	N/A
3	0.73	103
4	0.44	22
5	0.60	67
6	0.52*	44
7	0.94	161
8	0.79	119
9	1.20	233
10	1.20	233

All retrofitted specimens saw an increase in ductility. Specimens 4 and 8, joints retrofitted with gussets only, showed that the gussets help improve the ductility response of a

joint. An enhanced stiffness of gusset would greatly increase the ductility, an 80% increase of ductility from Specimen 4 to Specimen 8. Filler-modules added almost the same amount of ductility as a stiff gusset would. Specimen 8 only had an 8% increase of ductility when compared to Specimen 3.

Specimens that had both gussets and filler-modules showed the largest increase in ductility. A maximum of 233% increase in ductility was achieved by using filler-modules with a high stiffness gusset. Since the proposed retrofit scheme was able to delay the internal failure of the tenon on a conventional joint, the system was able to have a higher deflection at the peak load.

The shape of the filler-module and the inclusion of reinforcing dowels did not contribute to the ductility of the system. Gusset material on the other hand did greatly improve the ductility. The ductility increases in this study can be attributed to the ability of the retrofit system delaying the failure, and pushing the failure mode onto the system instead of locally affecting the joint.

4.4.3 Energy Absorption

Energy absorption is a good indication of how well a structure can resist until the ultimate failure. Energy absorption can be found as the area under the load-deflection curve. Since failure was not well defined at the start of this study, not all specimens were able to reach their own ultimate failure. Because of this the energy absorption for this report will be limited to 25% past the peak load. Table 4-5 displays the energy absorption for each testing specimen and the percent increase from the control specimen. Energy absorption was found in Excel using the trapezoidal method for finding the area under the curve.

Table 4-5: Energy Absorption Comparison for All Specimens (*Did Not Reach 25% Past Peak Load)

Specimen Number	Energy Absorption (lbs-in)	Percent Increase (%)
1	N/A	N/A
2	923.48	N/A
3	3,347.71	263
4	804.45	-13
5	3,594.10	289
6	2,700.10*	192
7	7,253.79	658
8	2,721.21	195
9	12,400.04	1,243
10	13,516.36	1,364

The majority of joints showed an increase in energy absorption. The one outlier is Specimen 4, which lost 13% of its energy absorption abilities. This could be expected, since Specimen 4 behaved like the control joints. Another joint that had a lower energy absorption than expected is Specimen 6, but this can be contributed to the test being concluded before reaching 25% deflection past the peak load. The specimens with the SikaWrap® Hex-106 G all behaved as if no gussets were attached.

Specimen 7 saw the first large jump in energy absorption, with a 658% increase over the control specimens. Knowing that a stiffer gusset is needed, Specimen 8 was tested to see how only an improved gusset would behave. With almost a 200% increase in energy absorption over conventional joints, Specimen 8 demonstrated how FRP is an excellent retrofitting material.

The improved specimens of 9 and 10 once again show the highest change. Over 1,350% more energy absorption than that of the control specimen, and an 86% increase compared to a similar testing scheme in specimen 7. The enhanced stiffness of the gussets and the inclusion of the longitudinal wrap on specimens 9 and 10 are the reasons why the system was able to absorb more energy than the conventional (control) joints.

Energy absorption comparison for this study was not perfect since the testing did not go to the ultimate failure, but the 25% method illustrates that a strong FRP gusset paired with filler-modules drastically increases the absorption ability.

4.5 *Failure Modes*

The main goal of this research is to investigate failure modes for future development/improvement of joint integrity, energy absorption, and avoiding catastrophic failures of structures. Thus, this section will go into great details to narrow down the failure mode for every specimen.

4.5.1 Control Specimens

The control joints behaved as expected. The “two-dowel” connection was reacting as a rigid joint, until the dowels started to bend. The peg bending is not the ultimate failure, because the joint was still reacting as a semi-rigid connection, and thus would continue to transfer load. It was until the ultimate failure of the tenon that the connection acted as a hinge. No more load beyond ~2,800 lbs was transferred through the connection, but deflection was continuing to increase without any load increase. Figure 4-8 and Figure 4-9 show the beam cross-section pullout away from the column by a large horizontal displacement. In those figures there is an almost infinite slope when loading begins and a change in slope until the joint fails. The ultimate failure of the joints happened due to shear failure on the tenon. As seen in Figure 4-10 the tenon splits and is no longer restrained by the dowels.

Figure 4-8: Load vs Tenon Pullout for Specimen 1

Figure 4-9: Load vs Tenon Pullout for Specimen 2

Figure 4-10: Failure Mode for the Control Specimens (Specimen 2)

Figure 4-11: Specimen 1 after Testing

Figure 4-12: Close up of the Tenon Pull Away from Mortise on Specimen 2

4.5.2 Retrofitted Specimens

4.5.2.1 Specimen 3

Upon continuous increase in loading, joint 3 went through a two-step failure process. The initial failure occurs when the top filler-module starts to peel off from the beam which occurred at ~5,000 lbs, which can be seen in Figure 4-14. The peel off starts at the tip of the module and continues to grow towards the face of the column. The joint acted as a rigid connection up until the filler-module pull off from the column at the maximum load. The filler-module was able to pull off from the column because the module was not fully debonded from the beam and was still acting as a single unit. Once the top filler-module pulled off from the column, the joint acted as a conventional connection and deflection continued to grow while the load plateaued at ~4,000 lbs. Figure 4-13 shows the final failure of specimen 3.

Figure 4-13: Failure Mode of Specimen 3

Figure 4-14: Load vs Top Module Deflection for Specimen 3

4.5.2.2 Specimen 4

Specimen 4 showed an external failure in the joint's top corner on the FRP gusset, as seen in Figure 4-15. As discussed in section 4.4 the FRP gusset used in this test did not contribute to any noticeable property, such as strength, ductility, or energy absorption. Therefore,

the joint behaved just like the conventional specimens and had an internal failure on the tenon with the added failure of FRP rupture. The failure of the tenon was consistent with that of the control specimens.

Figure 4-15: Failure Mode of Specimen 4

4.5.2.3 Specimen 5

Specimen 5 had rupture similar to specimen 3 on the FRP gusset as well. The gusset rupture occurred at the center of the tension side filler-module, as seen in Figure 4-16. The crack in the gusset happens on the module rather than the tip of the module, because there was no bond between the gusset and the tip of the module at the time of the initial module peel off, so the filler module was able to peel off with no resistance from the gusset. Then the peel off stops when it reaches a point where the FRP gusset is well bonded to the module. When the module still had some integrity with the beam thru bond forces, then the FRP ruptured. Similarly to the beam side, there was no bond between the filler-module and gusset near the tip on the column. Finally, once the loading reaches the peak value the filler-module pulls off from the column and

the joint fails and no longer behaves as a near rigid connection. This failure mode could be pushed back to the tip of the filler-module if better bond between filler-module and gusset was achieved.

Figure 4-16: Failure Mode of Specimen 5

4.5.2.4 Specimen 6

Specimen 6 had an FRP rupture failure at the tip of the top filler-module which can be seen in Figure 4-17. The addition of filler-module anchors on Specimen 6 was able to delay the filler-module peel off until a higher load of ~7,000 lbs is obtained, as seen in Figure 4-18. During testing, the joint acted as a rigid connection. The FRP ruptured at the 7,000 lbs level and the filler-module started to peel off. The test was concluded before the crack in the FRP gusset could progress too far and thus the filler-module was not able to pull off from the column. The joint was able to behave as a near rigid connection through the entire testing.

The joint did not have an increased strength when compared to Specimens 3 or 5, even though the module peel off was delayed until a higher load. This is due to the fact that the anchor wraps and gussets were very thin and did not have high stiffness to prevent failure in the wrap.

Figure 4-17: Failure Mode of Specimen 6

Figure 4-18: Module Debond on Specimen 6

4.5.2.5 Specimen 7

Predominant failure mode of Specimen 7 was FRP rupture at the top filler-module's tip, as seen in Figure 4-19. The joint was rigid until the ultimate load is reached; then the specimen behaved as a hinge. The load-deflection curve shows a change of slope at ~4,000 lbs, which correlates to the initiation of the top filler-module starting to peel off, which can be seen in Figure 4-20. The gussets transfer the load up to the peak load, which is the ultimate load capacity for the gussets. The filler-module did not pull off from the column, but the load decreased dramatically while still increasing in deflection. Thus, the filler-module must have peeled off completely from the beam and the joint no longer behaves as a rigid connection.

Figure 4-19: Failure Mode of Specimen 7

Figure 4-20: Top Filler-Module Tip Debond on Specimen 7

4.5.2.6 Specimen 8

Specimen 8 had a better behavior than that of Specimen 4, in terms of strength, ductility, and energy absorption. At ~2,000 lbs Specimen 8 is no longer rigid and switches to a semi-rigid connection. This can be seen in the joint strain plots, as a sudden change of slope is noted in the load-strain plots of Figure 4-2 and Figure 4-4. Then the joint will share the load with the FRP gussets and continued to act as a semi-rigid connection until the gussets rupture as displayed in Figure 4-21. At the ultimate load the gussets ruptured, refer to Figure 4-21, and the tenon can no longer support the load and fails internally at which point the connection turns into a hinge type mechanism.

Figure 4-21: Failure Mode of Specimen 8

4.5.2.7 Specimen 9

Specimen 9 with wedge shaped filler-modules preformed the best out of all the samples. The results showed that Specimen 9 was very rigid until the ~5,000 lbs level, at which point the FRP gussets help distribute the load. The tension side filler-module does not start taking load until ~7,000 lbs. The joint, still behaving as a single unit, and takes up to ~10,500 lbs before the filler-module starts to peel off from the column. Once the peel off initiates the FRP gussets take and distribute the load until the ultimate capacity of the gusset. The FRP gusset ruptures at the tip of the top filler-module, where a high stress concentration is located from the maximum load of 15,100 lbs. When the peak load is reached, then the load resistance drops to ~10,000 lbs, and the crack in the FRP continues to grow at ~45° towards the bottom filler-module. The filler-module did not pull off from the column, therefore the joint would still be able to resist additional loads and moments. The failure mode of Specimen 9 can be seen in Figure 4-22. The increase in gusset stiffness helps keep the filler-module resist the pull off forces.

Figure 4-22: Failure Mode of Specimen 9

4.5.2.8 Specimen 10

The behavior on Specimen 10 varied from that of Specimen 9. Both joints of specimens 9 and 10 acted as rigid connections until ~40% of their maximum load, but on Specimen 10, the top filler-module started taking load soon after applied load initiation. Since the filler-module had to distribute load sooner, the peel off also occurred at the lower level of loading, i.e. ~5,000 lbs. The reinforcing dowels in the filler-module could affect why the peel off occurred at an earlier load level. The dowels split the filler-module into section, and with the reduced area along the glue line, the filler-module peel off at a lower stress compared to a filler-module with no reinforcing dowels. The FRP gussets had to distribute the load until failure, at which point the FRP ruptured at the tip of the filler-module. Once again though the filler-module did not pull off from the column, which indicates that the joint still behaved as a rigid connection until the module fully peels off from the beam or the pulls off from the column occurs. Figure 4-23 shows

the FRP rupture, but also an area where the gusset buckled after the initial failure, as seen in Figure 4-24. The lighter area on the bottom filler-module is where the gusset debonded from the joint due to inter-planar shear forces and resulted in a buckling effect.

Figure 4-23: Failure Mode of Specimen 10

Figure 4-24: Buckling Effect on the Bottom Filler-Module of Specimen 10

4.6 Analytical Prediction

This section deals with strength predictions using principal stress theory and compared these predictions with the experimental results. A comparison will be made to material properties especially with reference to the principal bending and shear stresses, and also bond stress based on the failure modes described in section 4.5.

Table 4-6 displays the predicted values for failure stress, which are described in sections 4.6.1 thru 4.6.4, based on their failure modes. These predicted failure stresses are compared to the failing member's ultimate strength in bending or bond. Analytical predictions for Specimens 2, 3, 7, and 8 are examined in sections 4.6.1 thru 4.6.4 and compared with the experimental values.

Table 4-6: Analytical Prediction of Joint Strength vs Material Property Strength

Specimen Number	Predicted Failure Strength (psi)	Material Failure Strength (psi) (as per manufacture)	Failure Mode
2	2,157.35	1,500–1,600	Tenon Shear
3	1,136.19	1,260–1,680	Filler-Module Peel off
7	25,721.6	25,000–30,000	FRP Rupture
8	20,224.8	25,000–30,000	FRP Rupture

4.6.1 Conventional Joints

The control specimens were subjected to a combined effect due to the loading. Bending, shear, and axial forces were all affecting the failure of the joints. To determine the combined effect of the bending and shear, the Mohr's circle approach was taken wherein principal stress are determined for our test specimens subjected to bending and shear loads as shown in Figure 3-12. Equation 4-2 thru Equation 4-4 (Doane 2018) were used to calculate the Mohr's circle using the input values from Table 4-7.

$$\sigma_{1,2} = \frac{Mc}{2I} \pm \sqrt{\left(\frac{Mc}{2I}\right)^2 + \left(\frac{VQ}{It}\right)^2} \quad \text{Equation 4-2}$$

$$\tau_{max} = \sqrt{\left(\frac{Mc}{2I}\right)^2 + \left(\frac{VQ}{It}\right)^2} \quad \text{Equation 4-3}$$

$$\theta = \frac{\tan^{-1}\left(\frac{\frac{VQ}{It}}{\frac{Mc}{I}}\right)}{2} \quad \text{Equation 4-4}$$

Table 4-7: Input Values for Specimen 2 Prediction

Experimental Failure Load (lbs)	Loading Distance (in)	Tenon Depth (in)	Tenon Thickness (in)	Stress Level from Top of Beam (in)
2,820.01	18	5.5	1.5	1

Table 4-8: Mohr's Circle Results for Sample 2

σ_1 (psi)	σ_2 (psi)	$\tau_{\max}$ (psi)	θ (degrees)
4,293.01	-21.68	2,157.35	2.04

The results, seen in Table 4-7: Input Values for Specimen 2 Prediction

Experimental Failure Load (lbs)	Loading Distance (in)	Tenon Depth (in)	Tenon Thickness (in)	Stress Level from Top of Beam (in)
2,820.01	18	5.5	1.5	1

Table 4-8, from the control specimens resulted in a shear stress that is higher than the ultimate value for SYP. On average SYP fails under shear at ~1,500 to 1,600 psi (Forest Products Laboratory 2010), roughly 46% lower than the shear stress obtained in this study. Three factors could contribute to the higher shear stress. Timber is a highly variable material, and the numbers in the Wood Handbook's Table 5-3b (Forest Products Laboratory 2010) are average values in shear stress failure. The timber used in the study could be a higher grade material. Secondly the moisture content of the test specimens could be lower than the 12% moisture content that the handbook reports on because of dry conditions in WVU's Major Units Lab. A lower moisture content, obtained by having the test specimens stored in a dry area, increases material properties of timber. Lastly the tenon could have experiencing a confinement effect from the mortise. The tight fit of the mortise and tenon connection due to good bond thru adhesive along with the applied axial load on the column helps keep the joint tight even with the reduced thickness of the tenon inside of the column. Once the tenon starts to pull out from the mortise, then the shear stress is able to act on the reduced area of the tenon.

4.6.2 Joints Retrofitted with Filler-Modules

Since the specimen with filler-modules had a debond failure mode, the bending stress at the tip of the top module must be examined. The failure stress from the experimental results are compared with the bond strength of the bonding material (Sikadur®-31). Equation 4-5 will be used to find the bending stress, and Equation 4-6 is used to determine the stress concentration at the filler-module tip location.

$$\sigma = \frac{Mc}{I}$$

Equation 4-5

$$k = \frac{\sigma_{beam}}{\sigma_{FM}}$$

Equation 4-6

Using the results from Specimen 3, the analytical results are presented in Table 4-9. The stress was found on the top face of the beam, at the load when the filler-module starts to debond (5,000 lbs).

Table 4-9: Specimen with Filler-Module Analytical Results

σ_{beam} (psi)	σ_{FM} (psi)	k
2,163.78	1,893.65	1.14

That stress at the filler-module is well above the bond strength of Sikadur®-31, 420 psi. The manufacturer's number is lower, because it is a design value and not an ultimate value. The ultimate value can be assumed to be 3-4 times larger than the design value. A stress distribution factor can be applied to the stress at the filler-module of ~0.6. Using Equation 4-7, the stress is reduced to 1,136.19 psi. The stress with the stress distribution factor is close to the assumed bond strength of the Sikadur®-31, 1,260–1,680 psi. The predicted value is a bit lower could be due to

the materials that the glue was bonded to. Sikadur®-31 is recommended for concrete application, but for this study it was used to bond two timber materials together.

$$\sigma = \sigma_{FM} * Z \quad \text{Equation 4-7}$$

4.6.3 Joints Retrofitted with FRP Gussets

Since the failure occurred on the FRP gusset, the analytical strength should be compared to the failure stress of the gusset. Using Equation 4-5 with the values in Table 4-10 the predicted strength at the reentrant corner can be found.

Table 4-10: Input Values for Specimen 8 Prediction

Load (lbs)	Loading Distance (in)	Beam Depth (in)	Beam Thickness (in)
4,708.79	18	5.5	5.5

GFRP has a typical ultimate tensile strength of ~25,000–30,000 psi (Creative Pultrusions 2004). The bending stress equation gives the maximum bending stress to be 3,056.65 psi, but that is the stress on the beam and not the FRP. To convert the beam stress on to the FRP gusset the modular ratio must be used. Using Equation 4-8, a modular ratio of 3.308 was obtained, as seen in Table 4-11. Applying the modular ratio onto the bending stress, the stress on the gusset is 10,112.4 psi. A stress concentration factor is needed, since the change from beam to column is a 90° angle. A right angle corner usually has a stress concentration factor of ~2.0, which when applied onto the predicted value would increase it to 20,224.8 psi. The predicted value is now within 20% of the ultimate strength of the FRP. The difference can come from a handful of variable, such as voids in the gusset, uneven fiber distribution, bending modulus of wood higher than 1.2 Msi, etc.

$$n = \frac{E_{FRP}}{E_{wood}} \quad \text{Equation 4-8}$$

Table 4-11: Modular Ratio Calculation for Specimen 8

E_{FRP} (psi)	E_{wood} (psi)	n
3,970,000	1,200,000	3.308

4.6.4 Joints Retrofitted with Filler-Modules and FRP Gussets

Following the same method as the joints with just gussets, Equation 4-5 and Equation 4-8 were used to predict the failure stress of joints with filler-modules and FRP gussets. Specimen 7 was looked at for this section, because it had no other added variables such longitudinal wraps, reinforcing dowels, or filler-module anchors. Table 4-12 and Table 4-13 display the information needed to predict the failure strength of Specimen 7.

Table 4-12: Input Values for Specimen 7 Prediction

Load (lbs)	Loading Distance (in)	Beam Depth (in)	Beam Thickness (in)
11,691.35	12	5.5	5.5

Table 4-13: Modular Ratio Calculation for Specimen 7

E_{FRP} (psi)	E_{wood} (psi)	n
4,357,548	1,200,000	3.63

Using the bending stress equation (Equation 4-5) and the modular ratio from Table 4-13, the resulting bending stress at the tip of the filler-module on the FRP gusset is 18,372.57 psi. A stress concentration factor is still needed, but it will be lower than the 2.0 used on a 90° angle. Since wedge-shaped filler-modules were used on Specimen 7, the stress concentration factor can be used as 1.4. Applying that factor onto the bending stress, the resulting stress on the gusset is

25,721.6 psi. The predicted failure value is right in line with the typical failure strength of GFRP (25,000-30,000 psi).

Specimens 6, 9, and 10 would follow the same process, but could have a higher failure stress than predicted. This is due to the fact that these three specimens had added FRP wraps, which would increase the failure strength.

4.6.5 Conclusions

Working with the experimental data and using standard analytical evaluation methods, the joints were behaving close to what would be expected. Some factors need to be studied more, i.e. stress concentration and stress distribution, to fully understand why the joint is failing a lower load value. A designer should always follow design specifications and manufacture properties, since they will always result in a lower failure load than the ultimate failure load.

CHAPTER 5 CONCLUSIONS AND FUTURE WORK

5.1 *Conclusions*

Literature review revealed that timber has properties that are ideal for structural purposes but is limited due to the high variability within the material and its susceptibility to damages that other materials are not encountered typically. The uncertainty in moment connections in timber buildings can lead to possible premature failures and thus must be retrofitted. One such solution is the patented system of fiber reinforced polymer (FRP) composite gussets and filler-modules.

The purpose of this project was to study the behavior of timber beam-column moment connections with the invention created at West Virginia University. This was done through several key objectives:

- Design and fabrication of 10 mortise and tenon connections.
- Static load testing of control specimens until failure.
- Adding filler-modules and FRP gussets to increase load capacity, ductility, and energy absorption.
- Improving the retrofitting scheme(s), and achieving a higher capacity. Thru wraps, dowels and filler-modules.
- Improving the scheme once again to increase the ultimate load capacity.
- Analysis and comparison of strength, ductility, and energy absorption.
- A detailed evaluation into failure modes for all test specimens.

The retrofit system used in this work was able to achieve a 450% increase in strength, 1,200% increase in ductility, and a 500% increase in energy absorption. The failure of retrofitted

joints was not catastrophic and would result in huge energy absorption under the proposed retrofit scheme(s).

5.1.1 Effect of Filler-Modules

Filler modules were able to increase strength, ductility, and energy absorption while maintaining joint rigidity at a higher load.

5.1.2 Effect of Filler-Module Shape

Two filler-module shapes were tested throughout this study, a curve and a wedge. There was no noticeable increase in any joint characteristics due to filler-module shape, because the filler-module failure was linked to debond along the glue line. If the glue line was able to resist higher forces, then there could be a failure in the filler-module.

5.1.3 Effect of Filler-Module Reinforcing Dowels

Two similar test specimens were tested with the only difference being the addition of filler-module reinforcing dowels. The two specimens, 9 and 10, peaked at almost identical loads, but behaved differently. Specimen 10 saw the tension side filler-module peel off at a lower load level than Specimen 9. The reduced contact area with the glue line and filler-module could be the reason for this occurrence. Therefore the dowels did not affect the ultimate load capacity, but did affect how the load was transferred.

5.1.4 Effect of Longitudinal Wraps

The longitudinal wrapped specimens (9 and 10) performed the best, and showed a later peel off of the tension side filler-module. The addition of longitudinal wraps is one solution to delay the peel off effect of the top filler-module.

5.1.5 Effect of Gusset Stiffness

Three gusset sets were tested in this study. The gussets with the highest stiffness performed the best, while the gusset with the lowest stiffness did not contribute to the behavior of the joint.

5.1.6 Analytical Work

After testing was complete the experimental data was compared to principal stress theory. The predicted values for failure stress were close to the material failure strengths, and provide great insight on how the joint is behaving with the proposed retrofit scheme(s).

5.2 Future Work

- Investigate more variables such as filler-module material, confinement effect, filler-module dowel effect, and filler-module bonding methods.
- Testing and evaluation of joints under dynamic loading to understand behavior for extreme events like earthquakes and hurricanes.
- Full scale structure tests to study the combined effects a building provides under static, dynamic, and blast loading.
- Develop design equations based off of timber design codes and composite material standards.

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APPENDIX A – TESTING DATA PLOTS

Presented in this appendix is the complete raw data used throughout this study. Each section is broken up by test specimen, and within the sections will be plots of load versus deflection, joint strain, module strain, beam pullout, and module pull off.

A.1 Specimen 1 Data

Figure A-1: Load vs Beam Pull Out (Specimen 1)

A.2 Specimen 2 Data

Figure A-2: Load vs Deflection (Specimen 2)

Figure A-3: Load vs Beam Pull Out (Specimen 2)

A.3 Specimen 3 Data

Figure A-4: Load vs Deflection (Specimen 3)

Figure A-5: Load vs Module Strain (Specimen 3)

Figure A-6: Load vs Module Pull Off (Specimen 3)

A.4 Specimen 4 Data

Figure A-7: Load vs Deflection (Specimen 4)

Figure A-8: Load vs Joint Strain (Specimen 4)

A.5 Specimen 5 Data

Figure A-9: Load vs Deflection (Specimen 5)

Figure A-10: Load vs Joint Strain (Specimen 5)

Figure A-11: Load vs Module Strain (Specimen 5)

A.6 Specimen 6 Data

Figure A-12: Load vs Deflection (Specimen 6)

Figure A-13: Load vs Joint Strain (Specimen 6)

Figure A-14: Load vs Module Strain (Specimen 6)

A.7 Specimen 7 Data

Figure A-15: Load vs Deflection (Specimen 7)

Figure A-16: Load vs Joint Strain (Specimen 7)

Figure A-17: Load vs Module Strain (Specimen 7)

A.8 Specimen 8 Data

Figure A-18: Load vs Deflection (Specimen 8)

Figure A-19: Load vs Joint Strain (Specimen 8)

A.9 Specimen 9 Data

Figure A- 20: Load vs Deflection (Specimen 9)

Figure A-21: Load vs Joint Strain (Specimen 9)

Figure A-22: Load vs Module Strain (Specimen 9)

A.10 Specimen 10 Data

Figure A-23: Load vs Deflection (Specimen 10)

Figure A-24: Load vs Joint Strain (Specimen 10)

Figure A-25: Load vs Module Strain (Specimen 10)