EXPERIMENTAL DETERMINATION OF THE STIFFNESS AND STRENGTH OF
CONTINUITY TIE CONNECTIONS IN LARGE WOOD ROOF DIAPHRAGMS,
AND IMPACT ON THE COLLECTIVE CHORD MODEL

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Caroline Nicole Yarber

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COMMITTEE MEMBERSHIP

TITLE: Experimental determination of the stiffness and strength of
continuity tie connections in large wood roof diaphragms, and
impact on the collective chord model

AUTHOR: Caroline Nicole Yarber

DATE SUBMITTED: August 2012

COMMITTEE CHAIR: John Lawson, P.E., S.E., M.S.

COMMITTEE MEMBER: Peter Laursen, P.E., Ph.D.

COMMITTEE MEMBER: Brent Nuttall, P.E., S.E., M.S.

ABSTRACT

Experimental determination of the stiffness and strength of continuity tie connections in large wood roof diaphragms, and impact on the collective chord model

Caroline Nicole Yarber

The goal of this thesis is to determine whether continuity ties in large wood diaphragms are stiff enough to engage and provide diaphragm flexural stiffness in a collective chord model. Four series of continuity tie assemblies using Simpson Strong-Tie steel connectors were tested to determine the stiffness of each assembly. The results found from testing were applied to an example building and then analyzed using both the traditional chord method and the collective chord method.

The completed analysis on a typical size warehouse building showed that the collective chord model will act inadvertently on an existing building designed with a traditional chord, or alternatively will potentially act intentionally in the design of a new building. The relative stiffness of the continuity ties will determine if they engage and allow them to act collectively. The testing and analysis completed creates a basis for further research into the actual static and dynamic behavior of these diaphragms. The collective chord model does seem to be a reasonable approximation for how diaphragms actually behave. If more research is conducted into different shaped and sized buildings to confirm that the collective chord model will work on most buildings then it will be a more accurate way to design new diaphragms and analyze existing diaphragms than the current traditional model.

KEYWORDS: Wood diaphragms, Collective chord model, Continuity ties, Flexible diaphragm stiffness

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LIST OF NOMENCLATURE

A	= Cross Sectional Area of Purlin (in ²)
b	= Diaphragm Width (ft)
E	= Modulus of Elasticity of Purlin (psi)
F ₀	= Force in Perimeter Continuity Tie (Kip)
F _P	= Wall- Roof Anchorage Force (Kip)
F _x	= Force in Continuity Tie X (Kip)
K	= Stiffness (K/in)
K _{Effective}	= Stiffness of a Continuity Tie Unit, Consisting of a Single Purlin with a Steel Connector at Each End (K/in)
K _{GLB}	= Stiffness of the Glulam Beam Calculated Based on the Glulam Beam Properties (K/in)
K _{Steel Connector}	= Stiffness of the Steel Connectors Calculated Experimentally (K/in)
L	= Diaphragm Length (ft)
M _{Diaph}	= Diaphragm Bending Moment (Ft- Kips)
P _{all}	= Tested Strength Based Allowable Load
P _s	= Strength Level Factored Load
R _{s(assembly)}	= Strength Reduction Factor for the Hold-Down Device Used in Assembly Testing
s	= Continuity Tie Spacing (ft)
v	= Diaphragm shear (plf)
w	= Diaphragm Uniform Load (lbs/ft)
x	= Index number correlating to F _x

1.0 INTRODUCTION

The purpose of this thesis is to experimentally determine the stiffness of the repetitive continuity tie system often found in wood diaphragms and determine what impact the continuity tie stiffness has on the overall flexural stiffness of the diaphragm. Diaphragms have both flexural and shear components of stiffness. The shear component is mainly resisted by the sheathing of the diaphragm while the chords are designed to carry axial force couples created by the moment, or the flexural component. Proper nailing of the sheathing to the framing members develops full resistance to the shear component. This thesis is focusing on the flexural component of diaphragm stiffness and what impact the repetitive continuity tie system has on the flexural stiffness of the diaphragm. The continuity tie system acting as a group of chords is not yet being taken into account. The traditional diaphragm model used in practice today assumes that the continuity ties do not participate in the flexural diaphragm load resistance. In the current model used today, the chords of the diaphragm are located at the perimeter boundaries, often embedded within the building's exterior walls. These chords are often composed of the rebar that is in the exterior walls and are the only chords considered to be acting in the diaphragm. If the collection of continuity tie steel connectors is stiff enough, it is possible for the traditional diaphragm analysis, using a single perimeter chord, to be altered and assume a linear force distribution across a collection of chords distributed through the diaphragm. All of the continuity ties would then be considered to participate as chords in the diaphragm. The goal of this thesis is to determine whether the collective chord model is stiff enough overall to engage and provide diaphragm flexural stiffness.

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2.0 BACKGROUND

Horizontal diaphragms can be constructed of many systems. Large, big box retail and warehouse building diaphragms are typically constructed of wood panelized roof systems or steel decking. Per ASCE 7-05 12.11.2.2.3, continuity ties are a code requirement for diaphragms supporting concrete or masonry walls in seismically active areas. This thesis is looking at concrete or masonry walled buildings with flexible panelized wood diaphragms.

2.1 The Collective Chord Model

Beginning about fifteen years ago, engineers and researchers began to question the traditional chord model used on diaphragms (Bender, Gerbredehin and Pollock 1996). Only the perimeter members of the diaphragm were being used in practice as chords in the diaphragm. The interior continuity ties were considered to only distribute wall anchorage loads into the diaphragm, not to behave as collective chords. Figure A on the next page shows the traditional diaphragm model perimeter-chord force distribution. The green line is the perimeter chord force in tension. The red line is the perimeter chord force in compression.

Figure A: Traditional Distribution of Chord Forces in a Diaphragm

For Figure A above:

w = Diaphragm Load (lbs/ft)

L = Diaphragm Length (ft)

b = Diaphragm Width (ft)

Thus the diaphragm chords in the traditional model have high forces because the exterior chords are the only members being used to resist diaphragm flexure.

The chord forces could become significantly large in very wide buildings, thus making it difficult to adequately design the chords. Because the chord forces could become so large, a few professionals in the structural engineering industry began to investigate an alternative diaphragm design model. The alternative chord design model, which considers the interior continuity ties as diaphragm chords, is called a collective chord model (Lawson 2007). The collective chord model uses multiple chords distributed across the diaphragm to distribute flexural forces throughout the diaphragm. Figure B on the next page shows the collective chord model force distribution. The green lines show the continuity ties acting as collective chords in tension. The red lines show the continuity ties acting as collective chords in compression.

Figure B: Collective Chord Force Distribution

For Figure B above:

w = Diaphragm Load (lbs/ft)

L = Diaphragm Length (ft)

b = Diaphragm Width (ft)

s = Continuity Tie Spacing (ft)

F_0 = Force in Extreme Continuity Tie (kips)

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In the collective chord model the continuity ties receive linearly decreasing load starting at the exterior of the diaphragm, moving towards the center. This model greatly reduces the forces in the exterior chords, thus making the forces more manageable to design for. With the reduced demand on the exterior chords, there will be an increased demand on the interior continuity ties, ranging from no chord force demand at the diaphragm's neutral axis to an increasing chord force demand towards the diaphragm's perimeter. The continuity ties are currently designed for wall anchorage force demands, and thus may already have sufficient capacity for the collective chord forces with no additional strengthening.

2.2 Continuity Ties in Masonry and Concrete Buildings

This thesis is exploring the idea that the continuity ties in masonry and concrete buildings indeed see a linear increase in force distribution from the bending neutral axis of the diaphragm. Part of this assumption has roots in the fact that the steel connection and purlin assembly provides a certain amount of in-plane flexural stiffness to the diaphragm. A purlin is a horizontal structural beam in a roof that provides intermediate support for common rafters. The relative axial stiffness of the assembly provides a continuous force transfer through the continuity tie, meaning the assembly could be considered a contributing chord. Wood diaphragm buildings are common to the highly seismic western United States. Continuity ties are a code requirement in high seismic areas, per ASCE 7-05, for wood diaphragms supporting concrete or masonry walls.

Most wood continuity ties require steel connections at every girder interruption to splice together members that span diaphragms more than a single member length.

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Continuity tie connections can use a variety of steel connectors and assemblies. Different stiffness values can be expected based on the steel connector and assembly combination. The wall anchorage force demand of the system determines the products and assemblies used in the continuity ties. Figure C below shows a typical double sided interior steel connector connection of a continuity tie in a wood diaphragm. When the steel connectors use screws they must be offset to avoid fastener conflict. When the steel connectors use bolts they may line up so the bolts go through both steel connectors on each side of the glulam.

Figure C: Chord to Chord Continuity Tie Connection Detail at Girder

Source: APA The Engineered Wood Association 2011

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Steel connectors can use nails, screws or bolts at continuity tie connections. This thesis looks at all three types of continuity tie connections. The first steel connector investigated was a single sided Simpson Strong-Tie HDU8-S2.5. Figure C above shows a double sided HDU connector configuration that is also possible. This steel connector uses screws to attach to the purlin. There are no oversized holes in the purlin for installing the screws but there are oversized pre-drilled holes in the HDU8-S2.5 steel bracket. The second steel connector investigated was a Simpson Strong-Tie's HD7B. Both a single sided and double sided configurations were investigated in the testing. The HD7B uses pre-drilled, 1/16" oversized holes with bolts to attach to the purlin. The oversized holes allow for easier construction with any minimal misalignment during construction, and the possible slip that occurs is taken into account when the anchor rod is installed. When the anchor rod is attached to the bracket there is a washer and nut attached at the end of the anchor rod. This washer and nut are finger tightened and then tightened half a turn with a wrench per the manufacturer's installation instructions. The tightening of the nut removes the slip of the bolts in the oversized holes. These configurations were chosen to compare the relative stiffness of the two setups. The last steel connector investigated was a Simpson Strong-Tie's MST 60 Strap. The MST 60 strap is attached with nails through the wood OSB sheathing and into the main member. The nail attachment has the potential for nail slip behavior, which is generally a nonlinear behavior. This study compared the relative stiffness of these four systems.

2.3 Verification

If the continuity tie assembly stiffness was found to be comparable to an existing traditional rebar chord stiffness, the assumption that there is a distribution of forces throughout the continuity ties in the diaphragm could be supported. For example, if the rebar stiffness of a traditional chord is found to be twenty times as stiff as the continuity tie assembly, there would be few scenarios where the continuity tie assembly could be considered as an effective contributing chord member. The relative stiffness of the traditional rebar chord would attract significantly more of the force than the continuity tie assembly. If the collective chord model's assumption that the continuity tie's stiffness is equal to or greater than the traditional continuous rebar chord were challenged, more testing would need to be performed to justify the new design model for diaphragms.

3.0 LITERATURE REVIEW

The following review of articles provides some insight into why this thesis is important and why it is necessary to help further development in the structural engineering field.

3.1 Loading of the Diaphragm

Bender, Gerbredehin and Pollack proposed two design modifications to the traditional all-wood diaphragm design method. The modifications are load sharing among roof purlins, or chords, and an assumed fixed condition at the ends of the roof (1996). Load sharing among roof chords is the main consideration in the collective chord model. If a linear load distribution is assumed throughout the depth of a diaphragm, and interior chords are assumed to assist the exterior chords in resisting an interior diaphragm moment created from an applied seismic force, an 82 percent reduction in chord force from the traditional method is calculated for the configuration and size of the example diaphragm used (Bender, Gerbredehin and Pollack 1996). Load sharing among purlins would allow interior chords to contribute to resisting diaphragm moments proportional to their distance from the neutral axis of the roof.

Load sharing among purlins is the main concept proposed by Lawson's new collective chord model (Lawson 2007). The collective chord model allows interior chords to contribute to resisting diaphragm moments. A simple example was used by Lawson to show the difference in external chord forces between the collective chord model and the traditional model. There is a 36 percent reduction in exterior chord forces when using the collective chord model on a small sample building. The collective chord model relies on

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the idea that the continuity ties linearly resist loads throughout the diaphragm. Neither Lawson nor Bender, et al. mentioned anything about how the connections' behavior impacts the design of the diaphragm. This thesis is breaking ground on how the connection design impacts diaphragm behavior and as such is important to the further development of diaphragms.

3.2 Diaphragm Connection Detailing

The new proposed collective chord model's main concept is that the interior chords contribute to resisting diaphragm moments. Because these interior chords are traditionally not designed for chord forces, their connections are only designed for wall anchorage forces per ASCE 7-05. The continuity tie connections become more important in the collective chord model and need to be properly modeled and analyzed so their design is appropriate for the forces the continuity ties might encounter.

According to Falk and Moody there are performance differences in code design criteria and the analytical models (1989). These differences limit efficiency and uniformity in the design of wood buildings. Horizontal and vertical diaphragms are the main mechanism for resisting lateral loads in light frame buildings. The shear capacity in a diaphragm can be limited by the lateral connections at diaphragm boundaries.

Traditional analysis methods assume diaphragms behave like a deep beam with the chords acting as the flanges and the sheathing as the web. Little work has been done to further develop the actual behavior of diaphragms if they do not truly behave like a deep beam model. Models by Foshci, GangaRao et al. and, Falk and Itani have explored the interaction between the fasteners and the framing members of the diaphragm. Their

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models have been verified through testing of isolated diaphragms and have shown that the lateral load response of the building is dependent not only on the walls and diaphragms but also on the inter component connections of the fasteners and framing members. This thesis is focusing on the inter component connections and their impact on the overall horizontal diaphragm. The inter component connections are important in the accuracy of analysis of the diaphragm. Falk and Moody's discussion of the need for accurate modeling of inter component connections emphasizes the need for research on the inter component, lateral load connections, which this thesis will be focusing on.

To familiarize owners, engineers, and craftsmen with the importance of lateral load connections for roof diaphragms, APA The Engineered Wood Association published a data file on proper lateral load connections for roof diaphragms (APA The Engineered Wood Association 2000). The data file was originally produced in response to the code changes introduced after the San Fernando earthquake of 1971 into the UBC 1973 and 1976 requiring continuous cross ties for wall anchorage that connect opposite diaphragm chords. These changes came after failures of some panelized roof diaphragms during the San Fernando Earthquake. Currently the lateral connection details must meet the code requirements to transfer wind or seismic lateral forces. This thesis will be looking at the behavior of the lateral load connections of these continuity ties.

It is important to properly design diaphragm connections both at the interior and exterior of the diaphragm. An example of improper diaphragm connection detailing occurred during the Landers Earthquake of 1992 (Brandow 1992). There was a failure of

a KMART retail diaphragm connection due to improper connection detailing. Refer to Figure D below for a visual of connection failure

Figure D: Failure of Diaphragm Connection of Retail Building
Source: Brandow 1992

The failure of the rigid wall, flexible diaphragm (RW/FD) building demonstrated the lack of understanding of the seismic performance of this class of building and the design requirements for the structural system. RW/FD buildings rely on the structural detailing to ensure the seismic forces can be transferred from the walls to the flexible diaphragm based on the location and stiffness of the walls. Poor detailing of the diaphragm to wall connection of the KMART building led to the diaphragm pulling away from the walls, causing several inches of deflections. The large deflection of the flexible diaphragm was incompatible with the rigidity of the masonry walls, causing four to six inches of separation between the two. Overall the building failures resulted from detail design deficiencies in the distribution of the horizontal shear through detailing of diaphragm chords and collectors to the diaphragm boundaries. Even though Brandow's case study

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focused on the diaphragm to wall connection, the case study verifies the importance of all detailing of diaphragm connections, including continuity tie connections. The proposed testing of this thesis will be investigating diaphragm detail connections.

The collective chord model assumes a linear force distribution of chord forces in the diaphragm, thus suggesting very small chord forces. This thesis will check the assumption that typical design loads are small enough to stay in the linear elastic range.

3.3 Proposed Code Equation Alterations for Diaphragm Behavior

Another phase in more accurately modeling diaphragm behavior is updating the current code equations. Some of the industry has already recognized the need to update the code equations and have already proposed some modifications.

Freeman, Searer, and Gilmartin proposed that a new approach to the design of rigid wall/flexible diaphragm (RW/FD) buildings should be investigated (2002). Current code provisions use temporary fixes to address problems that have arisen with major earthquakes. Some of the problems that have arisen are inadequate provisions to tie the building components together to act as a complete structure and the horizontal diaphragm dominates the building's seismic response. Current codes assume the seismic forces in buildings are based on the dynamic response of the vertical walls and frames, but RW/FD buildings' seismic forces are more a function of the dynamic response of the flexible horizontal diaphragm. The relatively rigid shear walls have very short periods, and thus the ground motion has little amplification as it travels from the ground to the top of the walls and into the diaphragm. The diaphragms have relatively large periods and thus their behavior defines the fundamental mode of vibration of the building. Thus the diaphragm

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stiffness will impact the building's seismic response more than the shear wall stiffness. The actual dynamic responses mentioned have begun to be addressed by the building codes with temporary fixes, but the temporary fixes recommended by the building codes do not properly address the dynamic response problems long term. Due to an irrational approach that discourages engineers' ability to exercise engineering judgment within the context of the building code, a new approach based on the dynamics of RW/FD buildings needs to be developed. This thesis aims to break ground on determining an accurate flexural stiffness model that can be utilized by researchers investigating RW/FD dynamics.

The SEAOC Seismology Committee also sees the need to investigate a new way to approach the seismic design of low rise RW/FD buildings. An alternative view to the current equivalent lateral force analysis in ASCE-7 and the 2006 IBC was offered by the SEAOC Seismology Committee in 2008 (SEAOC Seismology Committee 2008). A more appropriate approach computes the seismic response based on the diaphragm and wall out-of-plane periods. This approach calculates the chord forces in the diaphragm based on the period of the overall building seismic response rather than as a direct function of building height and the lateral force-resisting system stiffness of vertical structural elements. The SEAOC Seismology Committee believes the proposed alternative analysis method proposed by Freeman, Searer, and Gilmartin in 2002 provides a rational approach that computes the seismic response more accurately, based on diaphragm and wall out-of-plane periods (SEAOC Seismology Committee 2008).

On the scholarly level, proposal Project Director Lawson created an Extramural Funding Initiative (EFI) submission to California Polytechnic State University of San Luis Obispo in October of 2010 to receive funding for research of seismic stiffness into large wood diaphragms (Lawson 2010). The proposed research is to develop an accurate approach to predict the behavior of large wood building diaphragms undergoing large earthquake forces. The Structural Engineers Association of California Rigid Wall/Flexible Diaphragm (RW/FD) Task Group is studying the seismic behavior of RW/FD buildings. Lawson's EFI submission directly relates to the research desired by the RW/FD Task Group. In the EFI submission graduate student and author of the present thesis, Caroline Yarber, carried out most of the experimental work that is being researched for this thesis. This thesis will be focusing on the connections of the interior chords of diaphragms.

4.0 SCOPE

The purpose of this thesis is to experimentally determine the stiffness of the repetitive continuity tie system in a wood diaphragm and determine what impact the continuity ties have on the overall flexural stiffness of the diaphragm. Four different experiments of steel connector assemblies were performed. Each experiment had three trials to compare results.

All steel connector designations were associated with Simpson Strong-Tie products. Simpson Strong-Tie holds over 90% of the market share for the types of products used in continuity tie assemblies, thus it is appropriate to select their patented products for this thesis. The first test series, H1, was the single sided HDU8-S2.5 test assembly. The second test series, H2, was the single sided HD7B test assembly. The third test series, H3, was the double sided HD7B test assembly. The fourth test series, S1, was the MST60 strap test assembly. Figure E on the next page gives a summary of the steel connector assemblies.

Each test assembly was a tension test. The steel connectors were installed per the manufacturer's requirements and Acceptance Criteria 155 (AC155). AC155 was also used as a guide to conduct the tests. For example, the number of specimens per series, the rate of loading, and steel connector placement were based on AC155 requirements.

There were multiple items being measured. The capacity of the steel connector was measured. The relative displacement between the steel connector's anchor rod and the glulam was measured. Figure E on the next page shows the general test setup.

Figure E: Drawing of the general test setup for the Simpson Strong-Tie hold down series

More detailed CAD drawings of the individual test setups can be found in the appendix.

Figure F, found on the next page, shows the general setup for the LVDTs on a test specimen.

Figure F: Drawing of the general LVDT setup for the Simpson Strong-Tie hold down series

More detailed CAD drawings of the individual LVDT setups can be found in the appendix.

5.0 TESTING

Simpson Strong-Tie was contacted to determine what the most typical steel connectors were for seismic continuity ties and seismic retrofitting. The HD7B was identified as being a common connector specified for this purpose and was determined to be suitable as a test specimen. In addition, the HDU8-S2.5 connector and the MST 60 strap were also identified as having the potential for similar usage in the industry as a purlin continuity tie yet with potentially very different testing results due to their use of screws and nails. There were four different configurations with the three different steel connectors. The HDU8-S2.5 was a single sided assembly. The HD7B was both a single and double sided assembly. The MST 60 strap was a single sided assembly. Figure G below is a summary of the steel connectors used and their assembly for the specimen.

Series	Steel Connector Type Simpson Strong-Tie brand	Single or Double Sided Assembly
H1	HDU8-S2.5	Single
H2	HD7B	Single
H3	HD7B	Double
S1	MST 60 Strap	Single

Figure G: Summary of Steel connectors Used

These steel connectors were chosen because they are used fairly commonly in the building construction.

5.1 Material Acquisition

Over the summer of 2011, contacts were made with Simpson Strong-Tie and Beven Herron by John Lawson. Simpson Strong-Tie was interested in the research being conducted and offered to donate all steel connectors, fasteners, and anchor rods needed.

They also supplied a specially built bracket for the test setup and two steel load plates.

Beven Herron of La Mirada, California donated all glulam materials needed for testing.

5.2 Testing Issues

There were multiple testing issues that needed to be considered when the test specimens were being designed. The linear variable differential transformer location, the eccentricity due to the single-sided steel connector configuration and the strap configuration all needed to be worked out before testing occurred.

5.2.1 Linear Variable Differential Transformer (LVDT) Locations

The Linear Variable Differential Transformer (LVDT) locations were determined from the best location to acquire data for linear movement between the steel connector and the glulam as well as any deformation within the bracket and any stretch of the anchor rod. The overall system deformation was being captured, not the isolated deformation of individual components. Two different types of LVDTs were used in collecting data. One of the LVDTs is a more precise instrument and records data to the nearest thousandth of an inch. The other LVDT records data to the nearest hundredth of an inch but over a wider range of movement. If both sets of LVDTs recorded the same deformation and the specimen deformed more than could be recorded by the more precise LVDT the moderately accurate LVDT data was used to calculate the stiffness. The moderately precise LVDT was used in all graphs because the longer deformation range was needed and the difference between deformation of the more precise LVDT and moderately precise LVDT was minimal.

Each wide face of the glulam had a blue, more precise, and red, moderately precise, LVDT attached to it. The blue and red LVDTs were on opposite corners of each face of the glulam. The alternating LVDT placement was done to help capture any specimen rotation that occurred during testing. The data from the each set of LVDTs was averaged after the test was complete to account for any rotation that occurred during testing. The difference in deformation between each pair of LVDTs at opposing locations was minimal indicating little rotation during loading and thus averaging was often unnecessary.

The placement of the LVDTs was put at the lowest bolt or screw connection of the steel connector. If the LVDT's were place above the lowest bolt or screw they would not have captured the desired movement between the steel connector and glulam.

5.2.2 Eccentricity Due to Single Sided Configuration

The design of the test setup for the single sided configurations had to take into account the eccentricity from the offset anchor rod. Therefore a two part apparatus was designed to help counteract the forces caused by the eccentric anchor rod. One piece was located at the bottom of the glulam and the second piece was located at the top of the glulam creating a resisting force couple. Each piece was attached to the Riehle Universal Testing Machine so there would be a stiff enough connection to truly counteract the forces created by the anchor rod's eccentricity. Figure H on the next page displays the forces being applied and the reactions formed from the eccentricity apparatus.

The two brackets at the top of the glulam were anchored to the upper head of the Riehle universal loading machine. These brackets had bolt head that could be adjusted to

fit snugly against the glulam and prevent any rotation at the top of the configuration due to eccentric loading. Figures I and J found on the following pages show the upper brackets used to counteract any eccentricity caused by the single sided test setup.

Figure H: Forces and Reactions of the Eccentricity Test Setup

Figure I: Upper Piece of Eccentricity Apparatus Attached to Upper Head of Riehle Machine

Figure J: Side View of Upper Piece of Eccentricity Apparatus. Smoothed Bolt Heads Fit Snugly Against Glulam to Prevent Rotation.

The bracket at the bottom of the glulam fit down into the open jaw area of the lower head of the Riehle universal loading machine. There were two wheels attached to this bracket that rested against the side near the bottom of the glulam and prevented any rotation caused by the anchor rod's eccentricity. Figure K below and Figures L, and M, found on the following pages, show different views of the lower eccentricity apparatus used for the single sided test setup.

Figure K: Bottom Bracket of Eccentricity Apparatus for Single Sided Test Setup. Piece Is set in Open Jaw Area of Bottom Jaw of Riehle Machine.

Figure L: Side View of Wheel Setup of Bottom Bracket of Eccentricity Apparatus for Single Sided Test Setup

Figure M: Overhead View of Wheel Setup of Bottom Bracket of Eccentricity Apparatus for Single Sided Test Setup

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5.2.3 MST 60 Strap Configuration

The MST 60 strap configuration was also a single sided configuration. But this steel connector was different from the other steel connectors and needed a slightly different configuration from what had already been designed. In practice, on panelized wood roof diaphragms the MST 60 strap is generally installed on top of plywood or OSB sheathing after the plywood or OSB sheathing has been installed on top of the glulams. While the test specimens had the strap nailed over the sheathing, the setup that was tested was slightly different from the traditional construction setup. For the testing setup, the OSB was attached to the wide face of the glulam beam to minimize the experimental eccentricity as seen in Figure N below.

Figure N: OSB Attached to Wide Face of Glulam

The nail spacing was similar to traditional construction setups. The MST 60 strap was attached on top of the OSB as seen in Figure O on the next page.

Figure O: MST 60 Strap Attached to Specimen on Top of OSB

Thirty 10d nails were used to secure the MST 60 strap to the OSB and glulam. One more nail could have been added but was decided against to help prevent any splitting in the glulam.

5.2.4 Anchor Rod Pre-Loading

Per the Simpson Strong-Tie Catalogue and ASTM standards, the anchor nut had to be finger-tight plus half turn with a hand wrench. This requirement pre-loaded the anchor rod but the anchor rod relaxed after a couple of a minutes. Before starting the testing the load cell was zeroed out. The pre-loading data was recorded but not considered as part of the test.

5.3 Test Setup and Procedure

Once the testing issues were worked out, the four test series were completed per a specified testing procedure based on the Acceptance Criteria 155 (AC155). AC155 is a guideline for demonstrating compliance with performance features of the applicable

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codes referenced in the acceptance criteria. It should be noted that each test completed only comprises half of an actual continuity tie connection assembly, from girder center line to girder center line. Installed continuity ties have two steel connectors per anchor rod, a steel connector on each end of the anchor rod with the system centered on the glulam girder.

5.3.1 H1 Series, Single Sided HDU8-S2.5

The H1 series was a single sided HDU8-S2.5. There were three tests completed with the HDU8-S2.5 setup. Figure P shown below shows the technical layout of H1 LVDT setup. Refer to the appendix for more detailed CAD drawings of the full test setup.

Figure P: Drawings of HDU8-S2.5 Instrumentation Placement

Figure Q on the next page shows the physical test setup of the front side of the glulam.

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Figure Q: Test Specimen H1.2 Front Side LVDT Locations

Figure R below shows the physical test setup of the back side of the glulam.

Figure R: Test Specimen H1.2 Back Side LVDT Location Setup

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Refer to the appendix for a detailed description of the testing procedure for the H1 series.

5.3.2 H2 Series, Single Sided HD7B

The H2 series was a single sided HD7B. There were three tests completed with the HD7B setup. Figure S below shows the technical layout of the H1 LVDT setup. Refer to the appendix for more detailed drawings of the full test setup.

Figure S: LVDT Setup for the HD7B Single Sided Configuration

Figure T below shows the physical test setup of the front side of the glulam.

Figure T: Test Specimen H2.1 Front Side LVDT Location Setup

Figure U below shows the physical test setup of the back side of the glulam.

Figure U: Test Specimen H2.1 Back Side LVDT Location Setup

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Refer to the appendix for a detailed description of the testing procedure for the H2 series.

5.3.3 H3 Series, Double Sided HD7B

The H3 series was a double sided HD7B. There were three tests completed with the HD7B setup. Figure V below shows the technical layout of H3 LVDT setup. Refer to the appendix for more detailed CAD drawings of the full test setup.

Figure V: LVDT Location for the HD7B Double Sided Setup

Figure W on the following page shows the physical test setup for the front side of the glulam.

Figure W: Test Specimen H3.3 Front Side LVDT Location Setup

Figure below shows the physical test setup for the back side of the glulam.

Figure X: Test Specimen H3.3 Back Side LVDT Location Setup

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Refer to the appendix for a detailed description of the testing procedure for the H3 series.

5.3.4 S1 Series, Single Sided MST 60 Strap

The S1 series was a single sided MST 60 strap. There were three tests completed with the MST 60 strap setup. Figure Y below shows the technical layout of S1 LVDT setup. Refer to the appendix for more detailed CAD drawings of the full test setup.

Figure Y: LVDT Locations for MST60 Strap Setup

Figure Z on the following page shows the physical test setup for the front side of the glulam.

Figure Z: Physical Test Setup of LVDTs on the Front Side of the Glulam for S1 Series

Figures AA below and BB on the following page show close up pictures of the LVDT setup for the MST 60 strap.

Figure AA: Close up of Upper LVDT for S1 Series

Figure BB: Close up of Lower LVDT for S1 Series

The back side of the glulam did not have any sensors located on it. The only item located on the back of the glulam was the eccentricity apparatus, as shown below in Figure CC.

Figure CC: Backside of S1 Series with Eccentricity Apparatus

Refer to the appendix for a detailed description of the testing procedure for the S1 series.

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6.0 RESULTS

For each test there was data from two highly sensitive LVDTs, two moderately sensitive LVDTs, and a load cell. All data acquisition devices were calibrated before the test. All data from the load cell and LVDTs were zeroed out before the loading started from the Riehle machine. The data from the two highly sensitive LVDTs were averaged to take into account any rotation that occurred during the testing. The same procedure was repeated for the moderately sensitive LVDTs. When all of the data analysis was completed, the data was graphed on a force vs. deflection graph.

This procedure was completed for each test. Once all tests for a series were completed, the three test graphs were then combined onto one graph to show the comparison of the tests for the series. The final graph, Figure RR, showed a comparison of each test series to all other test series.

6.1 Individual Series Results

For each series the stiffness of the configuration was found based on the collected data. The stiffness was calculated by reviewing the data from start to maximum tested strength based allowable load capacity per the AC155 specifications. The strength based design load and the allowable load lines shown on the graphs for each series were calculated with the tested data per AC155 without the deformational limit criteria. This portion of the data was generally linear and thus could easily provide a uniform stiffness for the configuration tested.

6.1.1 H1 Series

The H1 tests all yielded a similar two stage failure. The first failure occurred when a split formed down one side of the wood member adjacent to the steel connector.

Figure DD below, displays the split of the wood

Figure DD: Split Forming in the Wood Next to the Side of the HDU8-S2.5 During Testing

After that failure occurred, the configuration continued to take load for about two to three minutes longer until the next failure mode occurred, a shear plug failure. The entire steel connector assembly was grabbing the glulam which is what allowed the configuration to continue taking load after the split formed down the line of screws. Figure EE on the following page displays the steel connector gripping the wood.

Figure EE: Overhead View of HDU8-S2.5 Gripping Wood During Test

Figure FF below shows the shear plug failure.

Figure FF: Typical Shear Plug Failure for H1 Series

All three tests of the series failed first with a split then with a shear plug failure.

Figures GG and HH on the following pages are the graphs of the results from the HDU8-S2.5 tests. Force versus displacement was graphed because those were the data items of

interest that could be reasonably compared to current data. Figure GG below displays the force vs. displacement curves for the maximum capacity of each H1 series test. The graph also has a black line representing the average of all the three HDU8 tests. The tests were relatively consistent; therefore, the average force vs. deflection curve fell in the middle of the three test curves. The tested strength based allowable load and the strength level factored resistance per AC155 is also included on Figure GG below.

**Figure GG: Force vs. Displacement Curves for the HDU8-S2.5 Test Series.
This Graph Shows All Data Points.**

All three test results from the HUD8-S2.5 series test yielded about the same steel connector assembly stiffness. The maximum capacity of the steel connector assembly did vary by about 22% from the lowest to highest capacity. Maximum displacement values varied by about 23% from the smallest to largest displacement.

6.1.2 H2 Series

All three tests from the H2 series failed when a split formed along the bolt line of the configuration. Figure HH below displays the split in the glulam.

Figure HH: Back Side View of Typical Split in H2 Series on the Left Photo, Front Side View of Typical Split in H2 Series on the Right Photo

As the specimen was being loaded, the bolts would rotate. The rotating bolts and their washers would begin to grab the wood. Figure II on the following page shows the bolts and washers grabbing the glulam.

Figure II: Backside View of H2 Series Showing the Bolts and Washers Grabbing the Wood After Rotating While Load Was Applied

This grabbing caused some form of an artificial ductility in the configuration. The specimens were able to continue holding load after the split was fully formed in the glulam.

Figure JJ on the following page is a force vs. displacement curve for the maximum capacity data points of the three HD7B tests. The graph displays the consistency of the test in terms of slope, or stiffness, of the graphs. The black line is the average value of the maximum capacity of the tests. The black line almost covers the other tests entirely and is thus an extremely good summary of the test results. The tested strength based allowable load and the strength level factored resistance per AC155 is also included on Figure JJ on the following page. The maximum capacity of the steel connector was the only major difference between tests. There was about a 19% difference in capacity between the three tests and about a 40% difference in deflection.

Figure JJ: Force vs. Displacement Curve for Maximum Capacity Values for the H2 Series

6.1.3 H3 Series

The entire H3 series test failed in a shear plug failure. There was minimal ductility in the system. The failure was much more of a brittle failure than any of the other hold downs. Figures KK and LL on the next page show the shear plug failure from different angles.

Figure KK: Close Up View of Typical Shear Plug Failure for the H3 Series

Figure LL: Overall View of Typical Shear Plug Failure for H3 Series

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Figure MM below is the force vs. displacement graph for the maximum capacity values in the HD7B double sided test series. The black line is the average force vs. deflection curve for all three tests. Because the black line falls in between all three tests, it is a good summary of the test results. The tested strength based allowable load and the strength level factored resistance per AC155 is also included on Figure MM below. The three tests yielded fairly consistent results. The second test had a lower overall stiffness than the other two tests, but the specimen was still able to reach about the same maximum load as the other two tests. There was about a 17% difference in maximum capacity of the test and about 42% difference in the deflection between tests.

Figure MM: Force vs. Deflection Curve for Maximum Capacity Values of the H3 Series

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6.1.4 S1 Series

There were two items being measured for the S1 series. The displacement between the MST 60 strap and the glulam was recorded as well as the displacement between the OSB and the glulam. All three tests failed in a net tension through one of the holes in the strap designed for alternative bolting or sales display. The net tension failure is shown in Figure NN below and OO on the following page.

Figure NN: Net Tension Failure of MST 60 Strap in Relation to End of the Glulam Specimen

Figure OO: Close Up View MST 60 Strap Typical Net Tension Failure of S1 Series

Figure PP on the next page shows the force vs. displacement curve for the maximum capacity values of the MST 60 strap tests by computing the difference between the strap and the glulam deflection. Similarly, Figure QQ shows the force vs. displacement curve associated with the difference between the OSB and the glulam deflection. The failure sequence was exactly the same for all three tests. The graphs display the consistent behavior well. The black line is the average force vs. displacement curve. In Figure PP, it falls directly in the middle of all three tests and almost exactly on top of one of the tests. There was less than a 3% difference between maximum capacities of the strap for all tests. There was about a 28% difference in deflection between the strap and the glulam for all the tests.

Figure PP: Force vs. Deflection Curve for the Maximum Capacity Values of the S1 Series When Looking at the Deflection Between the Strap and the Glulam

The slip between the OSB and glulam did not contribute to the total assembly deformation until about 300 pounds of load. Once the OSB attachment began to contribute to the assembly deformation it contributed between 30% to 40% of the overall deformation, while the MST 60 strap and its attachment contributed the remaining 60% to 70% of total deformation.

Figure QQ below shows the maximum displacement between the OSB and the glulam. The black line is the average force vs. deflection curve for the deflection between the OSB and glulam.

Figure QQ: Force vs. Deflection Curve for Maximum Displacement of the S1 Series When Looking at the Deflection Between the OSB and Glulam

6.2 Series Comparison Results

Figure RR on the following page displays all four test series on the same force vs. displacement graph. This force vs. deflection curve only displays the average value curve for each test series. The average value curves are the average of the data of three tests in a series until the earliest failure occurred in one of the tests. Once a failure occurred the averaging of data was stopped because averaging of the two continuing tests would yield skewed results.

Figure RR: Force vs. Deflection Curves Comparison for all of the Series

The force vs. displacement curves for the S1 series were not nearly as linear as the other steel connector force vs. displacement curves. The S1 series, or the MST 60 strap, had a significant amount of nail slip during the test. The nail slip was to be expected because the strap was secured with nails. The straps had a significantly lower capacity than the other steel connectors as expected based on the capacity ratings. The straps' stiffness was also higher than the H1 and H2 series, but the force vs. displacement curve was approximately cubic vs. linear. The H3 series was the stiffest configuration tested, which is logical due to the double steel connector and rod configuration. The assembly

was stiffer and less ductile than the other configurations. The H1 and H2 series had relatively close stiffnesses to each other, although the H1 series had a higher capacity than the H2 series. The expected allowable strength was calculated with a multi-step process from AC155. The tested strength based allowable load was calculated first per AC155 using the equation below.

$$P_{all} = \frac{\text{Lowest Maximum Test Value} * R_{s(assembly)}}{3}$$

Where:

P_{all} = Tested Strength Based Allowable Load

$R_{s(assembly)}$ = Strength Reduction Factor for the Hold-Down Device Used in
Assembly Testing

A $R_{s(assembly)} = 1$ was used. The tested strength based allowable load was then multiplied by 1.4 to bring it up to the strength level factored load using the equation below.

$$P_s = 1.4P_{all}$$

Where:

P_s = Strength Level Factored Load

AC155 also has a tested displacement criterion to calculate the allowable load and strength based design load. Because this thesis is looking at the capacity of the steel connector assembly the displacement based criterion was not used. Figure SS on the following page is a summary table of the results from all of the testing.

Connector Type	HDU8-S2.5			HD7B Single Sided		
How Many Test Units	3			3		
Allowable Design Load (lbs)	Simpson Strong-Tie	AC155		Simpson Strong-Tie	AC155	
	6223	7697		6558	4797	
Observed Failure Mode	Shear Plug			Wood Split		
Tested Strength (lbs)	H1.1	H1.2	H1.3	H2.1	H2.2	H2.3
	23090	28182	28713	14390	16330	19004
Displacement at Failure (Inches)	0.389	0.634	0.572	0.223	0.281	0.363

Connector Type	HD7B Double Sided			MST60 Strap		
How Many Test Units	3			3		
Allowable Design Load (lbs)	Simpson Strong-Tie	AC155		Simpson Strong-Tie	AC155	
	13116	9697		4830 (assuming 16d common nails)	4582	
Observed Failure Mode	Shear Plug			Net Tension		
Tested Strength (lbs)	H3.1	H3.2	H3.3	S1.1	S1.2	S1.3
	29090	35185	36005	13747	13828	14096
Displacement at Failure (Inches)	0.135	0.189	0.183	0.124	0.175	0.181

Figure SS: Summary Table of the Results From All of the Testing

There are two allowable design loads given in Figure SS above. One allowable design load is taken from the Simpson Strong-Tie product catalog based on independent testing in conformance with AC155. Because my testing used 3 1/8" thick members, the Simpson Strong-Tie allowable design load values in Figure SS above were found from interpolation. Also the completed testing for the MST60 strap used 10d screw shank nails

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in which there is no allowable design load specifically given in the Simpson Strong-Tie product catalog. So the allowable design load for 10d common nails was used as a comparison for testing. The allowable values from the Simpson Strong-Tie catalog have already been increased for wind and earthquake with a possible load duration factor if applicable. The other design load is calculated from the test data per AC155 without the deformational limitation invoked.

After all of the data from the four test series was compiled on the same force vs. displacement curve, it was important to determine a line of best fit for the data so the stiffness of each system could be compared. If the maximum capacity curves were used, the line of best fit would either be linear or a quadratic. However, when looking at only the portion of data that would be considered allowable per the Simpson Strong-Tie the data was linear for three of the four series as can be seen in Figure TT below.

Figure TT: Series Comparison of Force vs. Deflection for Allowable Capacity per Simpson Strong-Tie

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The S1 series was not used in the comparison because of the non-linear properties of the assembly. The resulting nonlinear stiffness of the continuity ties assembly results in a nonlinear force distribution of the collective chord forces. This thesis did not evaluate the non-linear properties associated with nail slip and its impact on the collective chord model. The R^2 value, or coefficient of determination, provides a measure of how well future outcomes are likely to be predicted by the line of best fit used. The R^2 value ranges from 0 to 1. The closer the R^2 value is to 1 the better the line of best fit can predict future outcomes.

As shown in Figure TT on the previous page, the slope from the line of best fit yielded the stiffness values for each steel connector assembly. A summary of the stiffness values from Figure TT on the previous page is shown in Figure UU below. The stiffness values are the slope value from the line of best fit for each series. The single sided steel connectors had comparable stiffnesses to each other while the double sided steel connector was almost seven times as stiff as the single sided steel connectors.

Hold down Type	Stiffness Value, K (K/in)
HDU8- S2.5	74.0
HD7B Single Sided	70.0
HD7B Double Sided	470.0

Figure UU: Summary Table of Stiffness Values for Steel Connectors

6.3 Possible Conclusions Being Investigated

While the testing information is valuable, it needs to be put into context with current information. Two different investigations were done to compare chord forces and

stiffness values of traditional diaphragm models to the new proposed collective chord model. The two investigations were designed around on two possible conclusions.

The first conclusion being investigated is whether the connections' stiffness behavior is linear which would suggest that simple principle of mechanics can be used to compute collective chord behavior, without the need to rely upon experimentally determined stiffness values. The second conclusion being investigated is whether the axial stiffness of all of the continuity tie chords collectively create enough flexural diaphragm stiffness that an existing traditional chord participation is minimized. This conclusion considers the experimentally determined stiffness values in the analysis.

The support of the first conclusion came from the following train of thought. There are two independent portions of deformation in a diaphragm, shear deformation and flexural deformation. From basic diaphragm behavior it is known that diaphragm chords are assumed to resist only flexural deformation and resulting flexural stresses. Traditionally there are only diaphragm chords on the perimeter of the building. With the collective chord model the continuity ties, already required per code in the diaphragm, can act as multiple chords, collectively resisting the flexural stresses. If the axial stiffnesses of the continuity tie chords are linear with an increase in loading, then the force in each continuity tie chord can be computed using principles of mechanics regardless of what the actual stiffness is.

The experimental tests completed show the connection stiffnesses are indeed within the linear range of "expected" loading and thus the entire collective chord is a

linear response. The connections' linear stiffness behavior suggests simple principle of mechanics can be used to compute collective chord forces as outlined by Lawson (2007). Because continuity tie stiffness response is found to be linear, the developed equations for calculating collective chord forces are valid for large wood roof systems such as those looked at in this thesis. This conclusion does not rely on the actual stiffness values experimentally determined and is demonstrated in Investigation 1: Traditional Method, in section 6.4.1 on the following pages.

The next conclusion investigated is whether the axial stiffness of all the continuity tie chords collectively creates enough flexural diaphragm stiffness that an existing traditional chord participation is minimized. This concept is investigated in Investigation II: Collective Chord Method, in Section 6.4.2 on the following pages. The experimentally determined stiffness values were used in this investigation to determine the collective flexural diaphragm stiffness of the continuity tie chords, which gave evidence in the example building, had of enough flexural stiffness to make the traditional wall chord insignificant or marginalized.

6.4 Comparative Investigations

A typical building size and system is provided to investigate potential conclusions. The building is 480' x 480' with 48' bays and purlins at 8' on center. The concrete walls are 8" thick and 28' tall with a 2' tall parapet. Figure VV on the following page is the basic framing plan for the investigations. The first investigation completed was using the traditional design method for diaphragms. The second investigation completed was using the collective chord model design for diaphragms. The seismic and

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site data used represented a fairly high seismic region on the western coast of the United States. The information given on the following page is the other given information used.

Figure VV: Basic Framing Plan for Case Study Building

Given Information**Roof:**

Dead Load = 15 PSF
Live Load = 20 PSF (reducible)

Walls:

Thickness = 8 inches
Height = 30 Feet
Normal Weight Concrete = 150 PCF
f'c = 4000 PSI
A615, Grade 60 Rebar, F_y = 60 KSI

Roof Sheathing:

15/32" Structural - I Sheathing

Roof Structure:

Glulam purlins and girders. All wood is Douglas-fir

Seismic Force-Resisting system:

Bearing wall system consisting of intermediate precast shear walls, using a R of 4

Seismic and Site Data:

Mapped Spectral accelerations for the site

S_s = 1.5 g (Short Period)
S₁ = 0.6 g (1-Second Period)
Occupancy Category = II
Site Class = D

Wind

Assumed to not govern

The calculations on the following pages are how the diaphragm forces were generated.

Design Base Shear Coefficient

***Design Spectral Response Acceleration S_{DS} and S_{D1}**

For $S_S = 1.5g$ and $S_1 = 0.6g$ and Site Class D

$$F_a = 1.0$$

$$F_v = 1.5$$

$$S_{MS} = F_a S_S = 1.0 (1.5) = 1.5 \quad (\text{Short Period})$$

$$S_{M1} = F_v S_1 = 1.5 (0.6) = 0.9 \quad (\text{1-Second Period})$$

$$S_{DS} = 2/3 S_{MS} = 2/3 (1.5) = 1.0 \quad (\text{Short Period})$$

$$S_{D1} = 2/3 S_{M1} = 2/3 (0.9) = 0.60 \quad (\text{1-Second Period})$$

For $S_{DS} = 1.0$ and Occupancy II, $SDC = D$

For $S_{D1} = 0.60$ and Occupancy II, $SDC = D$

Using ASCE/SEI 7-05 Section 12.6 and Table 12.6-1, the Equivalent Lateral Force Procedure of Sections 12.8 can be used

$$T_a = C_t h_n^x$$

$$C_t = 0.02, x = 0.75, h_n = 28'$$

$$T_a = 0.02 (28)^{0.75} = 0.24 \text{ s}$$

Per Section 12.8.1.3 $S_s = 1.5$

***Base Shear Using the Equivalent Lateral- Force Procedure**

$$V = C_S W \quad C_S = \frac{S_{DS}}{(R/I)}$$

$R = 4$ For intermediate precast concrete shear walls

$I = 1.0$ For occupancy Category II

$$\text{Therefore } C_S = \frac{1}{(4/1)} = 0.25$$

Check max limit for C_s where $T \leq T_L$

$$C_{S\text{MAX}} = \frac{S_{D1}}{T (R/I)} = \frac{0.6}{0.24 (4/1)} = 0.63 > 0.25 \quad \text{OK}$$

Check min C_s

$$C_{S\text{MIN}} = 0.01 < 0.25 \quad \text{OK}$$

$$C_{S\text{MIN}} = \frac{0.5S_1}{(R/I)} = \frac{0.5 (0.6)}{(4/1)} = 0.08 < 0.25 \quad \text{OK}$$

$C_{S\text{MIN}} < 0.25 < C_{S\text{MAX}}$ Therefore OK and $C_S = 0.25$ Governs

$$V = C_S W = 0.25W$$

*Roof Diaphragm Shears

East-West Direction

w is computed using the diaphragm length and wall heights

$$\text{Roof Dead Load} = 15 \text{ PSF}$$

$$\text{Wall Dead Load} = \frac{8.00'' (150 \text{ PCF})}{12''/1'} = 100 \text{ PSF}$$

$$\text{Roof Height} = 28 \text{ Feet Average}$$

$$\text{Parapet Height} = 2 \text{ Feet Average}$$

$$w = 0.25(15 \text{ PSF})(480 \text{ FT}) + [0.25(100 \text{ PSF})(30 \text{ FT})(30/2)(1/28)]2 =$$

$$w = 2604 \text{ plf}$$

$$\frac{\text{Diaphragm Shear at Line A}}{480 \text{ FT}} = \frac{624960 \text{ \#}}{480 \text{ FT}} = 1302 \text{ \#/FT}$$

Because this building is doubly symmetric the roof diaphragm shears will be the same in the North-South direction

6.4.1 Investigation 1: Traditional Method

The first investigation uses the given information mentioned above to find the perimeter chord force, area of steel used for the perimeter chord, and the wall-roof anchorage force transmitted into the continuity ties. The stiffness of the steel chord was calculated using the equation $K = \frac{AE}{L}$.

Figure WW below is a summary of the findings from the traditional chord method.

	LRFD	ASD
Perimeter Chord Force (Kip)	156.2	109.4
Area of Steel Required (in²)	3.00	-
Continuity Tie Force (Kip)	10.29	-
Stiffness of Perimeter Chord (Kip/in)	15.1	-

Figure WW: Summary of Findings from the Traditional Chord Method

The traditional method considers only the continuous steel in the perimeter walls of the building to act as a chord for the building. Because only the continuous steel is considered, the chord force is very large, as shown in Figure WW above. The large chord force thus requires a large amount of chord steel to handle the load. The calculated wall-

roof anchorage force is much smaller than the perimeter chord force and is smaller than the allowable capacity found for the HD7B double sided system during testing. The allowable capacity for the HD7B double sided system was 11.73 Kip, which is greater than the 10.29 Kip wall-roof anchorage force and thus was used for the steel connector for the wall anchorage continuity ties.

While only the value of the stiffness for the perimeter chord is shown in Figure WW above, the perimeter chord stiffness value is compared to the stiffness value for a single line of purlins and their continuity tie connections. The stiffness value for the single line of purlins with their connections is $K_{\text{HD7B Double Sided Line}} = 6.1 \text{ K/in}$. The stiffness value for a single line of purlins with their connections was found in two steps. The first step was to find the stiffness of an individual system consisting of a purlin with a double sided HD7B installed on each end of the purlin as shown in Figure XX below. The purlin, a 3" X 27" DF 24F-V4, was designed for the loading stated in the given information on the previous pages. The equation shown in Figure XX below was used to calculate the system stiffness, $K_{\text{HD7B Double System}} = 79.7 \text{ K/in}$

$$K_{\text{Effective}} = \frac{1}{2 \left(\frac{1}{K_{\text{SteelConnector}}} \right) + \frac{1}{K_{\text{GLB}}}}$$

Figure XX: Equation Used to Find the System Stiffness for a Glulam Beam with a Double Sided HD7B Installed on Both Sides

For Figure XX on the previous page:

$K_{\text{Effective}}$ = Stiffness of a Continuity Tie Unit, Consisting of a Single Purlin With a Steel Connector at Each End (K/in)

K_{GLB} = Stiffness of the Glulam Beam Calculated Based on the Glulam Beam Properties (K/in)

$K_{\text{Steel Connector}}$ = Stiffness of the Steel Connectors Calculated After Testing (K/in)

The second step was to use the system stiffness found for a single purlin length to compute the stiffness for a single line of continuity ties of the same length as the steel perimeter chord. The same principle used to find the system stiffness was used to find the full-length single line stiffness. There were ten of the glulam beams with a double sided HD7B installed on each end in the single line of continuity ties and thus the stiffness of that line was $K_{\text{HD7B Double Sided Line}} = 6.1 \text{ K/in}$. The traditional steel chord stiffness, $K_{\text{Steel Chord}} = 15.1 \text{ K/in}$, is nearly double that of the single line of continuity ties.

But looking at the overall diaphragm shows the addition of all the continuity tie line stiffnesses far exceeds that of the traditional rebar chord. A small portion of the forces found in the continuity ties will be pulled toward the traditional rebar chord because it is stiffer than a single line of continuity ties, but overall the continuity ties will inadvertently act as collective chords. Therefore the continuity ties will engage based on the relative stiffness of the traditional steel chord compared to the collective continuity tie chord.

The stiffness of all of the cross tie lines (continuity ties) was found using a multi-step process described in detail in section 6.3.2. The stiffnesses found for all of the cross tie lines added together was $K_{\text{Diaphragm with all Cross Ties}} = 92.9 \text{ K/in.}$ Figure YY below is a summary of all the stiffness values found for the first case study.

	K (Kip/in)
K Steel Chord	15.1
K HD7B Double Sided Line	6.1
K Diaphragm with all Cross Ties	92.9

Figure YY: Summary of Stiffnesses Found For Investigation 1

Calculations for stiffness were focused on the locations where the diaphragm is in tension. In an ideal situation, when the diaphragm is in compression the diaphragm should have a higher stiffness than is assumed from the stiffness computed from the tension tests. This assumption is based on the expectation that wood bearing on wood during compression forces is stiffer than the observed tensile deformation. There is a more direct, full connection between members in an ideal compression situation which would make the diaphragm stiffness higher. In a worst case scenario of diaphragm compression, there are gaps between members from erection tolerances. In this worst case scenario the gaps would be large enough for the diaphragm to actually have a lower stiffness in compression than when in tension. In reality when the diaphragm is constructed the members are somewhere in between the ideal scenario and the worst case scenario. Because of this the diaphragm stiffness will most likely be somewhere in the middle of ideal and worst case, and thus about the same as the tension scenario. Another item considered when the diaphragm is in compression is if the compression side of the

diaphragm is more stiff than the tension side, then the neutral axis of the diaphragm moves towards the compression zone. When this occurs the tension forces in the diaphragm will decrease because there is a larger moment arm due to the neutral axis moving towards the compression zone. Therefore the tension forces become even lower than they already appear when using the collective chord model. Thus the concepts concluded from the collective chord model are still validated. For these two reasons this thesis did not investigate in more detail what occurs to the diaphragm when it is in compression.

6.4.2 Investigation 2: Collective Chord Method

The second investigation used the given information stated previously to find the chord forces of the perimeter chord and the interior chords, area of steel used for the perimeter chord, and the wall-roof anchorage force.

The perimeter chord force was found using the following equation from Lawson's paper, "Thinking Outside the Box: New approaches to very large flexible diaphragms" (Lawson, 2007).

$$F_0 = \frac{6sM_{Diaph}}{(b + s)(b + 2s)}$$

The interior chords were then found using the following equation from Lawson's paper.

$$F_x = F_0 \left(1 - \frac{2sx}{b} \right)$$

Where:

b = Diaphragm Width (ft)

F₀ = Force in Perimeter Continuity Tie (Kip)

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F_x = Force in Continuity Tie X (Kip)

s = Continuity Tie Spacing (ft)

x = Index number correlating to F_x

Figure ZZ below, graphically illustrates the distribution and labeling of collective chord forces calculated using the above equations. Lawson's paper assumes a linear force distribution for the collective chord model.

Figure ZZ: Graphical Illustration of the Distribution and Labeling of Collective Chord Forces

After all of the chord forces were found using the collective chord model, they were compared to the chord force found using the traditional method.

The bending stiffness of the diaphragm for the collective chord model was calculated using a multiple step process. The first step was finding the chord forces for

the collective chord model as stated above. The next step was to find the bending deformation of the diaphragm. For the collective chord method the moment of inertia of the diaphragm was calculated using the parallel axis theorem. Once the moment of inertia expression was found, it was then plugged into the bending deformation equation. Lawson went through the derivation for the collective chord moment of inertia and collective chord bending deformation in his paper, “Thinking Outside the Box: New approaches to very large flexible diaphragms” (2007). The bending deformation equation is as follows:

$$\Delta_{Bending} = \frac{15vL^3s}{4EA(b + 2s)(b + s)}$$

Where:

- A = Cross Sectional Area of Purlin (in²)
- b = Diaphragm Width (ft)
- E = Modulus of Elasticity of Purlin (psi)
- L = Diaphragm Length (ft)
- s = Continuity Tie Spacing (ft)
- v = Diaphragm shear (plf)

Once the bending deformation was found the stiffness for each chord line was found by

using the equation, $K_{Bending} = \sum \frac{0.7F_x}{\Delta_{Bending}}$.

There was about an 84% reduction in the perimeter chord force when using the collective chord method. Hence the area of steel needed in the perimeter chord was also

about 84% less than when the traditional method was used. The wall-roof anchorage force, $F_p = 10.29 K$, is the same regardless of which diaphragm chord method is being used. Figure AAA on the following page shows the chord forces found.

Chord Forces							ASD Continuity Tie (Kip)
	Traditional			Collective Chord			
	Stiffness	LRFD	ASD	Stiffness	LRFD	ASD	
Chord Line	K (K/in)	F (Kip)	0.7F (Kip)	K (K/in)	F _x (Kip)	0.7F _x (Kip)	
Perimeter Chord	15.1	156.2	109.4	1.6	14.87	10.41	10.29
1	0	0	0	6.1	14.37	10.06	10.29
2	0	0	0	5.9	13.88	9.71	10.29
3	0	0	0	5.7	13.38	9.37	10.29
4	0	0	0	5.5	12.89	9.02	10.29
5	0	0	0	5.3	12.39	8.67	10.29
6	0	0	0	5.0	11.90	8.33	10.29
7	0	0	0	4.8	11.40	7.98	10.29
8	0	0	0	4.6	10.90	7.63	10.29
9	0	0	0	4.4	10.41	7.29	10.29
10	0	0	0	4.2	9.91	6.94	10.29
11	0	0	0	4.0	9.42	6.59	10.29
12	0	0	0	3.8	8.92	6.25	10.29
13	0	0	0	3.6	8.43	5.90	10.29
14	0	0	0	3.4	7.93	5.55	10.29
15	0	0	0	3.2	7.43	5.20	10.29
16	0	0	0	2.9	6.94	4.86	10.29
17	0	0	0	2.7	6.44	4.51	10.29
18	0	0	0	2.5	5.95	4.16	10.29
19	0	0	0	2.3	5.45	3.82	10.29
20	0	0	0	2.1	4.96	3.47	10.29
21	0	0	0	1.9	4.46	3.12	10.29
22	0	0	0	1.7	3.97	2.78	10.29
23	0	0	0	1.5	3.47	2.43	10.29
24	0	0	0	1.3	2.97	2.08	10.29
25	0	0	0	1.1	2.48	1.73	10.29
26	0	0	0	0.8	1.98	1.39	10.29
27	0	0	0	0.6	1.49	1.04	10.29
28	0	0	0	0.4	0.99	0.69	10.29
29	0	0	0	0.2	0.50	0.35	10.29
N.A.	0	0	0	0.0	0.00	0.00	10.29
Σ	15.1	Σ		92.9			

Figure AAA: Summary of Forces and Stiffnesses Calculated for Investigation

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As shown in Figure AAA on the previous page, the calculated continuity tie force is larger than the calculated chord force on all chord lines except for the perimeter chord. With the continuity tie force being larger than the chord forces, the continuity tie force design governs the chord force design when determining steel connectors for continuity ties for this example building. Because the continuity tie force design governed over the collective chord force in this example, the existing buildings was already appropriately designed for the collective chord model to be accommodated. However, the perimeter rebar chord in existing building is over designed and overly stiff because it was designed per the traditional method where the perimeter chord force is significantly larger.

Figure AAA on the previous page also gives a comparison of the axial line stiffnesses for the traditional method and the collective chord model. With the decreased perimeter chord force, the amount of steel required is about 84% lower. Because the perimeter chord force decreased, there is a corresponding decrease in the cross sectional area required for the perimeter continuous steel chord. Due to the decreased cross sectional area of steel required, the axial stiffness of the continuous rebar perimeter chord in the collective chord model also decreased. With the decreased axial stiffness of the continuous rebar chord in the collective chord model, the axial stiffness of a single line of cross ties increased to about four times larger than the perimeter chord. As such, the perimeter chord barely engages based on the relative stiffness of the cross tie chord.

7.0 CONCLUSION

While the testing done in this thesis is only a stepping stone in the research needed to further develop the appropriate design of diaphragms, the testing did show that the collective chord model does have a future in wood diaphragm design.

7.1 Support for Thesis

Overall the results found during testing and analysis supported the collective chord model. The results from the testing phase of this thesis showed that the HDU8-S2.5, HD7B single and HD7B double sided metal connectors had no bolt or screw slip during initial loading. Any slip that occurred was taken out with the half turn of the nut on the anchor rod at the beginning of testing. The half turn of the nut was required by the manufacturer's installation guidelines for the steel connectors. All three of these steel connectors behaved in a linear manner until their allowable capacity was reached, then continued to behave in a gradual non-linear manner until their ultimate capacity was reached. The ultimate capacity was about three times as large as the allowable capacity and thus the connectors have a factor of safety of about three built into them.

The completed analysis on a typical warehouse size building showed that the collective chord model will act inadvertently on an existing building designed with a traditional chord, or alternatively will act intentionally in the design of a new building. The continuity tie stiffness is large enough compared to the perimeter steel chord to engage the continuity tie members to act collectively.

The example with the collective chord model also found that the wall-roof anchorage force was slightly larger than the chord forces. Thus the continuity ties could

be generally designed for the wall-roof anchorage force and not need to be redesigned for the chord forces. Because there was about an 84% reduction in chord forces in this building example, the amount of steel required in the perimeter chord also dropped by about 84%. The drop in cross sectional area of the steel greatly reduces the stiffness of the steel chord making it not engage nearly as much, if at all, as when it was designed traditionally.

When the collective chord model was used, the chord forces were about 84% lower than when using the traditional method to design the perimeter chord force for design level forces. Because the chord forces are so low at design level, they are in the linear range of the steel connector's capacity.

7.2 Industry Relevance

The testing and analysis completed created a basis for further research into the actual static and dynamic behavior of diaphragms. The collective chord model does seem to be a reasonable approximation for how diaphragms actually behave. If more research is conducted into different shaped and sized buildings to confirm that the collective chord model will work on most buildings, then the collective chord model will be a more effective way to design diaphragms than the current traditional model.

The continuity tie force was only slightly larger than the chord forces when using the collective chord model for design in the example building. But the continuity tie steel connectors have a factor of safety of about three built into them. For new construction, of different shape or size buildings from what was analyzed in this thesis, some of the perimeter chords might have a slightly larger force than the wall anchorage force. If the

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chord forces are larger than the wall anchorage force it might be a reasonable assumption that the wall anchorage force design would be adequate because of the factor of safety of three on the steel connectors. However careful consideration should be taken to ensure that the continuity tie connectors are adequately designed for the governing force.

More research and testing needs to be conducted to solidify the actual dynamic response of a large diaphragm, but this thesis does not suggest the direction this research should take. If design level chord forces do begin to extend into the non-linear range, experimentation needs to be done to see how non-linear impacts the collective chord model.

8.0 GLOSSARY

RW/FD: Rigid wall/ flexible diaphragm

LFRS: Lateral Force Resisting System

ASCE 7-05: American Society of Civil Engineers Minimum Design Loads for Buildings
and Other Structures

IBC: International Building Code

UBC: Uniform Building Code

9.0 REFERENCES

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10.0 APPENDIX

10.1 Test Procedures

Below are the individual test procedures.

10.1.1 H1 Series Test Procedure

1. Measure and record the moisture content of the wood specimen, sampling several locations across the surface.
2. Cut the 1" diameter high strength rod for the fabricated bracket to the necessary length.
3. Attach high strength rod to fabricated bracket with a double nut assembly.
4. Attach fabricated bracket to glulam with 36 SDS screws (18 each side).
5. Attach metal connector to glulam
 - a. Photograph wood prior to attaching hold downs to document any defects in the wood.
 - b. Place hold down on the center of the middle lam of glulam such that the bracket will be installed 4" clear of end of beam. Screw hold down in place.
6. Have Riehle machine prepared:
 - a. Remove clamping jaws.
 - b. Have top head at proper elevation.
7. Put bracket and glulam in Riehle machine with bracket at top
 - a. Place 1" thick steel load plate on top of top head (with 1" high strength rod passing through).
 - b. Thread load cell over 1" diameter high strength rod
8. Place a thick 1/4" washer plate and nut above the load cell and finger tighten
9. Attach 7/8" diameter rods to steel connectors and 1" steel plate at bottom jaw of Riehle machine with nuts and provide 1/4" plate washers if necessary against the 1" plate's oversized holes. If an additional 1" steel plate is available, sandwich the lower jaw to self support the steel connector anchor rod prior to attaching to the HD7B.
10. Install low friction rollers to counteract lateral movement from the assembly eccentricity.
11. Record unbraced length of anchor rod (per AC155).
12. Attach extensometers on glulam and steel connector to measure the movement between the test bed and the wood member.

13. Measure and record moisture content of wood again if more than a week has transpired since first measured.
14. Photograph the test set up prior to the start. Provide whiteboard with test number, test specimen, date, EFI Grant number, “ARCE”, “Cal Poly, SLO”, “Lawson/Yarber”.
15. Record time of start of test and date.
16. Finger tighten as much as possible the nuts on the 7/8” diameter rod then add exactly half turn.
17. Apply a constantly increasing axial tension load until failure.
 - a. The rate of loading should be between 0.03” and 0.2” per min until peak load (record actual rate)
 - b. Record loads to nearest 3 significant figures.
 - c. Record displacements to the nearest 0.001”
 - d. Record failure load, etc. Loads recorded to at least 3 significant figures.
18. Record time at failure, length of test, and the head movement. Photograph the test specimen’s failure mechanism prior to disassembly.
19. Photograph the test specimen’s failure mechanism after disassembly if additional views are noteworthy.
20. Record failure mechanism.

10.1.2 H2 Series Test Procedure

1. Measure and record the moisture content of the wood specimen, sampling several locations across the surface.
2. Drill holes (3) 13/16” diameter in the center of glulam as follows:
 - a. Use HD7B as a template for the bolt hole locations. Locate centerline of holes away from a glue joint in the glulam. With an odd number of lams, the center of the member should be exactly in the middle of a 1 1/2” lam.
 - b. Locate holes such that the HD7B bracket will be installed 4” clear of the end of the beam.
 - c. Consider drilling the holes with a drill press to ensure they are exactly perpendicular to the surface and don’t route-out the hole any larger than 13/16” diameter.
3. Cut the 1” diameter high strength rod for the fabricated bracket to the necessary length.
4. Attach high strength rod to fabricated bracket with a double nut assembly.
5. Attach fabricated bracket to glulam with 36 SDS screws (18 each side).
6. Have Riehle machine prepared:

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- a. Remove clamping jaws.
 - b. Have top head at proper elevation.
7. Put bracket and glulam in Riehle machine with bracket at top
 - a. Place 1" thick steel load plate on top of top head (with 1" high strength rod passing through).
 - b. Thread load cell over 1" diameter high strength rod
 - c. Place a thick washer plate and nut above the load cell and finger tighten
8. Attach metal connectors to glulam
 - a. Photograph wood prior to attaching hold downs to document any defects.
 - b. Loosely hand tighten the 3/4" bolts in place with standard cut washers against the wood. Do not fully tighten until after the anchor rod is tightened.
9. Attach 7/8" diameter rods to metal connectors and 1" steel plate at bottom jaw of Riehle machine with nuts and provide 1/4" plate washers if necessary against the 1" plate's oversized holes. If an additional 1" steel plate is available, sandwich the lower jaw to self support the hold down anchor rod prior to attaching to the HD7B.
10. Install low friction rollers to counteract lateral movement from the assembly eccentricity.
11. Tighten snugly the loose 3/4" nuts on the three hold down bolts (see step 7a).
12. Record unbraced length of anchor rod (per AC155).
13. Attach extensometers on glulam and steel connector to measure the movement between the test bed and the wood member.
14. Measure and record moisture content of wood again if more than a week has transpired since first measured.
15. Photograph the test set up prior to the start. Provide whiteboard with test number, test specimen, date, EFI Grant number, "ARCE", "Cal Poly, SLO", "Lawson/Yarber".
16. Record time of start of test and date.
17. Finger tighten as much as possible the nuts on the 7/8" diameter rod then add exactly half turn.
18. Apply a constantly increasing axial tension load until failure.
 - a. The rate of loading should be between 0.03" and 0.2" per min until peak load (record actual rate)
 - b. Record loads to nearest 3 significant figures.
 - c. Record displacements to the nearest 0.001"
 - d. Record failure load, etc. Loads recorded to at least 3 significant figures.

19. Record time at failure, length of test, and the head movement. Photograph the test specimen's failure mechanism prior to disassembly.
20. Photograph the test specimen's failure mechanism after disassembly if additional views are noteworthy.
21. Record failure mechanism.

10.1.3 H3 Series Test Procedure

1. Measure and record the moisture content of the wood specimen, sampling several locations across the surface.
2. Drill holes (3) 13/16" diameter in the center of glulam as follows.
 - a. Use HD7B as a template for the bolt hole locations. Locate centerline of holes away from a glue joint in the glulam. With an odd number of lams, the center of the member should be exactly in the middle of a 1 1/2" lam.
 - b. Locate holes such that the HD7B bracket will be installed 4" clear of the end of the beam.
 - c. Consider drilling the holes with a drill press to ensure they are exactly perpendicular to the surface and don't route-out the hole any larger than 13/16" diameter.
3. Cut the 1" diameter high strength rod for the fabricated bracket to the necessary length.
4. Attach high strength rod to fabricated bracket with a double nut assembly.
5. Attach fabricated bracket to glulam with 36 SDS screws (18 each side).
6. Have Riehle machine prepared:
 - a. Remove clamping jaws.
 - b. Have top head at proper elevation.
7. Put bracket and glulam in Riehle machine with bracket at top
 - a. Place 1" thick steel load plate on top of top head (with 1" high strength rod passing thru).
 - b. Thread load cell over 1" diameter high strength rod
 - c. Place a thick washer plate and nut above the load cell and finger tighten
8. Attach steel connectors to glulam
 - a. Photograph wood prior to attaching steel connectors to document any defects in the wood.
 - b. Loosely hand tighten the 3/4" bolts in place with nuts against the backside hold down. Do not fully tighten until after the anchor rod is tightened.
9. Attach 7/8" dia. rods to hold downs and 1" steel plate at bottom jaw of Riehle machine with nuts and provide 1/4" plate washers if necessary against the 1"

plate's oversized holes. If an additional 1" steel plate is available, sandwich the lower jaw to self support the hold down anchor rod prior to attaching to the HD7B.

10. Tighten snugly the loose 3/4" nuts on the three hold down bolts (see step 7a).
11. Record unbraced length of anchor rod (per AC155).
12. Attach extensometers on glulam and hold down to measure the movement between the test bed and the wood member.
13. Measure and record moisture content of wood again if more than a week has transpired since first measured.
14. Photograph the test set up prior to the start. Provide whiteboard with test number, test specimen, date, EFI Grant number, "ARCE", "Cal Poly, SLO", "Lawson/Yarber".
15. Record time of start of test and date.
16. Finger tighten as much as possible the nuts on the 7/8" diameter rod then add exactly half turn.
17. Apply a constantly increasing axial tension load until failure.
 - a. The rate of loading should be between 0.03" and 0.2" per min until peak load (record actual rate)
 - b. Record loads to nearest 3 significant figures.
 - c. Record displacements to the nearest 0.001"
 - d. Record failure load, etc. Loads recorded to at least 3 significant figures.
18. Record time at failure, length of test, and the head movement. Photograph the test specimen's failure mechanism prior to disassembly.
19. Photograph the test specimen's failure mechanism after disassembly if additional views are noteworthy.
20. Record failure mechanism.

10.1.4 S1 Series Test Procedure

1. Measure and record the moisture content of the wood specimen, sampling several locations across the surface.
2. Cut OSB into (2) 8 1/4" x 30 1/4" pieces
3. Measure a line 1" from the edge of OSB piece. Then mark every 2 1/2" on that line for nail placement to attach to glulam beam. Repeat for both pieces of OSB.
4. Align both pieces of OSB on the bottom of the short side of the glulam. Center the two pieces along the short edge of the glulam. Leave a 1/8" gap between OSB pieces.
5. Attach OSB to glulam beam with 10d nails at premarked spots.

6. Place MST 60 strap centered on top of OSB pieces. Leave a 3" gap from the end of the plywood to the end of the strap.
7. Attach MST 60 strap with 30 10d nails, starting at the finished end.
8. Cut the 1" diameter high strength rod for the fabricated bracket to the necessary length.
9. Attach high strength rod to fabricated bracket with a double nut assembly.
10. Attach fabricated bracket to glulam with 36 SDS screws (18 each side).
11. Have Riehle machine prepared:
 - a. Remove clamping jaws.
 - b. Have top head at proper elevation.
12. Put bracket and glulam in Riehle machine with bracket at top
 - a. Place 1" thick steel load plate on top of top head (with 1" high strength rod passing through).
 - b. Thread load cell over 1" diameter high strength rod
 - c. Place a thick 1/4" washer plate and nut above the load cell and finger tighten
13. Install low friction rollers to counteract lateral movement from the assembly eccentricity.
14. Attach extensometers on glulam and steel connector to measure the movement between the test bed and the wood member as well as the plywood and wood member.
15. Clamp lower head jaw of the Riehle machine onto MST 60 strap.
16. Measure and record moisture content of wood again if more than a week has transpired since first measured.
17. Photograph the test set up prior to the start. Provide whiteboard with test number, test specimen, date, EFI Grant number, "ARCE", "Cal Poly, SLO", "Lawson/Yarber".
18. Record time of start of test and date.
19. Apply a constantly increasing axial tension load until failure.
 - a. The rate of loading should be between 0.03" and 0.2" per min until peak load (record actual rate)
 - b. Record loads to nearest 3 significant figures.
 - c. Record displacements to the nearest 0.001"
 - d. Record failure load, etc. Loads recorded to at least 3 significant figures.
20. Record time at failure, length of test, and the head movement. Photograph the test specimen's failure mechanism prior to disassembly.

21. Photograph the test specimen's failure mechanism after disassembly if additional views are noteworthy.
22. Record failure mechanism.

10.2 Test Setup Configurations

Below are the technical drawings of the individual test setups.

10.2.1 H1Series HDU8-S2.5

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

10.2.2 H2 Series HD7B Single Sided

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10.2.3 H3 Series HD7B Double Sided

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

10.2.4 S1 Series MST 60 Strap Single Sided

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

10.3 Testing Data Tables

The following tables are samples of the raw data and calibrated data from the completed tests.

Readings	Load	Zeroed load	Zero after 1/2 turn	Head	1 Red	Zeroed red 1	2 Red	Zeroed red 2	Average red	1 Blue	Zeroed blue 1	2 Blue	Zeroed blue 2	Average blue	Difference between Avg Red and Avg Blue
129.002	-1	-10	0	0.004	0.001	0	-0.001	0	0	0	0	0	0	0	0
130.002	-3	-12	-2	0.005	0.001	0	-0.001	0	0	0	0	0	0	0	0
131.002	-3	-12	-2	0.005	0.001	0	-0.001	0	0	0	0	0	0	0	0
132.003	-4	-13	-3	0.005	0.001	0	-0.001	0	0	0	0	0	0	0	0
133.002	183	174	184	0.004	0.002	0.001	0.001	0.002	0.0015	0	0	-0.003	-0.003	0.0015	0
134.003	394	385	395	0.004	0.004	0.003	0.003	0.004	0.0035	0	0	-0.005	-0.005	0.0025	0.001
135.002	591	582	592	0.005	0.006	0.005	0.006	0.007	0.006	0	0	-0.007	-0.007	0.0035	0.0025
136.002	784	775	785	0.004	0.008	0.007	0.008	0.009	0.008	-0.001	-0.001	-0.01	-0.01	0.0055	0.0025
137.002	972	963	973	0.004	0.01	0.009	0.011	0.012	0.0105	-0.001	-0.001	-0.012	-0.012	0.0065	0.004
138.002	1154	1145	1155	0.004	0.012	0.011	0.014	0.015	0.013	-0.001	-0.001	-0.014	-0.014	0.0075	0.0055
139.002	1329	1320	1330	0.004	0.014	0.013	0.016	0.017	0.015	-0.001	-0.001	-0.016	-0.016	0.0085	0.0065
140.002	1503	1494	1504	0.004	0.016	0.015	0.018	0.019	0.017	-0.001	-0.001	-0.019	-0.019	0.01	0.007
141.003	1671	1662	1672	0.005	0.018	0.017	0.021	0.022	0.0195	-0.001	-0.001	-0.021	-0.021	0.011	0.0085
142.003	1849	1840	1850	0.005	0.02	0.019	0.022	0.023	0.021	-0.001	-0.001	-0.023	-0.023	0.012	0.009
143.003	2025	2016	2026	0.004	0.023	0.022	0.025	0.026	0.024	-0.001	-0.001	-0.026	-0.026	0.0135	0.0105
144.002	2199	2190	2200	0.004	0.025	0.024	0.027	0.028	0.026	-0.001	-0.001	-0.028	-0.028	0.0145	0.0115
145.002	2362	2353	2363	0.005	0.027	0.026	0.029	0.03	0.028	-0.001	-0.001	-0.031	-0.031	0.016	0.012
146.004	2530	2521	2531	0.006	0.03	0.029	0.032	0.033	0.031	-0.001	-0.001	-0.033	-0.033	0.017	0.014
147.003	2696	2687	2697	0.004	0.032	0.031	0.034	0.035	0.033	-0.001	-0.001	-0.036	-0.036	0.0185	0.0145
148.003	2860	2851	2861	0.004	0.035	0.034	0.036	0.037	0.0355	-0.001	-0.001	-0.038	-0.038	0.0195	0.016
149.002	3025	3016	3026	0.004	0.036	0.035	0.039	0.04	0.0375	-0.001	-0.001	-0.041	-0.041	0.021	0.0165

Theoretical Rod Field $\Delta = PL/AE$	Theoretical Rod Testing $\Delta = PL/AE$	$\Delta_{\text{calibration}} = \Delta_{\text{testing}} - \Delta_{\text{field}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Red}} - \Delta_{\text{calibration}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Blue}} - \Delta_{\text{calibration}}$	Machine Rate of Loading
-1.10091E-06	-2.56568E-06	-1.46477E-06	1.46477E-06	1.46477E-06	0
-1.10091E-06	-2.56568E-06	-1.46477E-06	1.46477E-06	1.46477E-06	0
-1.65137E-06	-3.84852E-06	-2.19716E-06	2.19716E-06	2.19716E-06	0.076
0.000101284	0.000236043	0.000134759	0.001365241	0.001365241	0.105
0.00021743	0.000506722	0.000289292	0.003210708	0.002210708	0.12
0.00032587	0.000759442	0.000433572	0.005566428	0.003066428	0.246
0.000432107	0.00100703	0.000574923	0.007425077	0.004925077	0.199
0.000535593	0.001248204	0.000712611	0.009787389	0.005787389	0.187
0.000635776	0.001481681	0.000845905	0.012154095	0.006554095	0.204
0.000732106	0.001706178	0.000974073	0.014025927	0.007525927	0.482
0.000827885	0.001929392	0.001101508	0.015898492	0.008988492	0.213
0.000920361	0.00214491	0.001224548	0.018275452	0.009775452	0.192
0.001018342	0.002373255	0.001354913	0.019645087	0.010645087	0.21
0.001115222	0.002599035	0.001483813	0.022516187	0.012016187	0.2
0.001211002	0.002822225	0.001611248	0.024388752	0.012888752	0.125
0.001300726	0.003031353	0.001730627	0.026269373	0.014269373	0.15
0.001393202	0.00324687	0.001853668	0.029146332	0.015146332	0.187
0.001484578	0.003459821	0.001975244	0.031024756	0.016524756	0.182
0.001574853	0.003670207	0.002095355	0.033404645	0.017404645	0
0.001665678	0.003881876	0.002216198	0.035283802	0.018783802	0.143

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

Readings	Load	Zeroed load	Zeroed load after 1/2 turn	Head	1 Red	Zeroed red 1	2 Red	Zeroed red 2	Average red	1 Blue	Zeroed blue 1	2 Blue	Zeroed blue 2	Average blue	Difference between Avg Red and Avg Blue
111.002	-9	-18	0	0.004	0	0	0.001	0	0	0	0	0	0	0	0
112.003	8	-1	17	0.005	0.001	0.001	0.001	0	0.0005	0	0	-0.001	-0.001	0.0005	0
113.007	253	244	262	0.005	0.002	0.002	0.002	0.001	0.0015	-0.001	-0.001	-0.003	-0.003	0.002	-0.0005
114.003	471	462	480	0.005	0.004	0.004	0.003	0.002	0.003	-0.003	-0.003	-0.005	-0.005	0.004	-0.001
115.003	686	677	695	0.004	0.006	0.006	0.005	0.004	0.005	-0.004	-0.004	-0.007	-0.007	0.0055	-0.0005
116.003	898	889	907	0.004	0.007	0.007	0.005	0.004	0.0055	-0.005	-0.005	-0.009	-0.009	0.007	-0.0015
117.002	1119	1110	1128	0.006	0.009	0.009	0.007	0.006	0.0075	-0.007	-0.007	-0.011	-0.011	0.009	-0.0015
118.003	1336	1327	1345	0.004	0.011	0.011	0.008	0.007	0.009	-0.008	-0.008	-0.014	-0.014	0.011	-0.002
119.002	1469	1460	1478	0.004	0.011	0.011	0.001	0	0.0055	-0.008	-0.008	-0.018	-0.018	0.013	-0.0075
120.002	1621	1612	1630	0.004	0.012	0.012	0.004	0.003	0.0075	-0.009	-0.009	-0.02	-0.02	0.0145	-0.007
121.003	1846	1837	1855	0.004	0.014	0.014	0.005	0.004	0.009	-0.011	-0.011	-0.023	-0.023	0.017	-0.008
122.003	2057	2048	2066	0.004	0.016	0.016	0.007	0.006	0.011	-0.012	-0.012	-0.025	-0.025	0.0185	-0.0075
123.002	2254	2245	2263	0.004	0.018	0.018	0.009	0.008	0.013	-0.014	-0.014	-0.027	-0.027	0.0205	-0.0075
124.003	2455	2446	2464	0.004	0.02	0.02	0.011	0.01	0.015	-0.016	-0.016	-0.03	-0.03	0.023	-0.008
125.002	2652	2643	2661	0.005	0.022	0.022	0.012	0.011	0.0165	-0.017	-0.017	-0.032	-0.032	0.0245	-0.008
126.002	2842	2833	2851	0.004	0.024	0.024	0.015	0.014	0.019	-0.019	-0.019	-0.035	-0.035	0.027	-0.008
127.002	3024	3015	3033	0.005	0.026	0.026	0.017	0.016	0.021	-0.02	-0.02	-0.037	-0.037	0.0285	-0.0075
128.002	3202	3193	3211	0.004	0.028	0.028	0.02	0.019	0.0235	-0.022	-0.022	-0.04	-0.04	0.031	-0.0075
129.003	3371	3362	3380	0.005	0.029	0.029	0.022	0.021	0.025	-0.024	-0.024	-0.042	-0.042	0.033	-0.008
130.002	3539	3530	3548	0.003	0.032	0.032	0.024	0.023	0.0275	-0.026	-0.026	-0.045	-0.045	0.0355	-0.008
131.002	3702	3693	3711	0.005	0.034	0.034	0.026	0.025	0.0295	-0.028	-0.028	-0.048	-0.048	0.038	-0.0085

Theoretical Rod Field $\Delta = PL/AE$	Theoretical Rod Testing $\Delta = PL/AE$	$\Delta_{\text{calibration}} = \Delta_{\text{reading}} - \Delta_{\text{field}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Head}} - \Delta_{\text{Calibration}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Blue}} - \Delta_{\text{Calibration}}$	Machine Rate of Loading
9.35774E-06	2.13325E-05	1.19747E-05	0.000488025	0	0
0.000144219	0.00028771	0.000184552	0.001315448	0.001815448	0.139
0.000264219	0.000602329	0.00033811	0.00266189	0.00366189	0.19
0.000382566	0.000872122	0.000489555	0.004510445	0.005010445	0.183
0.000499263	0.00113815	0.000638887	0.004861113	0.005361113	0.179
0.000620914	0.001415472	0.000794559	0.006705441	0.008205441	0.121
0.000740362	0.001687775	0.000947413	0.008052587	0.010052587	0.197
0.000813573	0.00185467	0.001041098	0.004658902	0.011958902	0.186
0.000897242	0.002045408	0.001148166	0.006351834	0.013351834	0.181
0.001021095	0.002327749	0.001306655	0.007693345	0.015693345	0
0.001137241	0.002592523	0.001455283	0.009544717	0.017044717	0.141
0.00124568	0.002839729	0.001594049	0.011405951	0.018905951	0.191
0.001356322	0.003091954	0.001735632	0.013264368	0.021264368	0.183
0.001464762	0.00333916	0.001874398	0.014625602	0.022625602	0.179
0.001569348	0.003577582	0.002008234	0.016991766	0.024991766	0.123
0.001669531	0.003805965	0.002136434	0.018863566	0.026363566	0.198
0.001767512	0.004029328	0.002261816	0.021238184	0.028738184	0.186
0.001860539	0.004241398	0.002380859	0.022619141	0.030619141	0.181
0.001953015	0.004452213	0.002499198	0.02500802	0.03300802	0
0.00204274	0.004656754	0.002614014	0.026885986	0.035385986	0.212

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

Readings	Load	Zeroed Load	Zeroed Load after 1/2 turn	Head	1 Red	Zeroed red 1	2 Red	Zeroed red 2	Average red	1 Blue	Zeroed blue 1	2 Blue	Zeroed blue 2	Average blue	Difference between Avg Red and Avg Blue
75.002	-6	-5	0	0.004	0	0	0.001	0	0	0	0	0	0	0	0
76.002	-5	-4	1	0.004	0	0	0.001	0	0	0	0	0	0	0	0
77.002	-6	-5	0	0.004	0	0	0.001	0	0	0	0	0	0	0	0
78.002	183	184	189	0.005	0	0	0.003	0.002	0.001	-0.002	-0.002	-0.002	-0.002	0.002	-0.001
79.001	309	310	315	0.004	-0.001	-0.001	0.005	0.004	0.0015	-0.003	-0.003	-0.006	-0.006	0.0045	-0.003
80.002	495	496	501	0.004	-0.001	-0.001	0.007	0.006	0.0025	-0.005	-0.005	-0.008	-0.008	0.0065	-0.004
81.003	671	672	677	0.004	0.001	0.001	0.01	0.009	0.005	-0.007	-0.007	-0.01	-0.01	0.0085	-0.0035
82.002	839	840	845	0.005	0.002	0.002	0.012	0.011	0.0065	-0.009	-0.009	-0.013	-0.013	0.011	-0.0045
83.003	1008	1009	1014	0.005	0.004	0.004	0.014	0.013	0.0085	-0.011	-0.011	-0.015	-0.015	0.013	-0.0045
84.004	1183	1184	1189	0.004	0.005	0.005	0.016	0.015	0.01	-0.013	-0.013	-0.018	-0.018	0.0155	-0.0055
85.003	1365	1366	1371	0.005	0.005	0.007	0.018	0.017	0.012	-0.015	-0.015	-0.02	-0.02	0.0175	-0.0055
86.003	1537	1538	1543	0.005	0.009	0.009	0.021	0.02	0.0145	-0.018	-0.018	-0.022	-0.022	0.02	-0.0055
87.003	1705	1706	1711	0.005	0.011	0.011	0.023	0.022	0.0165	-0.02	-0.02	-0.025	-0.025	0.0225	-0.006
88.002	1806	1807	1812	0.004	0.012	0.012	0.024	0.023	0.0175	-0.021	-0.021	-0.028	-0.028	0.0245	-0.007
89.003	1973	1974	1979	0.004	0.013	0.013	0.025	0.024	0.0185	-0.023	-0.023	-0.03	-0.03	0.0265	-0.008
90.003	2121	2122	2127	0.005	0.015	0.015	0.027	0.026	0.0205	-0.025	-0.025	-0.033	-0.033	0.029	-0.0085
91.002	2274	2275	2280	0.005	0.017	0.017	0.03	0.029	0.023	-0.027	-0.027	-0.035	-0.035	0.031	-0.008
92.003	2424	2425	2430	0.004	0.019	0.019	0.032	0.031	0.025	-0.03	-0.03	-0.038	-0.038	0.034	-0.009
93.003	2573	2574	2579	0.004	0.021	0.021	0.034	0.033	0.027	-0.032	-0.032	-0.041	-0.041	0.0365	-0.0095
94.003	2716	2717	2722	0.004	0.023	0.023	0.036	0.035	0.029	-0.034	-0.034	-0.043	-0.043	0.0385	-0.0095
95.003	2859	2860	2865	0.004	0.026	0.026	0.039	0.038	0.032	-0.036	-0.036	-0.046	-0.046	0.041	-0.009

Theoretical Rod Field $\Delta = P_L/AE$	Theoretical Rod Testing $\Delta = P_L/AE$	$\Delta_{calibration} = \Delta_{testing} - \Delta_{field}$	$\Delta_{actual} = \Delta_{red} - \Delta_{calibration}$	$\Delta_{actual} = \Delta_{blue} - \Delta_{calibration}$	Machine Rate of Loading
5.0455E-07	1.28284E-06	7.32385E-07	-7.32385E-07	-7.32385E-07	0
0	0	0	0	0	0.137
0.000104036	0.000242457	0.000138421	0.000861579	0.001861579	0.15
0.000173393	0.000404095	0.000230701	0.001269299	0.00269299	0.155
0.000275778	0.000642703	0.000366925	0.002133075	0.006133075	0.178
0.000372658	0.000868483	0.000495825	0.004504175	0.00804175	0.233
0.000465135	0.001084	0.000618866	0.005881134	0.01081134	0.194
0.000558162	0.0013008	0.000742639	0.00757361	0.01257361	0.184
0.000654491	0.001525298	0.000870806	0.009129194	0.014629194	0.202
0.000754674	0.001758775	0.0010041	0.0109559	0.0164959	0.393
0.000849353	0.001979423	0.001130071	0.013369929	0.01869929	0.206
0.000941829	0.00219494	0.001253111	0.015246889	0.021246889	0.188
0.000997425	0.002324507	0.001327082	0.016172918	0.023172918	0.207
0.001089351	0.002538742	0.001449391	0.017050609	0.025050609	0.198
0.001170818	0.002728602	0.001557784	0.018942216	0.027442216	0.118
0.001255038	0.002924877	0.001669839	0.021330161	0.029330161	0.195
0.001337606	0.003117303	0.001779697	0.023220303	0.032220303	0.185
0.001419624	0.003308446	0.001888822	0.025111178	0.034611178	0.18
0.001498339	0.003491892	0.001993553	0.027006447	0.036506447	0
0.001577054	0.003675339	0.002098284	0.029901716	0.038901716	0.207

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

10.3.4 H2.1

H2.1 12/8/11 12:50									
Spreadsheet calculation									
User Input									
		Average Rate of loading =	0.18	Inches/Minute					Moisture Content
		Test length :	12:46	Min:Sec					8.8
		Test Date :	12/8/2011						8.8
		Test Time :	12:50	pm					9.6
		Average Moisture content =	9.3	%					9.2
		Wood Temperature =	57	° F					8.9
		Testing Rod Length =	17.5	Inch					9.3
		Theoretical Rod Length =	7.375	Inch					9.8
		Head movement=	2.246	Inch					10
		Calculated Average rate of loading=	0.18	Inches/minute					9.2
								Avg MC	9.3 %
		3:49 First split. Split failure						7/8" diam. Rod properties	
		12:46 stopped. Took about half of the load for a long time.						A =	0.462 in ²
								E =	29000000 psi

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

Readings	Load	Zeroed Load	Zeroed load after 1/2 turn	Head	1 Red	Zeroed red 1	2 Red	Zeroed red 2	Average red	1 Blue	Zeroed blue 1	2 Blue	Zeroed blue 2	Average blue	Difference between Avg Red and Avg Blue
239.004	0	-12	0	-0.005	0	0	0	0	0	0.001	0	0	0	0	0
240.003	2	-10	2	-0.006	0	0	0	0	0	0.001	0	0	0	0	0
241.004	-7	-19	-7	-0.005	0	0	0	0	0	0.001	0	0	0	0	0
242.003	73	61	73	-0.005	0	0	0	0	0	0	-0.001	-0.002	-0.002	0.0015	-0.0015
243.003	200	188	200	-0.006	0.002	0.002	0.004	0.002	0.002	-0.001	-0.002	-0.004	-0.004	0.003	-0.001
244.003	290	278	290	-0.005	0.004	0.004	0.006	0.004	0.004	-0.002	-0.003	-0.006	-0.006	0.0045	-0.0005
245.002	369	357	369	-0.006	0.006	0.006	0.009	0.007	0.0065	-0.004	-0.005	-0.009	-0.009	0.007	-0.0005
246.003	442	430	442	-0.004	0.008	0.008	0.012	0.01	0.009	-0.006	-0.007	-0.011	-0.011	0.009	0
247.002	523	511	523	-0.005	0.01	0.01	0.014	0.012	0.011	-0.007	-0.008	-0.014	-0.014	0.011	0
248.003	607	595	607	-0.007	0.012	0.012	0.016	0.014	0.013	-0.009	-0.01	-0.016	-0.016	0.013	0
249.003	704	692	704	-0.005	0.014	0.014	0.019	0.017	0.0155	-0.011	-0.012	-0.018	-0.018	0.015	0.0005
250.002	817	805	817	-0.007	0.015	0.015	0.021	0.019	0.017	-0.013	-0.014	-0.02	-0.02	0.017	0
251.003	936	924	936	-0.006	0.017	0.017	0.024	0.022	0.0195	-0.015	-0.016	-0.022	-0.022	0.019	0.0005
252.003	1052	1040	1052	-0.007	0.019	0.019	0.026	0.024	0.0215	-0.018	-0.019	-0.024	-0.024	0.0215	0
253.002	1166	1154	1166	-0.007	0.021	0.021	0.029	0.027	0.024	-0.02	-0.021	-0.027	-0.027	0.024	0
254.002	1277	1265	1277	-0.006	0.023	0.023	0.032	0.03	0.0265	-0.022	-0.023	-0.029	-0.029	0.026	0.0005
255.003	1396	1384	1396	-0.006	0.025	0.025	0.034	0.032	0.0285	-0.024	-0.025	-0.031	-0.031	0.028	0.0005
256.003	1522	1510	1522	-0.005	0.026	0.026	0.036	0.034	0.03	-0.026	-0.027	-0.033	-0.033	0.03	0
257.002	1652	1640	1652	-0.007	0.028	0.028	0.039	0.037	0.0325	-0.028	-0.029	-0.035	-0.035	0.032	0.0005
258.003	1780	1768	1780	-0.007	0.03	0.03	0.042	0.04	0.035	-0.03	-0.031	-0.037	-0.037	0.034	0.001
259.002	1909	1897	1909	-0.006	0.032	0.032	0.044	0.042	0.037	-0.032	-0.033	-0.039	-0.039	0.036	0.001

Theoretical Rod Field $\Delta = P/AE$	Theoretical Rod Testing $\Delta = P/AE$	$\Delta_{\text{Calibration}} = \Delta_{\text{Testing}} - \Delta_{\text{yield}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Red}} - \Delta_{\text{Calibration}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Blue}} - \Delta_{\text{Calibration}}$	Machine Rate of Loading
1.10091E-06	2.61233E-06	1.51142E-06	-1.51142E-06	-1.51142E-06	0
-3.85319E-06	-9.14316E-06	-5.28997E-06	5.28997E-06	5.28997E-06	0
4.01832E-05	9.53501E-05	5.51668E-05	-5.51668E-05	0.001444833	0.054
0.000110091	0.000261233	0.000151142	0.001848858	0.002848858	0.14
0.000159632	0.000378788	0.000219156	0.003780844	0.009280844	0.157
0.000203118	0.000481975	0.000278857	0.006221143	0.006721143	0.16
0.000243301	0.000577325	0.000334024	0.008665976	0.008665976	0.162
0.000287888	0.000683124	0.000395236	0.010604764	0.010604764	0.887
0.000334126	0.000792842	0.000458716	0.012541284	0.012541284	0.229
0.000387521	0.00091954	0.00053202	0.01456798	0.01456798	0.199
0.000449722	0.001067137	0.000617415	0.016382585	0.016382585	0.189
0.000515226	0.001222571	0.000707344	0.018792656	0.018792656	0.183
0.000579079	0.001374086	0.000795007	0.020704993	0.020704993	0.141
0.000641831	0.001522989	0.000881158	0.023118842	0.023118842	0.157
0.000702931	0.001667973	0.000965041	0.025534959	0.025534959	0.161
0.000768436	0.001823406	0.001054971	0.0274445029	0.0274445029	0.162
0.000837793	0.001987983	0.00115019	0.02884981	0.02884981	0.969
0.000909352	0.002157785	0.001248433	0.031251567	0.031251567	0.23
0.00097981	0.002324974	0.001345163	0.033654837	0.033654837	0.2
0.001050819	0.002493469	0.00144265	0.03555735	0.03555735	0.189

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

Readings	Load	Zeroed Load	Zeroed load after 1/2 turn	Head	1 Red	Zeroed red 1	2 Red	Zeroed red 2	Average red	1 Blue	Zeroed blue 1	2 Blue	Zeroed blue 2	Average blue	Difference between Avg Red and Avg Blue
138.003	-56	-65	0	-0.006	-0.002	0	-0.002	0	0	0.001	0	0.001	0	0	0
139.003	100	91	156	-0.006	0	0.002	0.001	0.003	0.0025	-0.001	-0.002	-0.001	-0.002	0.002	0.0005
140.004	259	250	315	-0.005	0.001	0.003	0.003	0.005	0.004	-0.003	-0.004	-0.004	-0.005	0.0045	-0.0005
141.001	395	386	451	-0.005	0.003	0.005	0.006	0.008	0.0065	-0.005	-0.006	-0.006	-0.007	0.0065	0
142.009	521	512	577	-0.006	0.006	0.008	0.008	0.01	0.009	-0.007	-0.008	-0.008	-0.009	0.0085	0.0005
143.002	659	650	715	-0.005	0.008	0.01	0.011	0.013	0.0115	-0.009	-0.01	-0.01	-0.011	0.0105	0.001
144.003	797	788	853	-0.007	0.009	0.011	0.013	0.015	0.013	-0.011	-0.012	-0.013	-0.014	0.013	0
145.002	942	933	998	-0.006	0.011	0.013	0.016	0.018	0.0155	-0.012	-0.013	-0.013	-0.014	0.0145	0.001
146.002	1081	1072	1137	-0.006	0.014	0.016	0.019	0.021	0.0185	-0.014	-0.015	-0.017	-0.018	0.0165	0.002
147.003	1220	1211	1276	-0.005	0.015	0.017	0.022	0.024	0.0205	-0.016	-0.017	-0.02	-0.021	0.019	0.0015
148.002	1370	1361	1426	-0.006	0.018	0.02	0.025	0.027	0.0235	-0.018	-0.019	-0.022	-0.023	0.021	0.0025
149.002	1515	1506	1571	-0.005	0.019	0.021	0.027	0.029	0.025	-0.02	-0.021	-0.024	-0.025	0.023	0.002
150.003	1662	1653	1718	-0.006	0.021	0.023	0.03	0.032	0.0275	-0.022	-0.023	-0.027	-0.028	0.0255	0.002
151.003	1814	1805	1870	-0.007	0.023	0.025	0.033	0.035	0.03	-0.024	-0.025	-0.029	-0.03	0.0275	0.0025
152.003	1968	1959	2024	-0.006	0.025	0.027	0.036	0.038	0.0325	-0.026	-0.027	-0.031	-0.032	0.0295	0.003
153.003	2127	2118	2183	-0.006	0.027	0.029	0.038	0.04	0.0345	-0.028	-0.029	-0.034	-0.035	0.032	0.0025
154.003	2289	2280	2345	-0.006	0.029	0.031	0.041	0.043	0.037	-0.03	-0.031	-0.036	-0.037	0.034	0.003
155.002	2452	2443	2508	-0.006	0.031	0.033	0.043	0.045	0.039	-0.032	-0.033	-0.038	-0.039	0.036	0.003
156.003	2618	2609	2674	-0.006	0.034	0.036	0.046	0.048	0.042	-0.034	-0.035	-0.04	-0.041	0.038	0.004
157.002	2787	2778	2843	-0.007	0.036	0.038	0.048	0.05	0.044	-0.036	-0.037	-0.043	-0.044	0.0405	0.0035
158.003	2957	2948	3013	-0.007	0.038	0.04	0.05	0.052	0.046	-0.038	-0.039	-0.045	-0.046	0.0425	0.0035

Theoretical Rod Field $\Delta = PL/AE$	Theoretical Rod Testing $\Delta = PL/AE$	$\Delta_{calibration} = \Delta_{testing} - \Delta_{field}$	$\Delta_{actual} = \Delta_{load} - \Delta_{calibration}$	$\Delta_{actual} = \Delta_{load} - \Delta_{calibration}$	Machine Rate of Loading
8.5871E-05	0.000205945	0.000120074	0.002379926	0.001879926	0.091
0.000173393	0.000415885	0.000242457	0.003757543	0.004257543	0.476
0.000248255	0.000595392	0.000347137	0.006152863	0.006152863	0.213
0.000317613	0.000761732	0.000444119	0.008555881	0.008555881	0.192
0.000393576	0.000943914	0.000550339	0.010949661	0.00949661	0.21
0.000469538	0.001126096	0.000656558	0.01243442	0.01243442	0.2
0.000549354	0.001317519	0.000768165	0.014731885	0.013731835	0.124
0.000625868	0.001501022	0.000875154	0.017624846	0.015624846	0.15
0.000702381	0.001684524	0.000982143	0.019517857	0.018017857	0.187
0.000784949	0.001867548	0.001097599	0.022402401	0.019902401	0.182
0.000864765	0.002073971	0.001209206	0.023790794	0.021790794	0
0.000945682	0.002268034	0.001322352	0.026177648	0.024177648	0.142
0.001029351	0.002468699	0.001439347	0.028560653	0.026060653	0.192
0.001114122	0.002672003	0.001557882	0.030942118	0.027942118	0.184
0.001201644	0.002881909	0.001680265	0.032819735	0.030319735	0.18
0.001290818	0.003095775	0.001804957	0.035195043	0.032195043	0.25
0.001380542	0.003310961	0.001930419	0.037669581	0.034669581	0.2
0.001471917	0.003530107	0.002056819	0.03994181	0.03694181	0.187
0.001564944	0.003753214	0.00218827	0.04181173	0.03881173	0.182
0.001658522	0.003977641	0.002319119	0.043680881	0.040680881	0.508

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

Readings	Load	Zeroed Load	Zeroed load after 1/2 turn	Head	1 Red	Zeroed red1	2 Red	Zeroed red 2	Average red	1 Blue	Zeroed blue 1	2 Blue	Zeroed blue 2	Average blue	Difference between Avg Red and Avg Blue
129.003	-6	-5	0	-0.005	0	0	0.001	0	0	0	0	0	0	0	0
130.002	25	26	31	-0.005	0	0	0	0	0	0	0	-0.001	-0.001	0.0005	-0.0005
131.003	190	191	196	-0.005	0.002	0.002	0.003	0.002	0.002	0	0	-0.003	-0.003	0.0015	0.0005
132.003	341	342	347	-0.005	0.004	0.004	0.006	0.005	0.0045	0	0	-0.006	-0.006	0.003	0.0015
133.004	470	471	476	-0.006	0.006	0.006	0.008	0.007	0.0065	0	0	-0.008	-0.008	0.004	0.0025
134.003	595	596	601	-0.006	0.008	0.008	0.01	0.009	0.0085	0	0	-0.01	-0.01	0.005	0.0035
135.003	705	706	711	-0.005	0.011	0.011	0.012	0.011	0.011	0	0	-0.013	-0.013	0.0065	0.0045
136.003	818	819	824	-0.005	0.013	0.013	0.014	0.013	0.013	0	0	-0.015	-0.015	0.0075	0.0055
137.002	933	934	939	-0.006	0.016	0.016	0.016	0.015	0.0155	0	0	-0.018	-0.018	0.009	0.0065
138.002	1055	1056	1061	-0.004	0.018	0.018	0.018	0.017	0.0175	0	0	-0.02	-0.02	0.01	0.0075
139.003	1182	1183	1188	-0.006	0.02	0.02	0.021	0.02	0.02	0	0	-0.023	-0.023	0.0115	0.0085
140.003	1309	1310	1315	-0.006	0.022	0.022	0.023	0.022	0.022	0	0	-0.025	-0.025	0.0125	0.0095
141.003	1435	1435	1440	-0.006	0.023	0.023	0.025	0.024	0.0235	-0.001	-0.001	-0.027	-0.027	0.014	0.0095
142.002	1565	1566	1571	-0.005	0.025	0.025	0.027	0.026	0.0255	-0.001	-0.001	-0.03	-0.03	0.0155	0.01
143.002	1582	1583	1588	-0.006	0.027	0.027	0.031	0.03	0.0285	-0.001	-0.001	-0.035	-0.035	0.018	0.0105
144.003	1729	1730	1735	-0.006	0.028	0.028	0.033	0.032	0.03	-0.003	-0.003	-0.038	-0.038	0.0205	0.0095
145.002	1865	1866	1871	-0.006	0.03	0.03	0.036	0.035	0.0325	-0.004	-0.004	-0.04	-0.04	0.022	0.0105
146.003	2002	2003	2008	-0.006	0.032	0.032	0.038	0.037	0.0345	-0.006	-0.006	-0.043	-0.043	0.0245	0.01
147.003	2134	2135	2140	-0.006	0.033	0.033	0.041	0.04	0.0365	-0.007	-0.007	-0.045	-0.045	0.026	0.0105
148.003	2268	2269	2274	-0.006	0.035	0.035	0.043	0.042	0.0385	-0.009	-0.009	-0.048	-0.048	0.0285	0.01
149.003	2408	2409	2414	-0.006	0.037	0.037	0.046	0.045	0.041	-0.011	-0.011	-0.051	-0.051	0.031	0.01

Theoretical Rod Field $\Delta = P/\Delta E$	Theoretical Rod Testing $\Delta = P/\Delta E$	$\Delta_{\text{Calibration}} = \Delta_{\text{Field}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Red}}$ $\Delta_{\text{Calibration}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Blue}}$ $\Delta_{\text{Calibration}}$	Machine Rate of Loading
1.70641E-05	4.04911E-05	2.3427E-05	-2.3427E-05	0.00476573	0
0.000107889	0.000256008	0.000148119	0.001851881	0.001351881	0.141
0.000191008	0.000453239	0.000262231	0.004237769	0.002737769	0.191
0.000262017	0.000621735	0.000359718	0.006140282	0.003640282	0.183
0.000330824	0.000785005	0.000454182	0.008045818	0.004545818	0.179
0.000391374	0.000928683	0.00053731	0.01046269	0.00596269	0.123
0.000453575	0.00107628	0.000622705	0.012377795	0.006877795	0.149
0.000516878	0.001226489	0.000709612	0.014790388	0.008290388	0.156
0.000584033	0.001385841	0.000801808	0.016698192	0.009198192	0.159
0.000653941	0.001551724	0.000897783	0.019102217	0.010602217	0.479
0.000728849	0.001717607	0.000993758	0.021006342	0.011506342	0.213
0.000792656	0.001880878	0.001088222	0.022411778	0.012911778	0.192
0.000864765	0.002051985	0.00118722	0.02431278	0.01431278	0.184
0.000874123	0.002074919	0.001200667	0.027299933	0.016799933	0.18
0.00095504	0.002266196	0.001311157	0.028688943	0.019188943	0.125
0.001029902	0.002443835	0.001413933	0.031086067	0.020586067	0.2
0.001106314	0.00262278	0.001517465	0.032982535	0.022982535	0.187
0.001177974	0.002795193	0.001617219	0.034882781	0.024382781	0.182
0.001251735	0.002970219	0.001718484	0.036781516	0.026781516	0
0.001328799	0.003153083	0.001824283	0.039175717	0.029175717	0.143

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

Readings	Load	Zeroed Load after 1/2 Turn	Head	1 Red	Zeroed red 1	2 Red	Zeroed red 2	Average red	1 Blue	Zeroed blue 1	2 Blue	Zeroed blue 2	Average blue	Difference between Avg Red and Avg Blue
4.002	2	0	0	-0.001	0	0.001	0	0	0	0	0	0	0	0
5.002	86	84	0	0	0.001	0.001	0	0.0005	0	0	0	0	0	0.0005
6.002	466	464	0.001	0.002	0.003	0.003	0.002	0.0025	0	-0.001	0.0005	0.0005	0.00075	0.00175
7.003	833	831	0.004	0.003	0.004	0.005	0.004	0.004	0	-0.003	0.0015	0.0015	0.00225	0.00175
8.003	1177	1175	0.01	0.005	0.006	0.007	0.006	0.006	-0.001	-0.004	0.0025	0.0025	0.00325	0.00275
9.003	1516	1514	0.01	0.007	0.008	0.009	0.008	0.008	-0.001	-0.006	0.0035	0.0035	0.00475	0.00325
10.003	1834	1832	0.01	0.01	0.011	0.01	0.009	0.01	-0.002	-0.008	0.005	0.005	0.0065	0.0035
11.003	2155	2153	0.011	0.012	0.013	0.012	0.011	0.012	-0.002	-0.01	0.006	0.006	0.008	0.004
12.003	2488	2486	0.015	0.014	0.015	0.014	0.013	0.014	-0.002	-0.012	0.007	0.007	0.0095	0.0045
13.002	2749	2747	0.021	0.015	0.016	0.015	0.014	0.015	-0.003	-0.013	0.008	0.008	0.0105	0.0045
14.003	3099	3097	0.021	0.016	0.017	0.017	0.016	0.0165	-0.003	-0.015	0.009	0.009	0.012	0.0045
15.002	3446	3444	0.024	0.018	0.019	0.019	0.018	0.0185	-0.003	-0.017	0.01	0.01	0.0135	0.005
16.002	3788	3786	0.03	0.02	0.021	0.021	0.02	0.0205	-0.004	-0.018	0.011	0.011	0.0145	0.006
17.004	4133	4131	0.032	0.021	0.022	0.023	0.022	0.022	-0.004	-0.02	0.012	0.012	0.016	0.006
18.004	3945	3943	0.039	0.021	0.022	0.027	0.026	0.024	-0.006	-0.02	0.013	0.013	0.0165	0.0075
19.002	4279	4277	0.041	0.021	0.022	0.029	0.028	0.025	-0.007	-0.021	0.014	0.014	0.0175	0.0075
20.003	4588	4586	0.042	0.022	0.023	0.031	0.03	0.0265	-0.007	-0.023	0.015	0.015	0.019	0.0075
21.002	4888	4886	0.048	0.023	0.024	0.033	0.032	0.028	-0.007	-0.024	0.0155	0.0155	0.01975	0.00825
22.003	5191	5189	0.051	0.025	0.026	0.034	0.033	0.0295	-0.007	-0.026	0.0165	0.0165	0.02125	0.00825
23.003	5502	5500	0.052	0.026	0.027	0.034	0.033	0.03	-0.007	-0.027	0.017	0.017	0.022	0.008
24.003	5813	5811	0.052	0.027	0.028	0.034	0.033	0.0305	-0.007	-0.029	0.018	0.018	0.0235	0.007

Theoretical Rod Field $\Delta = \text{Pl/AE}$	Theoretical Rod Testing $\Delta = \text{Pl/AE}$	$\Delta_{\text{Calibration}} = \Delta_{\text{Testing}} - \Delta_{\text{Field}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Field}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Field}} - \Delta_{\text{Calibration}}$	Machine Rate of Loading
4.62382E-05	0.000112069	6.58307E-05	0.000434169	-6.58307E-05	0.13
0.000255411	0.000619048	0.000363636	0.002156364	0.00386364	0.146
0.000457428	0.001108682	0.000651254	0.003348746	0.001598746	0.178
0.000646785	0.001567631	0.000920846	0.005079154	0.002329154	0.175
0.000833389	0.00201991	0.00118652	0.00681348	0.00356348	0.215
0.001008434	0.002444171	0.001435737	0.008564263	0.005064263	0.235
0.00118513	0.002872434	0.001687304	0.010312696	0.006312696	0.21
0.001368432	0.0033316708	0.001948276	0.012051724	0.007551724	0.199
0.001512101	0.003664922	0.002152821	0.012847179	0.008347179	0.306
0.00170476	0.004131876	0.002427116	0.014072884	0.009572884	0.197
0.001895768	0.004594828	0.00269906	0.01580094	0.01080094	0.183
0.002084024	0.005051108	0.002967085	0.017532915	0.011532915	0.204
0.002273931	0.005511392	0.003237461	0.018762539	0.012762539	0.195
0.00247045	0.005960571	0.003490125	0.020909875	0.013409875	0.109
0.002654297	0.0064706178	0.003816319	0.021648119	0.014148119	0.189
0.002824388	0.006918432	0.004094044	0.022905956	0.01505956	0.181
0.002989525	0.007381678	0.004392154	0.024170846	0.015920846	0.177
0.003156313	0.007829227	0.004672914	0.025433386	0.016833386	0.316
0.003327504	0.008273849	0.004946345	0.026896555	0.017689655	0.198
0.003498696	0.008727771	0.005229075	0.028459525	0.018945925	0.184

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

10.3.8 H3.2**H3.2 11/4/2011 1:50**

Spreadsheet calculation

User Input

Average Rate of loading =	0.18	Inches/Minute
Test length :	2:42	Min:Sec
Head Movement:	0.496	Inch
Test Date :	11/4/2011	
Test Time :	1:50	pm
Average Moisture content =	8.4	%
Wood Temperature =	71	° F
Testing Rod Length =	19.06	Inch
Theoretical Rod Length =	7.375	Inch
1/2 turn load =	2200	lbs
Relaxed after 1/2 turn =	2075	lbs

7/8" diam. Rod properties

A =	0.462	in ²
E =	29000000	psi

Force vs Displacement for H3.2

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

Readings	Load	Zeroed Load after 1/2 Turn	Head	1 Red	Zeroed red 1	2 Red	Zeroed red 2	Average red	1 Blue	Zeroed blue 1	2 Blue	Zeroed blue 2	Average blue	Difference between Avg Red and Avg Blue
43.007	2	0	0	0.001	-0.001	0	0	0	0	0	0	0	0	0
44.105	21	19	0	0.001	-0.001	0	0	0	-0.001	0	0	0	0	0
45.203	283	281	0	0.001	-0.001	0	0	0	-0.002	-0.002	-0.001	-0.002	0.0015	-0.0015
46.079	584	582	0	0.002	-0.001	0.001	0	0.0005	-0.004	-0.004	-0.003	-0.004	0.0035	-0.003
47.176	808	806	0	0.004	-0.001	0.003	0	0.0015	-0.005	-0.005	-0.004	-0.005	0.0045	-0.003
48.054	1074	1072	0	0.005	-0.001	0.004	0	0.002	-0.006	-0.007	-0.006	-0.007	0.006	-0.004
49.15	1273	1271	0	0.007	-0.001	0.006	0	0.003	-0.007	-0.008	-0.006	-0.008	0.007	-0.004
50.029	1540	1538	0	0.008	-0.001	0.007	0	0.0035	-0.008	-0.01	-0.007	-0.01	0.0085	-0.005
51.127	1758	1756	0	0.01	-0.001	0.009	0	0.0045	-0.009	-0.012	-0.008	-0.012	0.01	-0.0055
52.004	2026	2024	0	0.012	-0.001	0.011	0	0.0055	-0.01	-0.014	-0.009	-0.014	0.0115	-0.006
53.1	2298	2296	0	0.014	-0.001	0.013	0	0.0065	-0.012	-0.016	-0.011	-0.016	0.0135	-0.007
54.198	2521	2519	0	0.016	-0.001	0.015	0	0.0075	-0.013	-0.018	-0.012	-0.018	0.015	-0.0075
55.077	2797	2795	0	0.018	-0.001	0.017	0	0.0085	-0.015	-0.02	-0.014	-0.02	0.017	-0.0085
56.172	3017	3015	0	0.02	-0.001	0.019	0	0.0095	-0.016	-0.022	-0.015	-0.022	0.0185	-0.009
57.05	3308	3306	0	0.021	-0.001	0.02	0	0.01	-0.018	-0.024	-0.017	-0.024	0.0205	-0.0105
58.148	3546	3544	0	0.023	-0.001	0.022	0	0.011	-0.019	-0.025	-0.018	-0.025	0.0215	-0.0105
59.025	3842	3840	0	0.024	-0.001	0.023	0	0.0115	-0.021	-0.027	-0.02	-0.027	0.0235	-0.012
60.123	4076	4074	0	0.026	-0.001	0.025	0	0.0125	-0.022	-0.028	-0.021	-0.028	0.0245	-0.013
61.001	4375	4373	0	0.027	-0.001	0.026	0	0.013	-0.023	-0.03	-0.022	-0.03	0.026	-0.014
62.099	4646	4644	0	0.029	-0.001	0.028	0	0.014	-0.025	-0.032	-0.024	-0.032	0.028	-0.0145
63.004	4885	4883	0	0.03	-0.001	0.029	0	0.0145	-0.026	-0.033	-0.025	-0.033	0.029	-0.0145

Theoretical Rod Field $\Delta = P/\Delta E$	Theoretical Rod Testing $\Delta = P/\Delta E$	$\Delta_{\text{Calibration}} = \Delta_{\text{Testing}} - \Delta_{\text{Field}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Red}} - \Delta_{\text{Calibration}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Blue}} - \Delta_{\text{Calibration}}$	Machine Rate of Loading
1.04587E-05	2.7033E-05	1.65743E-05	-1.65743E-05	1.65743E-05	0
0.000154678	0.000399803	0.000245125	-0.000245125	0.001254875	0
0.000320365	0.000828062	0.000507697	-7.69704E-06	0.002992303	0
0.000443667	0.001146766	0.000703099	0.000796901	0.003796901	0
0.000590088	0.001525228	0.00093514	0.00106486	0.00506486	0
0.000699629	0.001808362	0.001108734	0.001891266	0.005891266	0
0.0008466	0.002188246	0.001341646	0.002158354	0.007158354	0
0.000966599	0.002498414	0.001531814	0.002968186	0.008468186	0
0.001114122	0.002879721	0.001765599	0.003734401	0.009734401	0
0.001386597	0.003584	0.002197404	0.005302596	0.012802596	0
0.001538523	0.00397669	0.002438167	0.006061833	0.014561833	0
0.001659623	0.004289703	0.00263008	0.00686992	0.01586992	0
0.001819805	0.004703734	0.002883929	0.007116071	0.017616071	0
0.001950814	0.005042357	0.003091543	0.007908456	0.018408456	0
0.002113748	0.005463502	0.003349754	0.008150246	0.020150246	0
0.002242555	0.005796434	0.003553879	0.008946121	0.020946121	0
0.002407141	0.006221847	0.003814706	0.009185294	0.022185294	0
0.002556314	0.006607423	0.004051108	0.009488892	0.023948892	0
0.002687873	0.006947469	0.004259596	0.010240404	0.024740404	0

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

Readings	Load	Zeroed Load after 1/2 Turn	Head	1 Red	Zeroed red 1	2 Red	Zeroed red 2	Average red	1 Blue	Zeroed blue 1	2 Blue	Zeroed blue 2	Average blue	Difference between Avg Red and Avg Blue
56.004	-11	0	0.004	0.001	0.001	0	0	0	0	0	0	0	0	0
57.002	180	191	0.003	0.001	0.001	0	0	0	0	0	0	0	0	0
58.002	532	543	0.004	0.003	0.001	0.002	0	0.001	0	-0.001	0	-0.001	0.0005	0.0005
59.002	866	877	0.004	0.003	0.001	0.002	0	0	-0.002	-0.001	-0.001	-0.001	0.0015	-0.0005
60.003	1188	1199	0.004	0.004	0.001	0.003	0	0.0015	-0.004	-0.002	-0.004	-0.002	0.003	-0.0015
61.003	1515	1526	0.003	0.005	0.001	0.004	0	0.002	-0.006	-0.003	-0.003	-0.003	0.0045	-0.0025
62.002	1850	1861	0.002	0.005	0.001	0.004	0	0.002	-0.008	-0.004	-0.008	-0.004	0.006	-0.004
63.003	2186	2197	0.002	0.006	0.001	0.005	0	0.0025	-0.009	-0.005	-0.005	-0.005	0.007	-0.0045
64.003	2531	2542	0.003	0.007	0.001	0.006	0	0.003	-0.011	-0.006	-0.011	-0.006	0.0085	-0.0055
65.002	2871	2882	0.003	0.008	0.001	0.007	0	0.0035	-0.013	-0.007	-0.013	-0.007	0.01	-0.0065
66.003	3215	3226	0.003	0.009	0.001	0.008	0	0.004	-0.016	-0.007	-0.016	-0.007	0.0115	-0.0075
67.003	3557	3568	0.003	0.01	0.001	0.009	0	0.0045	-0.018	-0.008	-0.018	-0.008	0.013	-0.0085
68.002	3874	3885	0.004	0.011	0.001	0.01	0	0.005	-0.019	-0.009	-0.019	-0.009	0.014	-0.009
69.003	4198	4209	0.004	0.012	0.001	0.011	0	0.0055	-0.021	-0.009	-0.021	-0.009	0.015	-0.0095
70.002	4517	4528	0.004	0.012	0.001	0.011	0	0.0055	-0.023	-0.01	-0.023	-0.01	0.0165	-0.011
71.003	4839	4850	0.003	0.013	0.001	0.012	0	0.006	-0.025	-0.01	-0.025	-0.01	0.0175	-0.0115
72.002	5159	5170	0.003	0.014	0.001	0.013	0	0.0065	-0.027	-0.011	-0.027	-0.011	0.019	-0.0125
73.003	5462	5473	0.003	0.015	0.001	0.014	0	0.007	-0.029	-0.011	-0.029	-0.011	0.02	-0.013
74.003	5772	5783	0.002	0.015	0.001	0.014	0	0.007	-0.03	-0.011	-0.03	-0.011	0.0205	-0.0135
75.002	6078	6089	0.003	0.016	0.001	0.015	0	0.0075	-0.032	-0.012	-0.032	-0.012	0.022	-0.0145
76.002	6363	6374	0.004	0.017	0.001	0.016	0	0.008	-0.034	-0.012	-0.034	-0.012	0.023	-0.015

Theoretical Rod Field $\Delta = PL/AE$	Theoretical Rod Testing $\Delta = PL/AE$	$\Delta_{\text{Calibration}} = \frac{\Delta_{\text{Actual}} - \Delta_{\text{Theoretical}}}{\Delta_{\text{Theoretical}}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Red}}$	$\Delta_{\text{Actual}} = \Delta_{\text{Blue}}$	Machine Rate of Loading
0.000105137	0.000258387	0.00015325	-0.00015325	-0.00015325	0.03
0.000298897	0.000734578	0.000435681	0.000564319	6.43193E-05	0.067
0.000482749	0.001186418	0.000703668	0.000296332	0.000796332	0.353
0.000659996	0.001622024	0.000962028	0.000537972	0.002037972	0.202
0.000839995	0.002064394	0.001224399	0.000775601	0.003275601	0.186
0.001024397	0.002517587	0.001493189	0.000506811	0.004506811	0.206
0.00120935	0.002972132	0.001762782	0.000737218	0.005237218	0.197
0.00139257	0.003438853	0.002039595	0.000964005	0.006464005	0.114
0.001586412	0.00389881	0.002312397	0.001187603	0.007687603	0.192
0.00175769	0.004364177	0.002588409	0.001411591	0.008911591	0.183
0.001964024	0.00482684	0.002862815	0.001637185	0.010137185	0.179
0.002138519	0.005255682	0.003117163	0.001882837	0.01162837	0.365
0.002316866	0.005693994	0.003377127	0.002122873	0.01322873	0.203
0.002492462	0.006125541	0.00363308	0.002351866	0.01481866	0.187
0.002669708	0.006561147	0.003891439	0.00258561	0.01640561	0.206
0.002845854	0.006994048	0.004148194	0.00281806	0.018006	0.197
0.003012642	0.00740395	0.004391308	0.003048692	0.019608692	0.115
0.003183283	0.007823323	0.00464004	0.003285996	0.021285996	0.193
0.003351722	0.008237284	0.004885561	0.003523784	0.023523784	0.183
0.003508602	0.008622835	0.005114233	0.003768561	0.025768561	0.179

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

Readings	Load	Zeroed Load after Jaw Attachment	Head	Overall Deflection		Plywood Deflection		Strap Deflection	% Deflection of Strap	% Deflection of Wood	% Check	Rate of Loading
				1 Red	Zeroed Red 1	1 Blue	Zeroed blue 1	$\Delta_{Strap} = \Delta_{Overall} - \Delta_{Plywood}$				
33.003	5	0	-0.007	0.001	0.001	0	0	0.001	100	0	100	0.184
34.002	50	45	-0.007	0	0	0	0	0	0	0	0	0.179
35.002	63	58	-0.006	0	0	0	0	0	0	0	0	0.176
36.002	89	84	-0.006	0	0	0	0	0	0	0	0	0.11
37.003	128	123	-0.007	0	0	0	0	0	0	0	0	0.19
38.002	195	190	-0.007	0	0	0	0	0	0	0	0	0.181
39.002	257	252	-0.006	0	0	0	0	0	0	0	0	0.177
40.003	322	317	-0.007	0	0	-0.001	0.001	0.001	0	0	0	0
41.003	381	376	-0.007	0	0	-0.001	0.001	0.001	0	0	0	0.198
42.011	447	442	-0.007	0.001	0.001	-0.001	0.001	0	0	100	100	0.185
43.003	513	508	-0.006	0.001	0.001	-0.001	0.001	0	0	100	100	0.179
44.002	591	586	-0.007	0.001	0.001	-0.001	0.001	0	0	100	100	0.176
45.002	673	668	-0.006	0.001	0.001	-0.001	0.001	0	0	100	100	0.223
46.002	756	751	-0.006	0.001	0.001	-0.001	0.001	0	0	100	100	0.191
47.002	847	842	-0.007	0.001	0.001	-0.001	0.001	0	0	100	100	0.182
48.003	937	932	-0.007	0.002	0.002	-0.001	0.001	0.001	50	50	100	0.178
49.002	1037	1032	-0.006	0.002	0.002	-0.001	0.001	0.001	50	50	100	0
50.002	1133	1128	-0.007	0.002	0.002	-0.001	0.001	0.001	50	50	100	0.201
51.002	1228	1223	-0.007	0.002	0.002	-0.001	0.001	0.001	50	50	100	0.186
52.003	1330	1325	-0.006	0.002	0.002	-0.001	0.001	0.001	50	50	100	0.18
53.002	1432	1427	-0.006	0.003	0.003	-0.001	0.001	0.002	67	33	100	0.177

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

10.3.11 S1.2**S1.2 1/10/12 10:30 am**

		Average Rate of loading =	0.17	Inches/Minute		
		Test length :	3:36	Min:Sec		
		Test Date :	1/10/2012			
		Test Time :	10:30	am		
		Average Moisture content =	9.8	% (Test)	11.95	% (Assembly)
		Wood Temperature =	58	° F		
		Head movement=	0.68	Inch		
		Calculated Average rate of loading=	0.19	Inches/minute		
		2:45 Load starts to drop				
		3:22 Strap starts to split (net shear)				
		3:36 Stop test

Readings	Load	Zeroed Load after Jaw Attachment	Head	Overall Deflection		Plywood Deflection		Strap Deflection $\Delta_{Strap} = \Delta_{Overall} - \Delta_{Plywood}$	% Deflection of Strap	% Deflection of Wood	% Check	Rate of Loading
				1 Red	Zeroed Red 1	1 Blue	Zeroed blue 1					
23.002	-17	0	-0.006	0.002	0	0	0	0	0	0	0	0.198
24.003	33	50	-0.006	0.003	0.001	0	0	0.001	100	0	100	0.184
25.003	78	95	-0.005	0.003	0.001	0	0	0.001	100	0	100	0.204
26.004	135	152	-0.007	0.003	0.001	0	0	0.001	100	0	100	0.195
27.004	197	214	-0.005	0.003	0.001	0	0	0.001	100	0	100	0.111
28.003	258	275	-0.004	0.003	0.001	0	0	0.001	100	0	100	0.189
29.003	323	340	-0.006	0.003	0.001	0	0	0.001	100	0	100	0.181
30.002	385	402	-0.006	0.003	0.001	0	0	0.001	100	0	100	0.177
31.003	447	464	-0.005	0.003	0.001	0	0	0.001	100	0	100	0.326
32.003	512	529	-0.007	0.004	0.002	0	0	0.002	100	0	100	0.199
33.003	578	595	-0.005	0.004	0.002	0	0	0.002	100	0	100	0.185
34.003	648	665	-0.006	0.005	0.003	-0.001	0.001	0.002	67	33	100	0.179
35.002	721	738	-0.005	0.005	0.003	-0.001	0.001	0.002	67	33	100	0.196
36.002	800	817	-0.007	0.004	0.002	-0.001	0.001	0.001	50	50	100	0.222
37.002	881	898	-0.006	0.005	0.003	-0.001	0.001	0.002	67	33	100	0.19
38.003	967	984	-0.006	0.005	0.003	-0.001	0.001	0.002	67	33	100	0.212
39.002	1056	1073	-0.007	0.005	0.003	-0.001	0.001	0.002	67	33	100	0.2
40.002	1151	1168	-0.006	0.005	0.003	-0.001	0.001	0.002	67	33	100	0
41.003	1241	1258	-0.006	0.006	0.004	-0.001	0.001	0.003	75	25	100	0.2
42.003	1329	1346	-0.006	0.007	0.005	-0.001	0.001	0.004	80	20	100	0.185
43.003	1394	1411	-0.007	0.007	0.005	-0.001	0.001	0.004	80	20	100	0.179

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior

Readings	Load	Zeroed Load after Jaw Attachment	Head	Overall Deflection		Plywood Deflection		Strap Deflection	% Deflection of Strap	% Deflection of Wood	% Check	Rate of Loading
				1 Red	Zeroed Red 1	1 Blue	Zeroed blue 1	$\Delta_{\text{strap}} = \Delta_{\text{overall}} - \Delta_{\text{plywood}}$				
24.002	-32	0	-0.005	0.002	0	0	0	0	0	0	0	0.184
25.003	22	54	-0.007	0.002	0	0	0	0	0	0	0	0.204
26.002	99	131	-0.006	0.002	0	0	0	0	0	0	0	0.195
27.002	160	192	-0.006	0.002	0	0	0	0	0	0	0	0.219
28.002	220	252	-0.006	0.002	0	0	0	0	0	0	0	0.189
29.003	293	325	-0.006	0.003	0.001	0	0	0.001	100	0	100	0.211
30.002	353	385	-0.007	0.003	0.001	0	0	0.001	100	0	100	0.199
31.002	403	435	-0.006	0.003	0.001	0	0	0.001	100	0	100	0.324
32.003	462	494	-0.006	0.003	0.001	-0.001	0.001	0	0	100	100	0.199
33.003	520	552	-0.005	0.003	0.001	-0.001	0.001	0	0	100	100	0.185
34.002	589	621	-0.005	0.003	0.001	-0.001	0.001	0	0	100	100	0.205
35.002	654	686	-0.006	0.003	0.001	-0.001	0.001	0	0	100	100	0.196
36.002	730	762	-0.006	0.004	0.002	-0.001	0.001	0.001	50	50	100	0.221
37.003	804	836	-0.007	0.004	0.002	-0.001	0.001	0.001	50	50	100	0.19
38.002	878	910	-0.007	0.004	0.002	-0.001	0.001	0.001	50	50	100	0.212
39.003	953	985	-0.006	0.004	0.002	-0.001	0.001	0.001	50	50	100	0.2
40.002	1032	1064	-0.007	0.004	0.002	-0.001	0.001	0.001	50	50	100	0.331
41.006	1102	1134	-0.007	0.005	0.003	-0.001	0.001	0.002	67	33	100	0.2
42.002	1132	1164	-0.006	0.005	0.003	-0.001	0.001	0.002	67	33	100	0.185
43.002	1187	1219	-0.005	0.005	0.003	-0.001	0.001	0.002	67	33	100	0.205
44.002	1251	1283	-0.005	0.005	0.003	-0.001	0.001	0.002	67	33	100	0.196

Investigation of how the stiffness of continuity tie connections in wood diaphragms impacts collective chord behavior