LOAD DURATION AND SEASONING EFFECTS ON MORTISE AND TENON JOINTS

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A Report on Research Sponsored by

USDA NRI/CGP
Washington, DC

Timber Frame Business Council
Hanover, NH

Timber Framers Guild
Becket, MA

August 2000

UNIVERSITY OF
WYOMING

REPORT DOCUMENTATION PAGE

1. REPORT NO.

2.

3. Recipient's Accession No.

4. Title and Subtitle

Load Duration and Seasoning Effects on Mortise and Tenon Joints

5. Report Date

August 2000

6.

7. Author(s)

Richard J. Schmidt & Garth F. Scholl

8. Performing Organization Report No.

9. Performing Organization Name and Address

Department of Civil and Architectural Engineering
University of Wyoming
Laramie, WY 82071

10. Project/Task/Work Unit No.

11. Contract(C) or Grant(G) No.

(C)
(G)

12. Sponsoring Organization Name and Address

USDA NRI/CGP	Timber Frame Business Council	Timber Framers Guild
CSREES	PO Box B1161	PO Box 60
Wash., DC 20250	Hanover, NH 03755	Becket, MA 01223

13. Type of Report & Period Covered

interim

14.

15. Supplementary Notes

USDA NRI/CGP Contract No. 97-35103-5053

16. Abstract (Limit: 200 words)

The objective of this research is to determine the load duration and seasoning effects on mortise and tenon joints in tension. Design of mortise and tenon joints is currently beyond the scope of the National Design Specification for Wood Construction. This and previous research have been conducted to find minimum detailing requirements for joints of this type. Load duration research served a dual purpose in verifying the previously established detailing requirements and finding the load duration and seasoning effects on mortise and tenon joints. In order to determine these effects, load duration tests on full size mortise and tenon joint specimens were conducted. Drawboring and peg diameter effects were also analyzed in the long-term load study. Strength tests were performed at the conclusion of long-term testing to find the resulting effects due to long-term loading. A method of analyzing combined dowel bearing material properties of the base material and pegs was also studied.

17. Document

a. Descriptors

traditional timber framing, heavy timber construction, wood peg fasteners, mortise and tenon connections, joint testing, duration of load, seasoning effects, drawboring.

b. Identifiers/Open-Ended

c. COSATI Field/Group

18. Availability Statement

Release unlimited.

19. Security Class (This Report)

Unclassified.

21. No. of Pages

111

Acknowledgments

This report is based on the research conducted by Mr. Garth F. Scholl, under the direction of Dr. Richard J. Schmidt, in partial fulfillment of the requirements for a Masters of Science Degree in Civil Engineering at the University of Wyoming. Primary funding for this research was provided by the USDA-NRI/CGP under contract #9702896. Additional funding was provided by the Timber Frame Business Council and the Timber Framers Guild. Joint specimens were donated by Big Timberworks, Red Suspenders Timber Frames, Benson Woodworking, and Riverbend Timber Framing. Northcott Wood Turning supplied the pegs.

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1. Introduction

1.1. Timber Frame Introduction/History

Timber frames, consisting of heavy timber members with carpentry-style joinery, played an integral part in construction for centuries, providing strong and durable frames for structures of all kinds. Traditional timber framing utilizes several different types of joints for different connection needs. Tension connections often use a mortise and tenon joint (Figure 1-1); these joints use a wooden peg to fasten the tenon inside of the mortise.

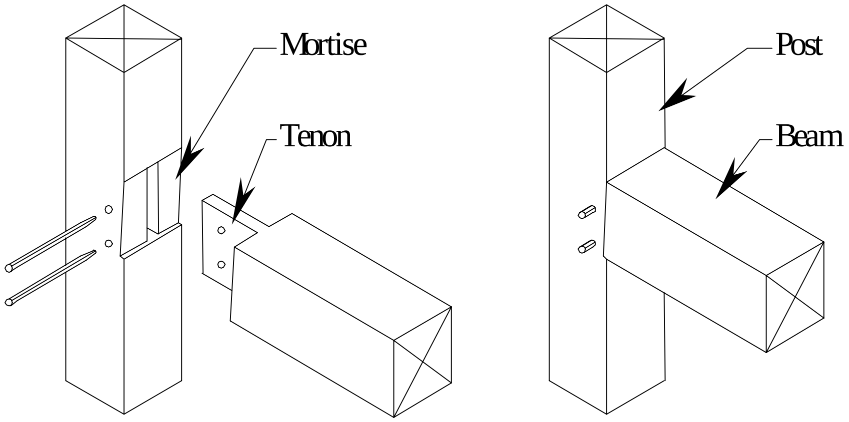


Figure 1-1 Mortise and Tenon Joint from Schmidt and Daniels (1999)

Increased production rates of saw mills and the ability to construct stick-frame structures in a short period of time lead to a shift in building methods away from of timber framing in the 19th century. In recent decades however, timber framing has experienced a revival. With the revival in timber framing, new methods of enclosing the frame have been developed. Prefabricated panels can span between bays of the timber frame to provide a well insulated enclosure system. This development along with the rugged traditional style has helped lead to an ever increasing number of newly built and restored traditional timber framed structures.

1.2. Purpose/Need of Research

In the past traditional timber frame joinery detailing was based on the craftsman's experience. Currently specifications and detailing requirements for traditional timber frame joinery are not included in the National Design Specification (NDS) (AFPA, 1997) or in any other recognized code or design standard. Therefore values for strength and stiffness of these joints are often not known. This produces a need for design equations and specifications that can be used to obtain the strength and stiffness of a mortise and tenon joint.

Tension strength of these joints is of primary interest, because it relies on the ability of the wood peg fasteners to carry the load. Tension can be developed in mortise and tenon joints under both gravity and lateral loads. For instance, under gravity loads on floor girders, knee braces carry compression, producing a lateral thrust on the posts. This thrust is resisted by a tension connection between the girder and the post.

The lateral load resistance of many timber-framed structures originates from a knee brace design. Knee braces are commonly seen in pairs. Under lateral load one knee brace is in compression while the other is in tension. Examples of typical bents are shown below in Figure 1-2.

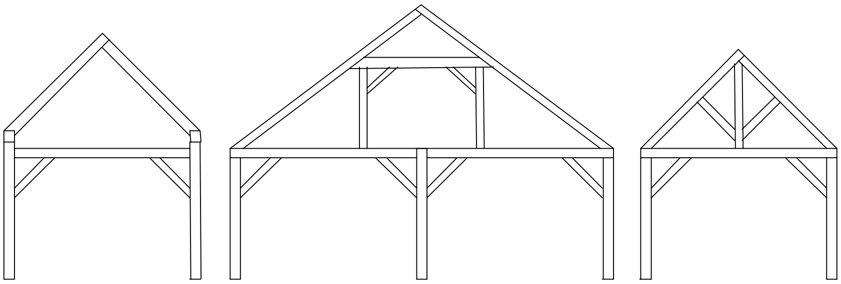


Figure 1-2 Typical Bent Types from Schmidt and Daniels (1999)

Often a timber frame designer has to over design a compression knee brace because of the uncertainty in strength and stiffness of a knee brace in tension. The compression joint is over designed because the knee brace in tension is assumed have zero tensile capacity. The majority of timber frame knee brace connections are mortise and tenon joints. A set of design standards would allow a timber frame designer to let the tension brace carry a portion of the lateral load.

Load duration and seasoning effects are also of concern when designing a timber frame joint. Timber frames are frequently cut and assembled while timbers are still green. In most cases cost and schedule constraints limit the amount of time that timbers can be seasoned prior to cutting for a frame. This results in frames with high initial moisture content. Long term effects on joint strength and stiffness are of concern particularly when analyzing or designing for serviceability. These long-term effects on traditional timber frame joinery are also beyond the scope of current design specifications. This research addresses and considers the effects of load duration on strength, stiffness and detailing requirements of mortise and tenon joints

1.3. Literature Review

Previous research concerning mortise and tenon joint strength and stiffness included joint tests by Schmidt and Daniels (1999) who performed full-scale tests on mortise and tenon joints of several different species of wood. Schmidt and Daniels (1999) tested several green or partially seasoned joints to determine minimum end, edge and spacing distances in order to ensure a ductile peg failure of the joint. The minimum detailing requirements are then used along with the European Yield Model equations adapted by Schmidt and MacKay (1997) and Schmidt and Daniels (1999) to find a joint strength.

Work at Michigan Technological University (Reid, 1997; Sandberg *et al*, 2000) with simplified mortise and tenon joints has also shown be of value in modeling, testing and defining strength and stiffness of mortise and tenon joints. This work with simplified mortise and tenon joints incorporated a single peg with three separate pieces of sawn lumber making up the rest of the joint, a single main member, representing the tenon, and the mortise consisting of two side members.

Duration of load effects are included in design of timber members through an adjustment factor based on the Madison curve (Figure 1-3). This relationship between load duration and member strength was developed by research at the Forest Products Laboratory (Breyer *et al*, 1999) using small clear specimens in bending. Nevertheless, the time effects are assumed to apply to connection strength as well.

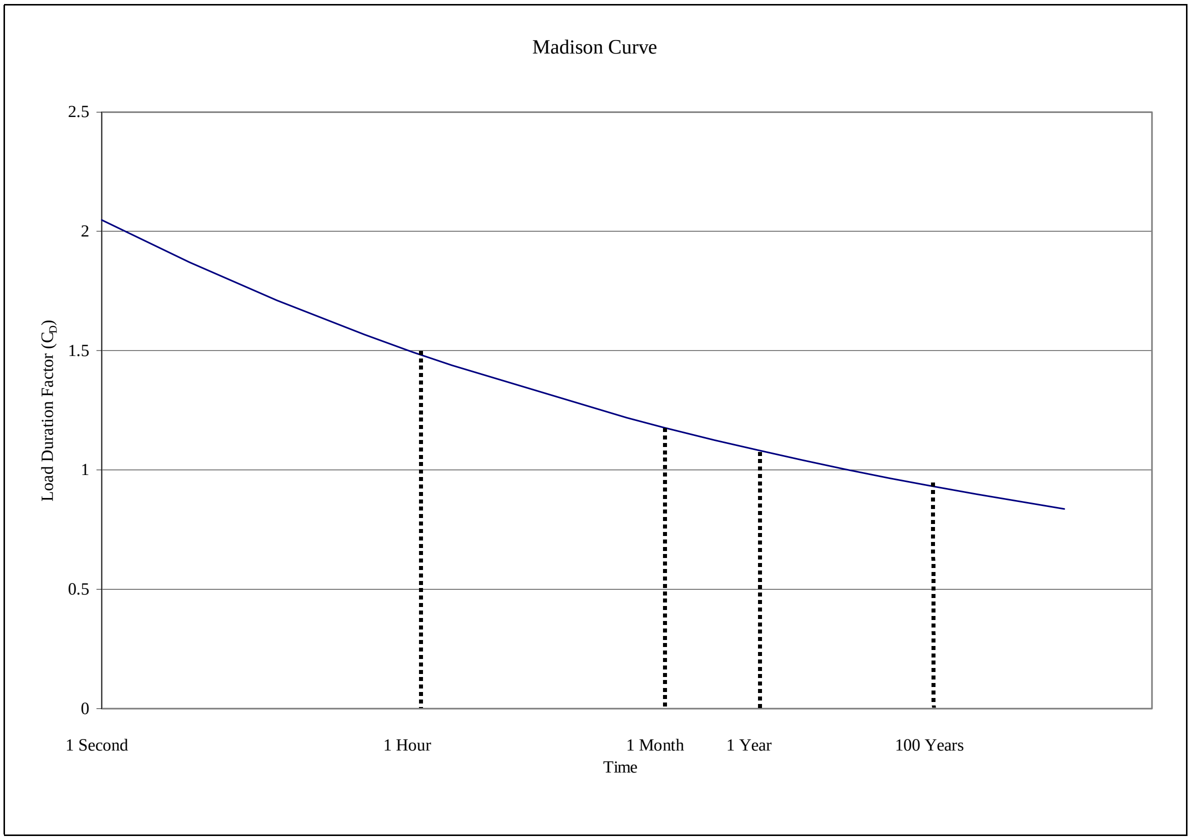


Figure 1-3 Madison Curve

Research relevant to load duration and seasoning of mortise and tenon joinery is limited. Researchers at the Forest Products Laboratory (Wilkinson, 1988) investigated effects of load duration on bolted connections. Sixty-four Douglas fir joints were evaluated; a ½ inch diameter steel bolt, hand tight, was used to secure the three pieces together. Each piece was loaded parallel to grain with an end distance of four inches. The center member was three inches wide and the two side members were each 1-1/2 inches wide. The sixty-four joints were divided into four groups, consisting of sixteen joints per group. The first group, the control group, was subjected to only short-term ramp load to failure with a constant rate of deflection. The second, third and fourth groups were each subjected to a constant load for one year at 85%, 60%, and 30% of the short term mean ultimate load. A few of the joints failed during the year of constant load.

However these failures were away from the joint area and not related to the joint itself. The joints were then tested to failure in a similar fashion as the first group. Each of the three groups subjected to the long-term load produced a higher mean load than the control group. The group that was loaded to 30% of the short-term load had the highest average maximum load of the three loaded groups followed by the 85% and the 60% groups respectively. The reason for this strength increase is not known or understood. The creep rate of the joints was also monitored; the 30% and 60% groups approached a zero creep rate while creep in the 85% group decreased in rate, but creep was still occurring after one year (Wilkinson, 1988).

More recently, research has involved effects of load rate (Rosowsky and Reinhold, 1999) and short-term duration of load (Fridley and Rosowsky, 1998) on wood connections. In the former study, nailed and screwed connection specimens were loaded at a rate from 0.1 to 1000 in/min. These tests revealed no obvious effects of load rate on either lateral load or withdrawal resistance of the test specimens. In the latter study, nailed connections were loaded to15, 20, and 30% of their average strength for 25 days to study creep response, and other specimens were loaded to 80, 90, and 95% of average static strength for 60 days to study effects on strength. Repeated loading at the latter high load levels was performed to study cyclic load effects. The creep and constant load specimens showed no ill effects of their load histories, whereas the cyclic load specimens did show reduced residual strength.

No research on the seasoning of mortise and tenon joints under load has been found. Often timber frame structures are constructed with green timber and dried while in service conditions. Therein lies the motivation for this research.

1.4. Objectives and Scope

Three primary objectives exist for this research. The first is to determine effects of seasoning and load duration on traditional mortise and tenon joints under tension. To the extent possible, load duration effects are separated from seasoning effects and each is analyzed.

The second objective is to continue the work of Schmidt and Daniels (1999). This research will continue to develop end, edge and spacing distances for different species of wood. This phase of research will also serve in further development and validation of a method in which dowel bearing strength and stiffness of a base material loaded with a wood peg fastener can be predicted mathematically. The advantages of mathematically predicting strength and stiffness could be of great value to future research by eliminating the need to perform combined material tests.

The third objective is to use results from the long-term joint tests to confirm or reassess detailing procedures for design of mortise and tenon joints. If appropriate a load duration factor could then be defined for use in connection design to adjust for load duration effects on strength.

The scope of the long-term research is inclusive of four different species of wood: southern yellow pine, Douglas fir, white oak, and eastern white pine. During the long-term load study, loading ranged from no load on specimens in the control groups to sustained load of 1000 lb or 2000 lb on the remaining specimens. The magnitude of the long-term load is dependent upon the short-term strength of the joints.

1.5. Overview

Primary among the three objectives given above is to determine the effects of long-term loading and seasoning on mortise and tenon joints in tension. In order to achieve this objective, tests and monitoring of mortise and tenon joints were required. However, the first tests that were conducted involved short-term joint tests on eastern white pine joints; these tests were a continuation of the research conducted by Schmidt and Daniels (1999). These tests were needed to determine the minimum detailing requirements of the eastern white pine joints that were used in long-term tests.

Following the short-term tests; joints of four different species were assembled. For each species, the joints were divided into a load group and a control group. The control group was not loaded and served as a basis for comparison in later strength testing. Each of the remaining joints was subjected to a sustained load of 1000 lb or 2000 lb for a period of up to 348 days. Moisture content was monitored in only the control group. Effects of drawboring and peg diameter were also compared using the time-deflection plots produced from the long-term tests.

Following the long-term tests, short-term load tests to failure were performed on all the joints. The yield values and stiffness of the loaded and unloaded groups were then compared. Additional factors such as peg diameter and effects of drawboring will also be analyzed. With the load duration tests completed, minimum detailing requrements were then revisited with the load duration tests completed and adjustments were made if needed.

As a secondary objective a method of mathematically combining dowel bearing strength and stiffness was tested and verified. The material for this group of tests came from the short-term eastern white pine joint tests. Base material was tested both parallel and perpendicular to grain..

In the next chapter, short-term tests of eastern white pine joints are described. These tests were performed to establish target strength values and detailing requirements for the joints used in the long-term study. Chapter 3 describes the method for determining the dowel bearing strength of wood with nonmetalic (in this case, wood) fasteners. The time-dependent behavior of pegged mortise and tenon joints under long-term load is presented in Chapter 4, and Chapter 5 contains the results of failure testing of the specimens subjected to long-term load. Analysis of the test results, plus a summary and conclusions are presented in Chapter 6.

2. Joint Tests (Eastern White Pine)

2.1. Introduction

Schmidt and Daniels (1999) reported joint detailing requirements along with tension test results for three different species of wood. The reported results were from full-scale tests on southern yellow pine, recycled Douglas fir and red oak joints. In a continuation of this work, tests of a similar nature were performed on eastern white pine joints. Detailing requirement are composed of end (l_e), edge (l_v) and spacing (l_s) distances. These distances are illustrated in Figure 2-1 below.

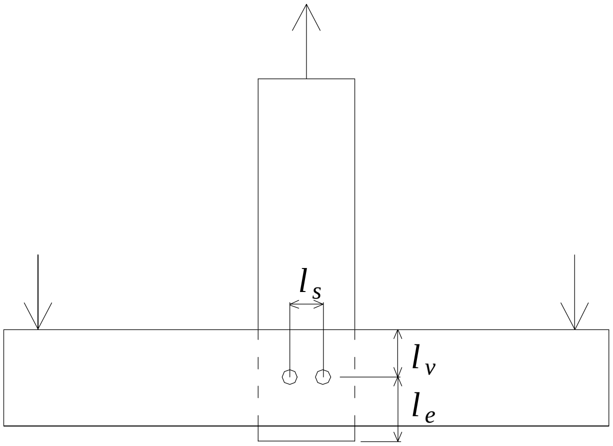


Figure 2-1 Detailing Distances from Schmidt and Daniels (1999)

Yielding of the peg is the preferred mode of joint failure. There are two primary reasons for this. First, peg yielding leads to a ductile failure of the joint under tension loading. The second reason is that the joint can be repaired by replacing the failed pegs with new ones. This mode of failure also helps to isolate the peg as the primary design criterion of the joint. Alternate joint failure modes include mortise splitting and tenon rupture. Bearing failure of the peg, mortise or tenon could also control the joint design, but such bearing failures have not been observed.

2.2. Test Frame Set-up

In order to find the minimum end, edge and spacing requirements, full-scale joint tests were performed on mortise and tenon joints constructed from eastern white pine. The test frame was the same as was used in previous research (Schmidt and MacKay, 1997). The test set up consists of an “A” frame with an Enerpac RCH 123 hydraulic ram, which applies a tensile force to the tenon member; see Figure 2-2. The base of the frame restrains motion of the mortise piece. Two 2” linear potentiometers record joint displacement. The potentiometers are attached to the tenon member with the tip resting on the mortise member. Labview data acquisition software was used to record and average the two potentiometer readings. Readings from a pressure transducer were recorded and combined with the potentiometer readings to plot load verses deflection. The load-deflection plot was used during the test to determine when the joint was yielding and when the test could be stopped.

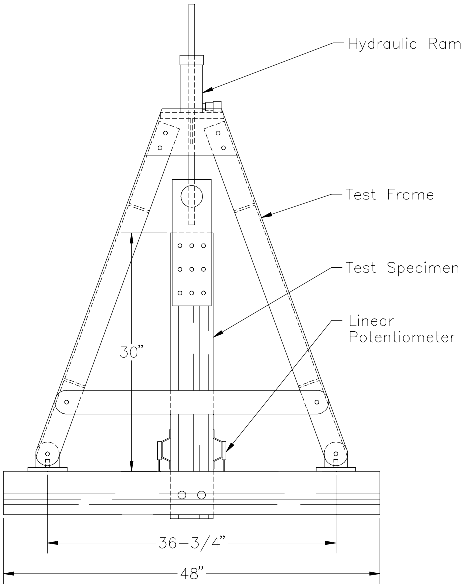


Figure 2-2 Short Term Test Set-up from Schmidt and MacKay (1997)

2.3. Short Term Test Procedure

The short term monotonic test procedure was modeled after research conducted by Schmidt and Daniels (1999). Timber frame members for each joint were randomly selected and checked for defects. The joint was lightly clamped together to assure a secure fit. Two peg holes were then drilled at a location that was thought to the minimum end and edge distance required to achieve peg failure. Two pegs were randomly selected out of the same population used by Schmidt and Daniels for their joint tests. The pegs were oriented tangentially, with growth rings in the same direction as applied force. The pegs were then driven with a mallet until secure.

The joint was placed into the test frame and the two linear potentiometers were fastened to the tenon with wood screws. A troubleshooting Labview data acquisition program was run to check for data acquisition errors. If no errors were detected, the program used for testing was started. Start time then was recorded and loading began.

Pressure was applied to the hydraulic ram by way of a hand pump. A constant rate of deflection was maintained through the test. A deflection rate of 0.001 inches per second was used. The test was continued until the load deflection plot had clearly flattened or started to decline and a yield value using the 5% offset method could be established. The 5% offset method of analysis will be discussed later in this chapter. After the joint yielded and had shown signs of failure, it was removed from the test frame. The pegs were then driven out and the joint was inspected. Observations about the test and corresponding failure were then recorded.

Dowel bearing tests followed the short-term joint tests. Two dowel bearing test samples were cut from each mortise member and two from each tenon member. Test results were recorded and moisture content and specific gravity tests were also performed on the test samples.

2.4. Failure Modes

Joint failure is the result of failure in one or more of the three joint components. The mortise member can split due to tension perpendicular to the grain (Figure 2-3). The split usually propagates from the peg holes and grows away from the joint parallel to the mortise member. This type failure of often occurs suddenly and without warning. It is a result of inadequate edge distance on the loaded edge of the member.

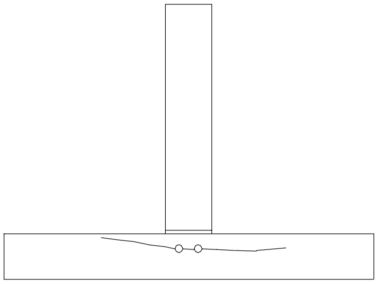


Figure 2-3 Typical Mortise Member Failure from Schmidt and Daniels (1999)

The tenon can fail (Figure 2-4); tenon failure is also referred to as a relish failure. The portion of the tenon behind the peg holes can develop a single split, or a condition of block shear failure is also common. Providing adequate end distance on the tenon can control this failure mode.

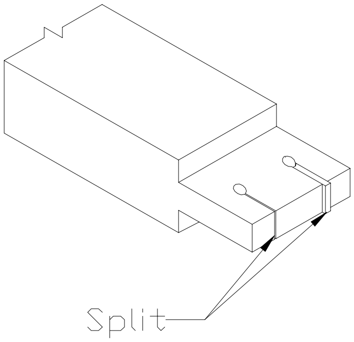


Figure 2-4 Typical Tenon Member Failure from Schmidt and Daniels (1999)

Peg failure results in the most ductile failure mode. Typically two transverse failure planes form at the mortise-tenon interfaces as in Figure 2-5. The failure planes are formed from a combination of shear and bending stress. Peg failure of another type is also possible. A single plastic hinge can develop in the center of the tenon, shown in

Figure 2-6. This type failure can develop in some connections with relatively large diameter pegs and thin tenons. Failure of this type is common with base material of low dowel bearing strength.

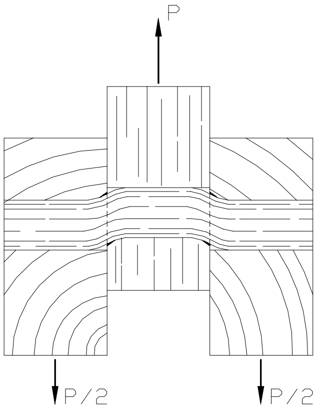


Figure 2-5 Peg Shear Bending Failure from Schmidt and Daniels (1999)

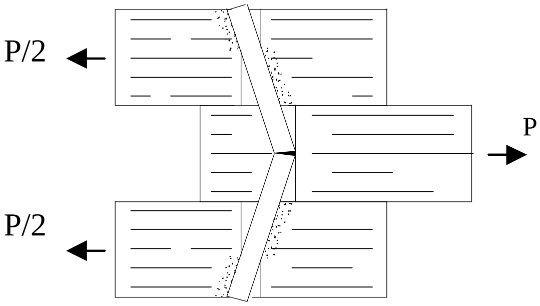


Figure 2-6 Peg Bending Failure Mode

2.5. Analysis Methods (5% offset)

A 5% offset method (ASTM D5764)(ASTM, 1999) was used to determine yield values in this research. The first step in this analysis method is to identify the initial linear portion of the load deflection plot. The 5% offset method then uses an intercept line that is parallel to the linear portion of the load deflection plot. This intercept line is offset horizontally a distance of 5% of the peg diameter of the test in question. The intersection of the load deflection line and the 5% offset intercept line is then taken as the yield value. If a higher value for load is observed before the intercept, then that higher value will become the yield value. Figure 2-7 shows a typical load deflection curve and the yield value found from that curve using the 5% offset method for determining yield value. A spreadsheet program was created and used to automate this process for this research.

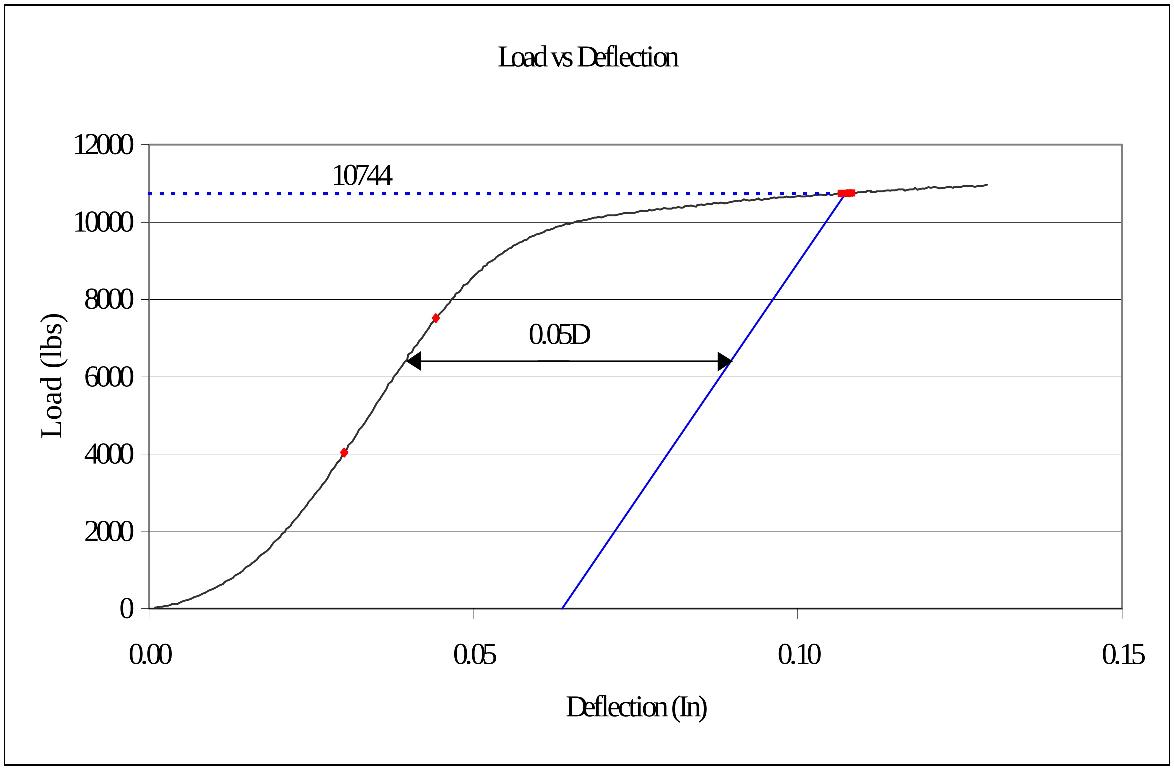


Figure 2-7 5% Offset Yield Value Example

2.6. Results

Nine eastern white pine joints were fabricated and tested with white oak pegs. Bensen Woodworking of Alstead Center New Hampshire donated the joints. Pegs were taken from the same sample group that Schmidt and Daniels (1999) used for their joint tests. End and edge distance was varied to achieve a minimum distance and still achieve ductile peg failure. Peg spacing was constant at three inches. If a joint was tested and only the pegs failed, a repair was made by replacing the pegs. The joint was then tested again and is denoted by a B following the test joint number. A summary of the eastern white pine joint tests follows in Table 2-1.

Table 2-1 Eastern White Pine Joint Test Summary

Test	Peg Diameter (in)	End Dist. (D)	Edge Dist. (D)	Spacing Dist. (D)	Yield Disp. (in)	Yield Load (lbs)	Stiffness (lbs/in)	Ult. Disp (in)	Ult. Load (lbs)	Ave. Peg G	Failure Type @ Yield	Failure Type @ Ultimate
EWP 01	1	2.5	2.5	3	0.13	4720	55,900	0.27	5160	0.649	Mortise/Peg	Mortise/Peg
EWP 02	1	2.5	2.5	3	0.22	5010	31,300	0.26	5370	0.451	Tenon/Peg	Tenon/Peg
EWP 03	1	3	3	3	0.17	5160	40,200	0.24	5870	0.842	Mortise/Peg	Mortise/Peg
EWP 04	0.75	4	4	4	0.11	2540	28,400	0.32	3520	0.668	Peg	Peg
EWP 04B	0.75	4	4	4	0.19	3320	18,100	0.32	3580	0.718	Peg	Peg
EWP 05	0.75	3	3	4	0.10	2400	34,000	0.31	3350	0.719	Peg	Peg
EWP 06	0.75	3	3	4	0.08	2570	54,700	0.25	3340	0.762	Tenon	Tenon
EWP 07	0.75	4	4	4	0.13	3530	30,100	0.25	3860	0.675	Peg	Mortise
EWP 08	1	4	4	3	0.15	5790	53,900	0.18	5920	0.652	Peg	Peg
EWP 08B	1	4	4	3	0.21	6750	41,900	0.25	7090	0.813	Peg	Mortise
EWP 09	0.75	4	4	4	0.14	3090	25,700	0.23	3480	0.612	Peg	Mortise/Peg
					Mean 3/4"	2910	31,830					
					Mean 1"	5490	44,640					

The relatively small number of joints tested and the different peg diameters make statistical work, such as determining a lower 5% exclusion limit on strength, to be of questionable value. Mean stiffness and yield values are reported in Table 2-1 as a function of peg diameter. Minimum detailing requirements for end and edge distances were found. Spacing distance (l_s) was considered to be an issue of construction detailing according to Schmidt and Daniels (1999). Minimum end (l_e) and edge (l_v) distances were found to be 4 peg diameters for eastern white pine (see Figure 2-1).

This distance is somewhat larger than the end and edge distances that Schmidt and Daniels (1999) reported. However, the strength and specific gravity of the eastern white pine is lower than that of the species they tested.

2.7. Dowel Bearing Strength

Dowel bearing tests were conducted following the joint tests. Testing procedures of Schmidt and Daniels were followed. Two 4”x 4”x 1-1/2” blocks were cut from each mortise member and each tenon member. The samples were knot and check free if possible. The samples were orientated in the direction they would be in the joint. The mortise member samples were loaded perpendicular to grain and tension member samples were loaded parallel to grain. The 5% offset method of analysis was used. All the dowel bearing samples were tested with a one-inch diameter steel rod. A time delay occured between the joint tests and cutting of dowel bearing specimens. This delay resulted in a loss of moisture content in the material. Additional specimens were cut for the purpose of verifying a spring theory that will be discussed later in Chapter 3. A summary of the dowel bearing results is provided in Table 2-2. In the table, the value *K* is the number of standard deviations between the mean yield value and the lower 5% exclusion limit, using a 75% confidence level (see Table 3, ASTM D 2915).

Table 2-2 Eastern White Pine Dowel Bearing Test Results

Number	Yield Value (lbs/in^2)	Stiffness (lbs/in^3)	Number	Yield Value (lbs/in^2)	Stiffness (lbs/in^3)
EWP01M1	1,950	20,300	EWP01T1	4,890	115,300
EWP01M2	1,720	16,000	EWP01T2	4,590	118,200
EWP02M1	1,640	16,600	EWP02T1	4,960	120,300
EWP02M2	1,420	12,900	EWP02T2	4,660	98,300
EWP03M1	1,530	12,900	EWP03T1	4,800	86,400
EWP03M2	1,490	14,500	EWP03T2	4,570	92,100
EWP04M1	1,800	16,200	EWP04T1	5,830	137,500
EWP04M2	1,960	23,300	EWP04T2	5,560	140,600
EWP05M1	1,390	12,100	EWP05T1	5,270	120,500
EWP05M2	1,730	15,600	EWP05T2	6,100	162,000
EWP06M1	1,680	17,500	EWP06T1	5,140	160,400
EWP06M2	1,720	17,800	EWP06T2	4,580	105,900
EWP07M1	2,000	28,600	EWP07T1	3,840	96,000
EWP07M2	2,950	27,100	EWP07T2	3,940	87,100
EWP08M1	1,490	13,100	EWP08T1	4,390	98,200
EWP08M2	1,640	14,500	EWP08T2	4,100	118,900
Mean	1,760	17,400	Mean	4,830	116,100
St. Dev.	370	5,000	St. Dev.	640	24,000
5% Exclusion	1,030		5% Exclusion	3,560	
COV	0.210		COV	0.133	
K	1.977		K	1.977	

Dowel bearing test results are reported in a different way than in previous research. The procedure used to report bearing stiffness in this research was to divide the initial slope of the load deflection plot by the specimen width and the peg diameter. The yield load of the sample has been converted to a yield stress. Yield stress has also been found using the width of the specimen and the peg diameter, similar to past procedures used by Schmidt and MacKay (1997) and Schmidt and Daniels (1999).

The stiffness calculations of previous research did not take into account the exact width of the specimen, introducing the potential for error. The width of the specimen is directly related to the stiffness of the sample. A solution to this discrepancy is to report the stiffness in units of lbs/in^3. The change in reporting stiffness values will lead to more accurate comparisons between tests.

2.8. Detailing Requirements (End/Edge/Spacing)

Required end and edge distances for eastern white pine joints and the joint species that were tested by Schmidt and Daniels (1999) are summarized in Table 2-3. These detailing requirements resulted in peg failures using the 5% offset method of yield analysis. All of the joints tested to obtain these distances were unseasoned and subjected to short term loading; failure was reached in approximately 10 to 15 minutes. Long-term loading and seasoning effects were not taken into consideration when determining these minimum detailing requirements. A factor of safety is also not considered in these calculations. However a factor of safety will not be applied in this area of design, but rather it will be incorporated into the design load of the joint.

Table 2-3 Minimum Detailing Requirements (Used for long-term tests)

Species	End (D)	Edge (D)	Spacing (D)
Douglas Fir	2	2.5	2.5
Eastern White Pine	4	4	3*
Red/White Oak	2	2	2.5
Southern Yellow Pine	2**	2	3

*A constant value of 3" was used for testing
**3D with drawbore

2.9. Joint Strength Correlation

A correlation between joint strength and the specific gravity of the joint material was examined. The joints in the correlation study consisted of the recycled Douglas fir, red oak and southern pine joints tested by Schmidt and Daniels (1999), plus the eastern white pine joints tested in this research. All of the pegs in this study came from the same sample group of white oak. The yield stress for peg shear was then found as the average value on the peg cross section, using the yield load and assuming four shear planes, two shear planes per peg. This type of peg failure is the most common throughout all of the joints tested. Comparing shear stress rather than joint yield load also makes it possible to include results for the joints that were tested with 3/4” diameter pegs. A plot of the average shear yield stress versus base material specific gravity is shown in Figure 2-8.

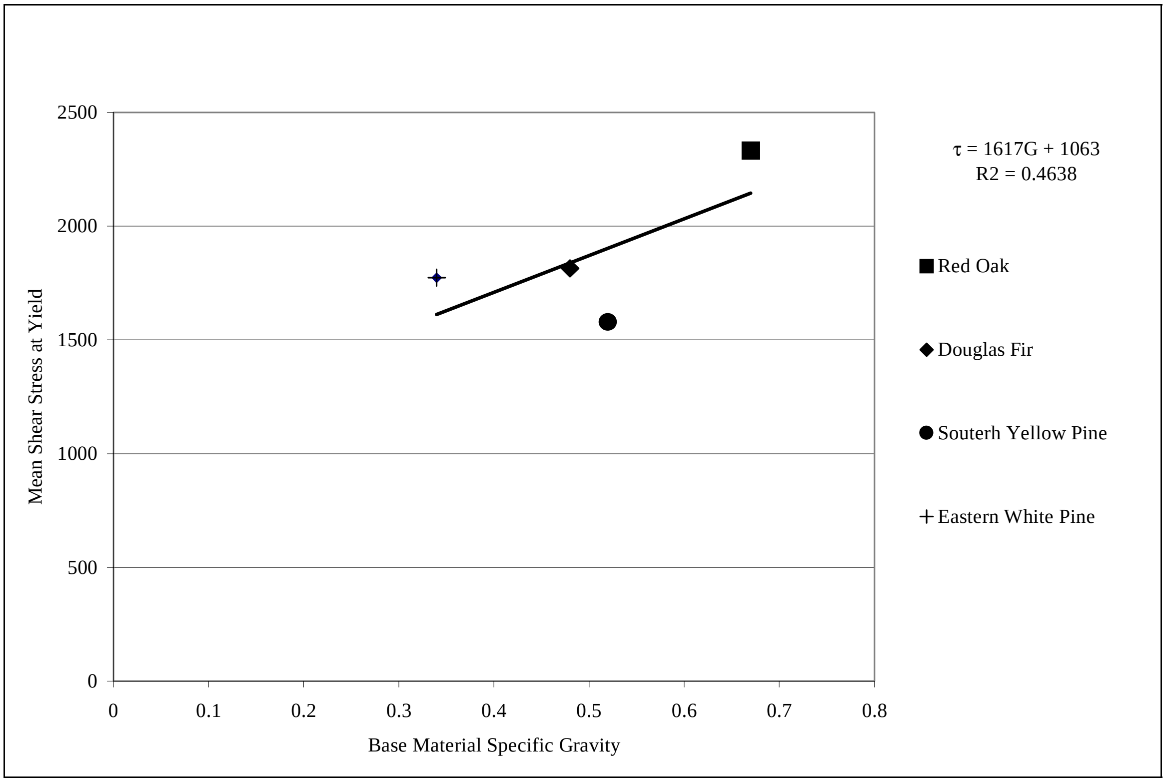


Figure 2-8 Correlation of Specific Gravity to Peg Joint Shear Stress

The base material specific gravity is related to base material strength. With a relationship between base material specific gravity and base material strength, a correlation between confinement strength and specific gravity is assumed. In other words a higher base material specific gravity equates to a larger peg shear yield value and higher joint yield strength. With increased confinement strength the peg is subjected to a smaller shear span *L* (Figure 2-9) over which it can deform producing a higher joint yield value.

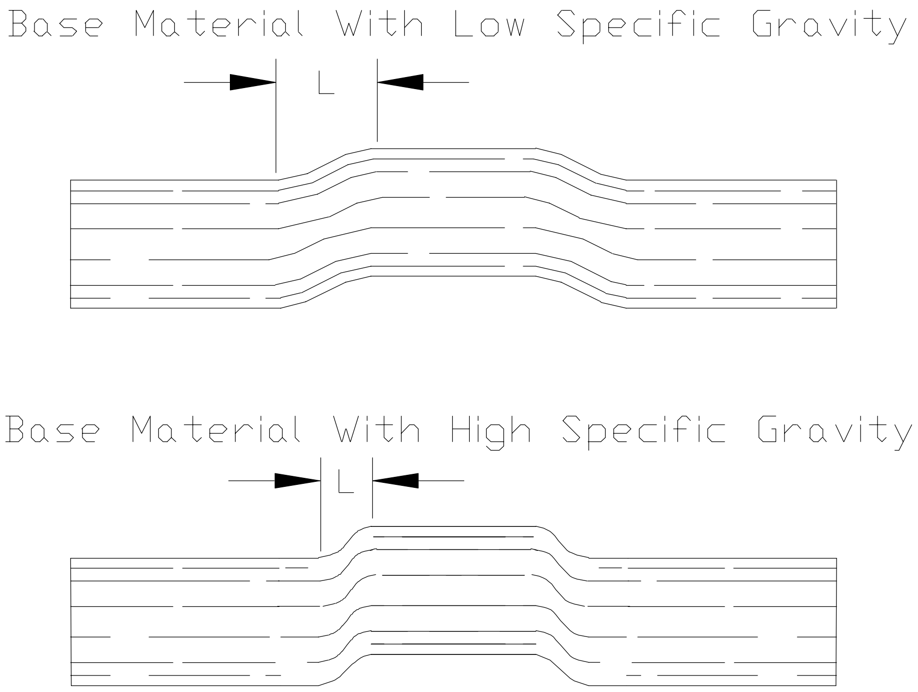


Figure 2-9 Illustration of Peg Failure

3. Spring Theory

3.1. Theory/Possible Uses

Currently dowel bearing strength and stiffness for different species combinations of base and peg material can be found only by testing each base material species with the corresponding peg species. Obviously many tests would have to be performed in order to obtain a comprehensive table of strengths and stiffnesses for varying combinations of base and peg species. One possible solution would be to test the peg material and base material separately and then add the properties mathematically. The behavior of the combined materials is based on the theory that the two components of the joint, the base material and the peg, carry load as two springs in series. Figure 3-1 is a visual representation of the spring theory.

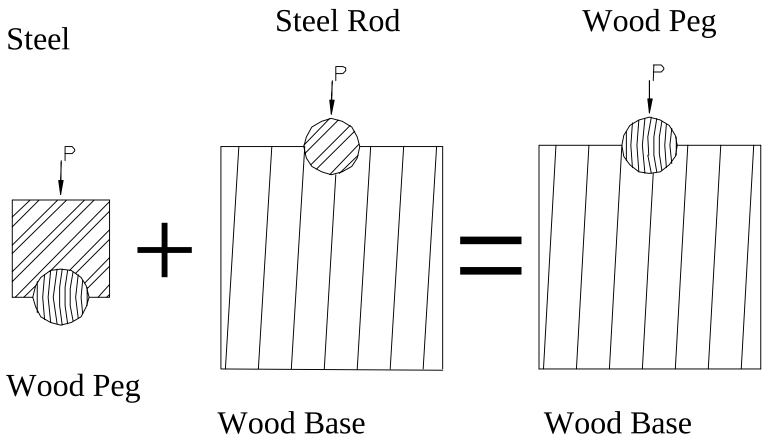


Figure 3-1 Spring Theory Concept from Schmidt and Daniels (1999)

A procedure to combine the material data mathematically would reduce the need for future testing and make better use of the data that has already been acquired. This method of mathematically combining material properties is limited to the dowel bearing properties of a base material loaded with a wood peg. Schmidt and Daniels (1999) developed the theory. It is verified in this research.

3.2. Test Procedures

The species used were eastern white pine for the base material and white oak for the pegs. Test procedures were modeled after those performed by Schmidt and Daniels (1999). Three types of tests were needed in an effort to validate the spring theory. The first test is a dowel bearing test of the eastern white pine base material. The dowel bearing tests conformed to ASTM D5764 with a stroke rate of 0.024 in/min (ASTM 1999). (This stroke rate was increased to .050 in/min in some cases for the dowel bearing strengths of the long-term joints, reported in the appendices). The dowel bearing strength of the eastern white pine is found using a 4”x 4”x 1-1/2” specimen with a half circle 1” in diameter in the top (see Figure 3-2 below). A steel dowel is placed in the 1” diameter trough; load is then applied to the steel dowel.

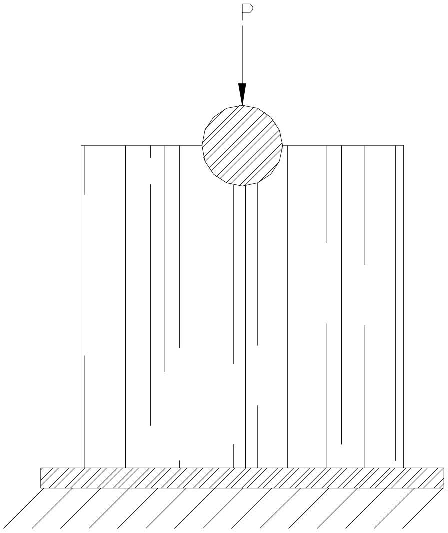


Figure 3-2 Base Material Dowel Bearing Test (From Schmidt and Daniels 1999)

A peg bearing test is the second type of test used. A peg bearing test uses a 1-1/2" square steel load block with a 1" diameter half circle in one face. The peg is placed into a long shallow trough cut into a steel base plate with the ends of the peg clamped to the base plate to hold the peg flat during the test. The load block is placed on top of the peg. Load is applied to the load block to test the peg bearing strength (see Figure 3-3 below).

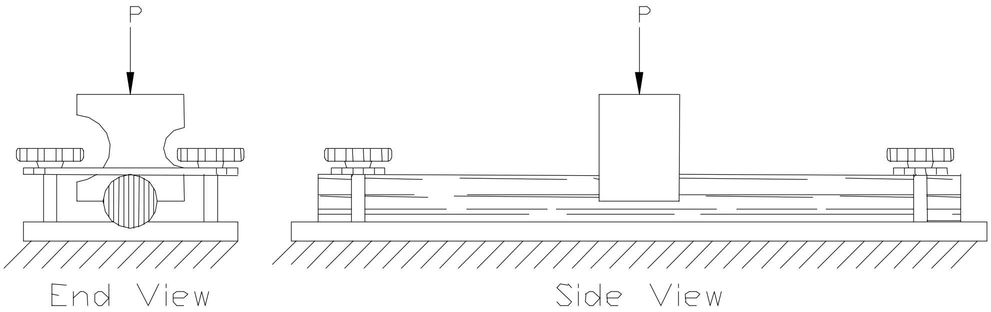


Figure 3-3 Peg Dowel Bearing Test (From Schmidt and Daniels 1999)

The third test involves a combination of the eastern white pine base material block and a white oak peg. The peg is secured in the trough of the base plate as in Figure 3-3, but the load is applied through the base material block. The shallow trough prevents bearing failure of the peg remote from the interface between the peg and the base material. The test is similar to that shown in Figure 3-3 with the base material in place of the steel load block.

3.3. Method and Results

Load-deflection results of base material bearing and peg bearing tests were processed by a program that performed a filtering operation on the load-displacement curve. The program smoothed the test data into uniform load increments of 25 pounds and found the corresponding displacement. The displacements of the two tests at the same load were then added in order to synthesize a load-displacement curve for the combined materials. The 5% offset method with a 1” peg diameter was once again used for all of the tests in question.

In order to reduce variability and to achieve a higher degree of confidence, matched bearing samples were cut from each piece of eastern white pine. Four bearing samples were cut from each tenon member and each mortise member. Two of the four samples were used for the conventional dowel bearing tests (Figure 3-2) and two for combined tests. To obtain matched specimens for the pegs, two-foot long pegs were cut in half so that one half of the peg could be used in the peg bearing test (Figure 3-3) and the other half in the combined test. A test specimen distribution table is given in Table 3-1.

Table 3-1 Spring Theory Test Distribution

Base Material Bearing Tests			Peg Bearing Tests		Mathematically Combined	Physically Combined
Tenon 1, Test 1	Tenon 1 Average	+	Peg 1	=	Tenon 1 Average + Peg 1	Tenon 1 with Peg 1
Tenon 1, Test 2			Peg 2	=	Tenon 1 Average + Peg 2	Tenon 1 with Peg 2
Tenon 2, Test 1	Tenon 2 Average	+	Peg 3	=	Tenon 2 Average + Peg 3	Tenon 2 with Peg 3
Tenon 2, Test 2			Peg 4	=	Tenon 2 Average + Peg 4	Tenon 2 with Peg 4

In total thirty-two comparisons were made. Sixteen comparisons were made from material taken from the mortise and sixteen comparisons were made from material taken from the tenon. Mortise material was loaded perpendicular to the grain while the tenon material was loaded parallel to the grain. As expected the difference in grain direction has a substantial effect on both the strength and stiffness of the material. The two-foot pegs were chosen randomly from a separate supply; they were not from the sample group used for virtually every other peg from both this research and the research of Schmidt and Daniels (1999). The pegs from that sample group were one-foot long, making it impossible to achieve matched peg specimens for the study. Table 3-2 is a summary of results of the comparison.

Table 3-2 Spring Theory Summary

Test	Mathematically Combined		Physically Combined		Ratio (Mathematically/Physically)	
Number	Yield Stress (lbs/in^2)	Stiffness (lbs/in^3)	Yield Stress (lbs/in^2)	Stiffness (lbs/in^3)	Yield Stress	Stiffness
M01	1,820	12,900	1,760	15,900	1.04	0.81
M02	1,810	13,900	1,870	16,000	0.97	0.87
M03	1,520	11,600	1,420	10,700	1.07	1.08
M04	1,530	10,400	1,350	10,400	1.13	1.00
M05	1,520	11,400	1,480	11,900	1.03	0.96
M06	1,530	10,400	1,390	9,700	1.10	1.07
M07	1,630	13,500	1,460	13,700	1.12	0.99
M08	1,750	15,200	1,690	16,000	1.03	0.95
M09	1,490	9,900	1,460	12,300	1.03	0.80
M10	1,490	10,100	1,440	10,900	1.04	0.92
M11	1,720	13,400	1,580	13,200	1.09	1.02
M12	1,740	13,600	1,540	12,900	1.12	1.05
M13	2,030	17,700	1,960	29,500	1.03	0.60
M14	2,160	18,900	2,160	26,100	1.00	0.72
M15	1,630	10,500	1,540	13,200	1.06	0.79
M16	1,630	10,400	1,510	11,600	1.08	0.90
				Mean	1.06	0.91
				Standard Deviation	0.047	0.135

Test	Mathematically Combined		Physically Combined		Ratio (Mathematically/Physically)	
Number	Yield Stress (lbs/in^2)	Stiffness (lbs/in^3)	Yield Stress (lbs/in^2)	Stiffness (lbs/in^3)	Yield Stress	Stiffness
T01	3,530	46,320	3,060	57,590	1.15	0.80
T02	2,630	46,350	2,660	45,310	0.99	1.02
T03	2,300	41,310	2,280	30,370	1.01	1.36
T04	3,200	48,710	3,370	48,360	0.95	1.01
T05	3,050	46,830	3,450	34,000	0.88	1.38
T06	2,850	42,700	2,570	34,870	1.11	1.22
T07	3,300	49,420	3,190	44,990	1.03	1.10
T08	3,130	45,170	2,670	43,610	1.17	1.04
T09	2,980	44,940	2,620	46,250	1.13	0.97
T10	2,910	45,220	2,520	47,140	1.15	0.96
T11	4,150	57,860	3,240	46,130	1.28	1.25
T12	3,420	43,820	3,200	51,820	1.07	0.85
T13	3,410	50,560	3,340	45,130	1.02	1.12
T14	2,330	41,380	2,330	42,300	1.00	0.98
T15	2,560	43,860	2,650	36,850	0.97	1.19
T16	2,750	47,690	2,330	45,640	1.18	1.04
				Mean	1.07	1.08
				Standard Deviation	0.105	0.166

In order to compare the strengths and stiffnesses of the tests, the mathematically combined results were divided by the physically combined results. A ratio of 1.00 would therefore correspond to the mathematical model perfectly representing the physically model. The 5% offset method of analysis was used to find results for both the mathematically combined and physically combined tests.

Based on examination of the table, it is evident that the spring theory represents the combined material tests relatively well, given the natural variability of wood. In general the spring theory showed a higher value for strength. With unity values ranging between 0.88 and 1.28, the spring theory accurately predicted the combined material test yield values.

The difference in stiffness values was also within a reasonable range. Unity values varied between 0.60 and 1.28 with an average perpendicular to grain ratio of 0.91 and an average parallel to grain ratio of 1.08. In general the mathematically combined tests represented the physically combined tests well.

The eastern white pine base material has a significantly lower bearing strength perpendicular to grain than the white oak used for the pegs (see Figure 3-4). The difference in bearing strength meant that the weaker eastern white pine material dominated the test results; often in the combined tests, little peg damage was visible. While difficult to quantify, this effect is a consideration.

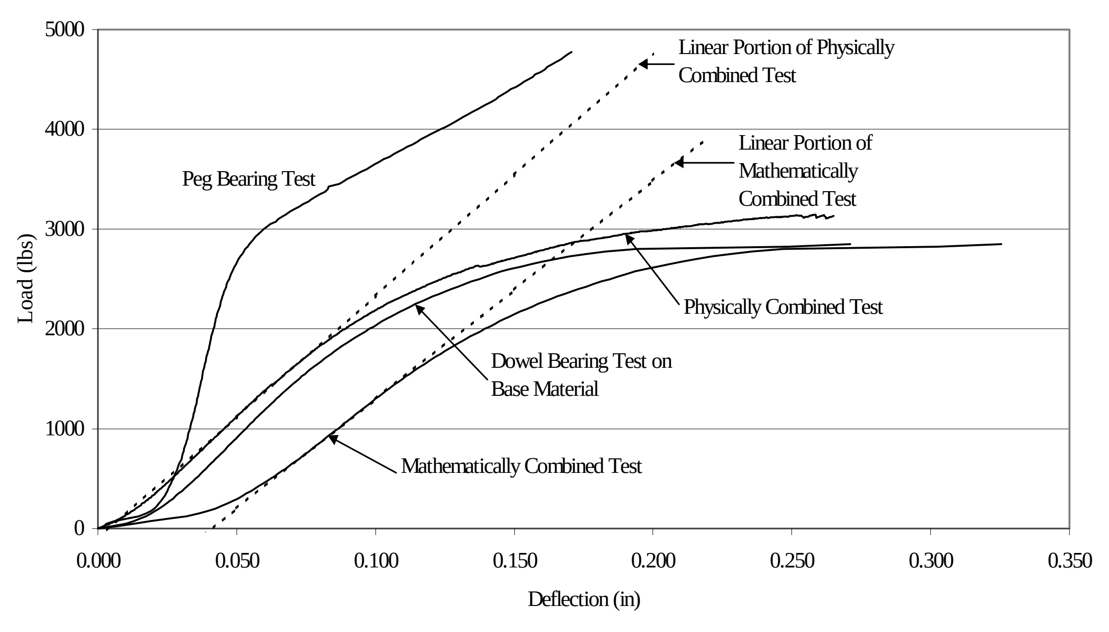


Figure 3-4 Typical Spring Theory Plot (Base Material Loaded Perpendicular to Grain)

Combined tests with the base material loaded parallel to grain resulted in more peg damage. In the tests with the base material loaded parallel to grain the base material was the stiffer of the two materials; resulting in the peg material properties dominating the combined tests (see Figure 3-5).

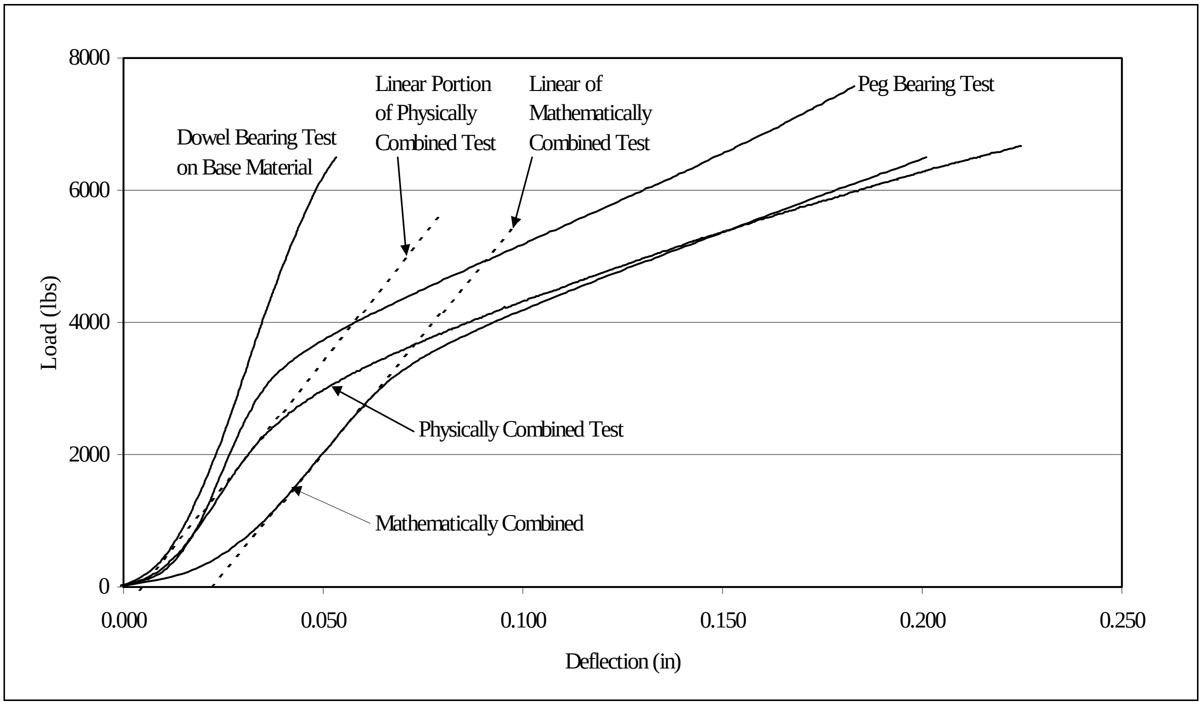


Figure 3-5 Typical Spring Theory Plot (Base Material Loaded Parallel to Grain)

With the material of lower strength and stiffness dominating the combined test results, a comparison of the combined test results verses the test results of only the weaker/softer material was performed. Table 3-3 below is a comparison of the properties of the weaker/softer material versus the combined test results. The weaker/softer material being the base material when loaded perpendicular to grain and the peg when the base material is loaded parallel to grain (see Figure 3-4 and Figure 3-5).

Table 3-3 Comparison of Combined Test Results with Weaker/Softer Material

Test	Mathematically Combined		Physically Combined		Difference (Mathematically/Physically)	
Number	Yield Stress (lbs/in^2)	Stiffness (lbs/in^3)	Yield Stress (lbs/in^2)	Stiffness (lbs/in^3)	Yield Stress	Stiffness
M01	1,760	15,900	1,840	18,200	0.96	0.87
M02	1,870	16,000	1,840	18,200	1.02	0.88
M03	1,420	10,700	1,530	14,700	0.93	0.73
M04	1,350	10,400	1,530	14,700	0.88	0.71
M05	1,480	11,900	1,510	13,700	0.98	0.87
M06	1,390	9,700	1,510	13,700	0.92	0.71
M07	1,460	13,700	1,880	19,700	0.78	0.70
M08	1,690	16,000	1,880	19,700	0.90	0.81
M09	1,460	12,300	1,560	13,800	0.94	0.89
M10	1,440	10,900	1,560	13,800	0.92	0.79
M11	1,580	13,200	1,700	17,600	0.93	0.75
M12	1,540	12,900	1,700	17,600	0.91	0.73
M13	1,960	29,500	2,470	27,900	0.79	1.06
M14	2,160	26,100	2,470	27,900	0.87	0.94
M15	1,540	13,200	1,570	13,800	0.98	0.96
M16	1,510	11,600	1,570	13,800	0.96	0.84
				Mean	0.92	0.83
				Standard Deviation	0.06	0.10

Test	Mathematically Combined		Physically Combined		Difference (Mathematically/Physically)	
Number	Yield Stress (lbs/in^2)	Stiffness (lbs/in^3)	Yield Stress (lbs/in^2)	Stiffness (lbs/in^3)	Yield Stress	Stiffness
T01	4,890	115,300	3,460	90,400	1.41	1.28
T02	4,590	118,200	2,620	84,400	1.75	1.40
T03	4,960	120,300	2,270	80,600	2.19	1.49
T04	4,660	98,300	3,170	100,100	1.47	0.98
T05	4,800	86,400	3,070	95,200	1.56	0.91
T06	4,570	92,100	2,810	100,000	1.63	0.92
T07	5,830	137,500	3,220	94,400	1.81	1.46
T08	5,560	140,600	3,000	93,100	1.85	1.51
T09	5,270	120,500	2,870	89,500	1.84	1.35
T10	6,100	162,000	2,860	83,200	2.13	1.95
T11	5,140	160,400	4,180	112,100	1.23	1.43
T12	4,580	105,900	3,270	88,800	1.40	1.19
T13	3,840	96,000	3,500	109,500	1.10	0.88
T14	3,940	87,100	2,330	71,100	1.69	1.23
T15	4,390	98,200	2,570	70,100	1.71	1.40
T16	4,100	118,900	2,760	82,500	1.49	1.44
				Mean	1.64	1.30
				Standard Deviation	0.29	0.28

The results of the comparison indicate that data from the weaker/softer material alone is not sufficient to accurately predict the strength and stiffness of the combined materials. Instead, the two test curves must be added mathematically and then the resulting strength determined by the 5% offset method applied to the combined response curve.

Spring theory tests performed by Schmidt and Daniels (1999) used red oak base material and white oak pegs. Schmidt and Daniels reported the mathematically combined results to have, on average, a 0.4% larger yield value and 25.3% lower stiffness. A trend of underestimating the stiffness when the base material is stiff is developed in both sets of data. An explanation of this trend is not known.

4. Long Term Seasoning/Creep Tests

4.1. Introduction

Load duration effects relating to mortise and tenon joints are of concern in two aspects of timber frame design. The first is the relationship between load duration and joint strength. What is a safe long-term design load? The second area of concern is one of serviceability. How much will the joint deflect under typical sustained loading; is this value allowable for the structure and the structure's components? In an effort to answer these questions, long-term load tests were conducted using four different commonly used wood species: Douglas fir, southern yellow pine, white oak and eastern white pine. The corresponding pegs were white oak; taken from the same supply that was used for both the eastern white pine tests discussed earlier and the research performed by Schmidt and Daniels (1999).

Detailing requirements used for the long-term tests were based upon minimum values contained in Table 2-3. These end, edge and spacing requirements were used to evaluate their suitability for long-term load. Excessive deflection under load, cracking of the tenon or mortise, or a loss of yield strength may indicate the need for a load duration factor applied in joint design.

Seasoning effects on mortise and tenon joints can be both a strength and a serviceability issue. In standard practice, timber frame structures are often erected with timbers that have significantly higher moisture content than the eventual equilibrium moisture content. Moisture content in the realm of 20% or higher is common during construction. In a dry environment equilibrium moisture content can be in the single digits. This drop of moisture content can have the obvious effect of shrinkage. The effects on joint strength and stiffness are investigated in this research.

The investigation included three different load levels and four different species of wood. The load levels were zero load for the control group, and 1000 lb and 2000 lb. The magnitude of the long-term load was determined by the strength of the short-term tests conducted in this research and by Schmidt and Daniels (1999). The joints were not kept in a special conditioning chamber, but rather they were allowed to season in an environment in which both the temperature and humidity were subject to variation.

Short-term joint tests to failure were conducted on all of the joints after the interval of sustained load and seasoning was concluded. A short-term test procedure similar to that of the eastern white pine joint tests was used.

4.1.1. Test Frame Set-up

To test the effects of load duration on mortise and tenon joints, a long-term load test frame was designed. A test frame was constructed to utilize a coil spring that could be adjusted to maintain a desired load. The load frame held two joints at the same time, each joint pulling against the other. Figure 4-1 shows the test frame with two joint specimens. Two-inch diameter schedule 40 pipe was used to hold the two joints apart. The pipes were connected to the joints with floor flanges that were bolted to the ends of the mortise member.

The spring was contained within a piece of four-inch square tubing, three inches long. Side plates were welded to the sides of the square tubing. The side plates had a dual purpose. The first was structural and allowed connection to the tenon of one of the test joints. The second purpose was to serve as a surface for calibration markings. Locations of the calibration markings were obtained by compressing the spring to known loads of 1000 and 2000 pounds using an Instron model 1332 servo-hydraulic testing machine. The springs all came from the same source and have stiffnesses of approximately 1000 lb/in. Each spring was calibrated individually in order to eliminate any inconsistencies in spring stiffness.

Two plates were bolted on each tenon and secured with lag screws. The plates connected to the rest of the test frame by way of a one-inch diameter hole that allowed a length of all-thread or a length of round stock to run through the plates that were attached to the tenon.

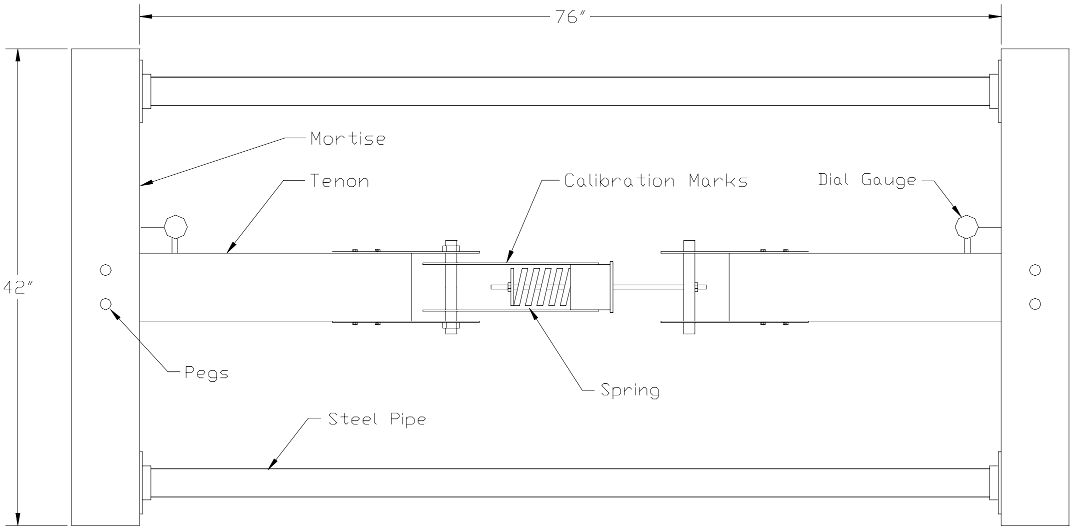


Figure 4-1 Long Term Test Frame

4.1.2. Joint Preparation

An effort was made to prepare the joints in a manner that would be similar to standard timber frame practice. Some exceptions were made to allow for improved observation of the joints. For instance, all of the mortised members had a through mortise; that is, the mortise hole extended all of the way through the mortised member. A through mortise allows for visual inspection of the end of the tenon member. Paraffin wax was also applied to all of the tenon tips. The procedure was to apply a layer of wax, which was rubbed on the end of the tenon. Then the wax was melted with a hot air blower. The wax helped to seal the end of the tenon. The sealed end reduced moisture loss through the end grain in an attempt to prevent checking of the tenon, particularly the end of the tenon that is subjected to high stresses. In practice end grain on timbers is usually sealed to control checking. Also the end of the tenon is usually hidden inside the mortise, away from air circulation. Hence, the specimen preparation is regarded as representative of that for actual in-service joints.

For all of the long-term test specimens, the supplier cut the mortises and tenons. However, none of the joints arrived with peg holes, since tenon length and peg hole locations were test parameters selected at the time of joint assembly.

Some of the Douglas fir and southern yellow pine joints were assembled with a drawbore. Drawboring is a method of “pre-stressing” the joint. Drawboring is preformed in practice to make a tighter joint that will remain closed after the timbers are seasoned. The procedure used when drawboring was to drill the mortise peg hole with the tenon member out of the joint. The joint was then clamped together and a mark was made on the tenon at the center of the peg hole with the drill bit. The tenon was then removed and the mark was offset 3/32” toward the tenon shoulder and a hole was drilled at the location of the new mark.

With the exception of the previously discussed drawbored joints, the procedure for construction is as follows. The joint was clamped together and a mark was made in the appropriate location for the center of the peg hole. The peg holes were then drilled through the joint.

Pegs were driven in the peg holes in such a way that the load was applied tangentially to the peg. The growth rings were parallel to the tenon member and the load to be applied. This orientation was followed in both the eastern white pine short-term tests and the research conducted by Schmidt and Daniels (1999).

4.1.3. Monitoring and Load Adjustment Procedure

During the period of long-term loading, joint displacement was recorded approximately every seven days. Date, temperature and relative humidity were recorded along with the deflection given by one or two dial gauges attached to each joint. Moisture content of the control specimens was recorded approximately every month. Moisture content was recorded with a Delmhorst J-2000 moisture meter with 1.25" penetration pins. The moisture content of the loaded joints was not monitored, because the impacts due to the insertion of the moisture meter pins could affect the joint deflection. With the loaded joints, even a slight disturbance could be detected on the dial gauges.

Load adjustments were made when deemed necessary. The amount that the spring was compressed relative to the calibration mark severed as a guide when the load needed to be adjusted. Load was adjusted when the spring was off the target by approximately 1/8-inch. With a spring constant of approximately 1000 lb/in, this results in a variation of 125 lb. Load was not adjusted more often because this adjustment also disturbed the joint deflection. Adjustment of the load without minor disturbances on the joint was impossible. When load adjustment was performed, the procedure consisted of recording the joint deflection prior to any adjustment. The load was then adjusted by compressing the spring to the calibration mark by tightening the nut down further on the length of all-thread rod. The joint deflection was then recorded again. This adjustment procedure is visible as a jump in deflection on the time deflection graphs that follow.

4.2. Douglas Fir

Long-term seasoning and creep tests were conducted on twelve Douglas fir joints. Six joints were loaded, while the control group was composed of the remaining six joints. Six joints were drawbored in an effort to investigate benefits or possible drawbacks to drawboring. The drawbored joints were divided equally between the loaded and control groups of joints. Detailing requirements made by Schmidt and Daniels were followed: 2.5D edge distance, 2.0D end distance and 2.5D spacing. All of the Douglas fir joints were connected with 1” diameter white oak pegs.

4.2.1. Loading and Load Duration

The load group of six joints was loaded for 348 days at 2000 lb. This long-term load is 35% of the average yield load reported by Schmidt and Daniels (1999) from testing of recycled Douglas fir joints with 1” diameter pegs. Note that the joints used in the long-term load test were fabricated from green material, not recycled. The characteristics of the individual joints are given in Table 4-1. The time-deflection curves of each loaded joint are shown in Figure 4-2.

Table 4-1 Douglas Fir Long-Term Joint Parameters

Joint Number	Long Term Load (lb)	Drawbore	Peg Dia. (In)
DF21	2000	No	1
DF22	2000	No	1
DF23	2000	No	1
DF24	0	No	1
DF25	0	No	1
DF26	0	No	1
DF27	2000	Yes	1
DF28	2000	Yes	1
DF29	2000	Yes	1
DF30	0	Yes	1
DF31	0	Yes	1
DF32	0	Yes	1

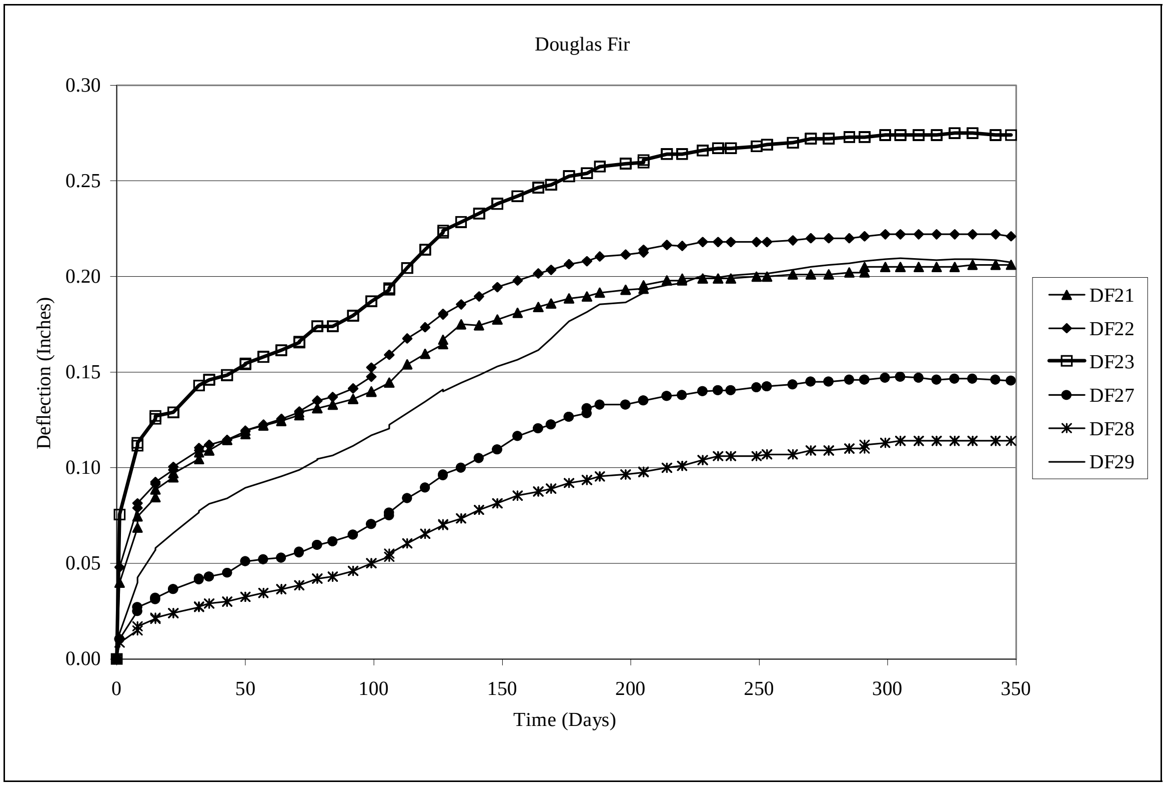


Figure 4-2 Douglas Fir Joint Deflection versus Time

To examine joint behavior after the initial load was applied, the time-deflection data was normalized at a time of one day after the start of the long-term test; the deflection at day 1 was set to zero. By normalizing the data, the highly variable initial deflection is eliminated; this process reveals the joints that had the largest variance in deflection after the test was started. The normalized time-deflection plot for Douglas fir is shown below in Figure 4-3.

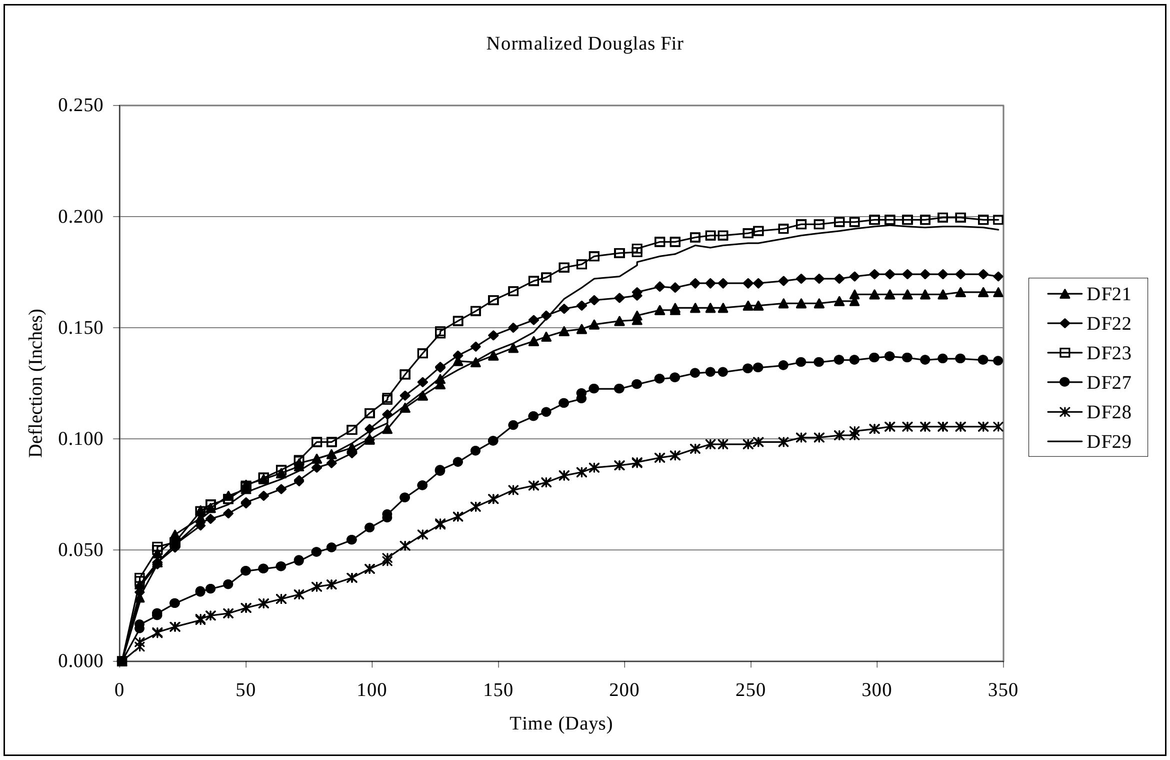


Figure 4-3 Normalized Douglas Fir Deflection versus Time

A plot showing the change with time of the normalized mean joint deflection and its standard deviation (σ) in either direction of the mean is shown in Figure 4-4. The normalized mean deflection at the conclusion of the long-term testing was 0.162”.

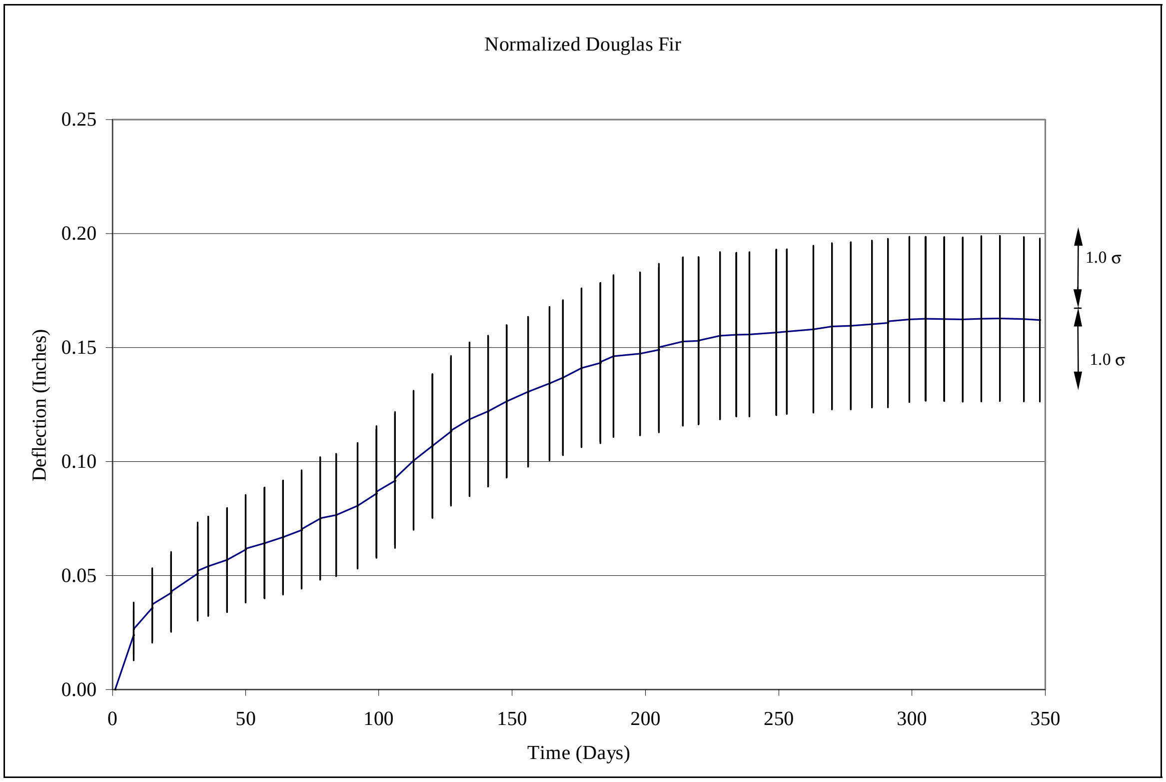


Figure 4-4 Douglas Fir Normalized Mean Joint Deflection versus Time

4.2.2. Moisture Content

The average moisture content of the control group at the beginning of long term testing was 18% based on moisture meter readings. The average moisture content at the end of testing was 7%. Plots of moisture content for the individual joints and mean moisture content for the group of control joints versus time are shown in Figure 4-5 and Figure 4-6. The standard deviation of the moisture content is also illustrated in Figure 4-6.

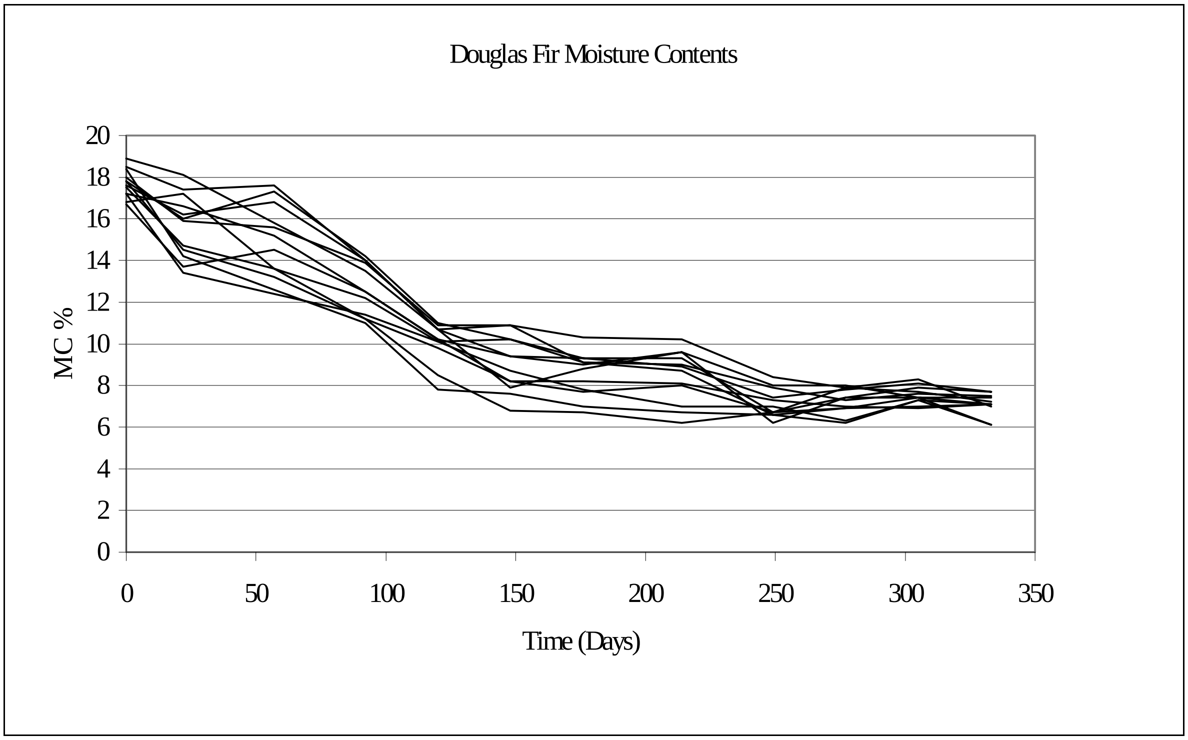


Figure 4-5 Douglas Fir Moisture Content

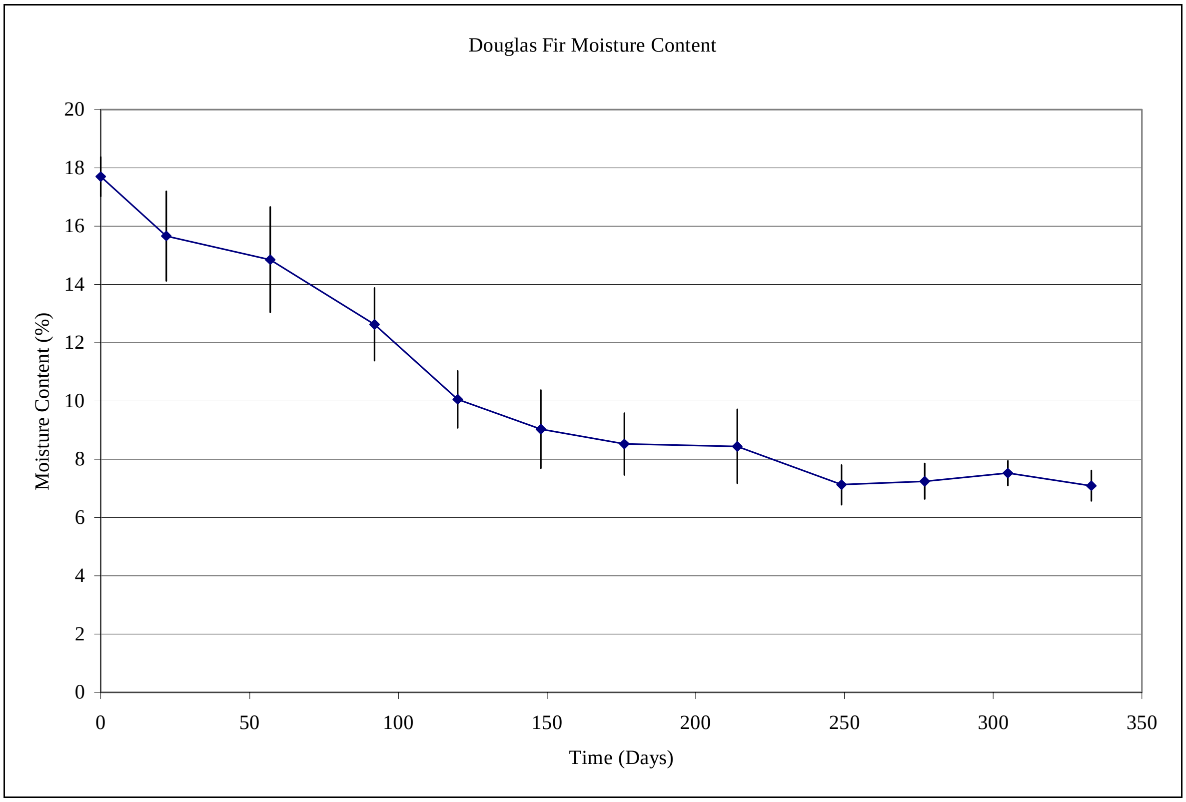


Figure 4-6 Douglas Fir Mean Moisture Content

4.2.3. Results and Conclusions of Time-Deflection Behavior

As can be seen from Figure 4-4 the deflection rate of the joints slowed to nearly zero after approximately 225 days. A slight amount of creep continued until the conclusion of the long-term testing.

Comparison of Figure 4-2 and Figure 4-3 reveals that joints with high initial flexibility also experienced more creep and shrinkage deflection than those with high initial stiffness. Since the materials used in construction of the joints were as identical as possible, this suggests that variations in fabrication and assembly (cutting tolerances) have a major influence on both initial and long-term deflections of mortise and tenon joints in tension.

The plot in Figure 4-7 shows the average deflection of the drawbored and the non-drawbored joints. Drawboring had a significant effect on the initial deflection when the load was applied; the initial deflections of the drawbored joints were substantially less than those of the non-drawbored joints. Drawboring also reduced the creep rate. Long-term deflection for the drawbored joints averaged about 20% less than that of the non-drawbored joints.

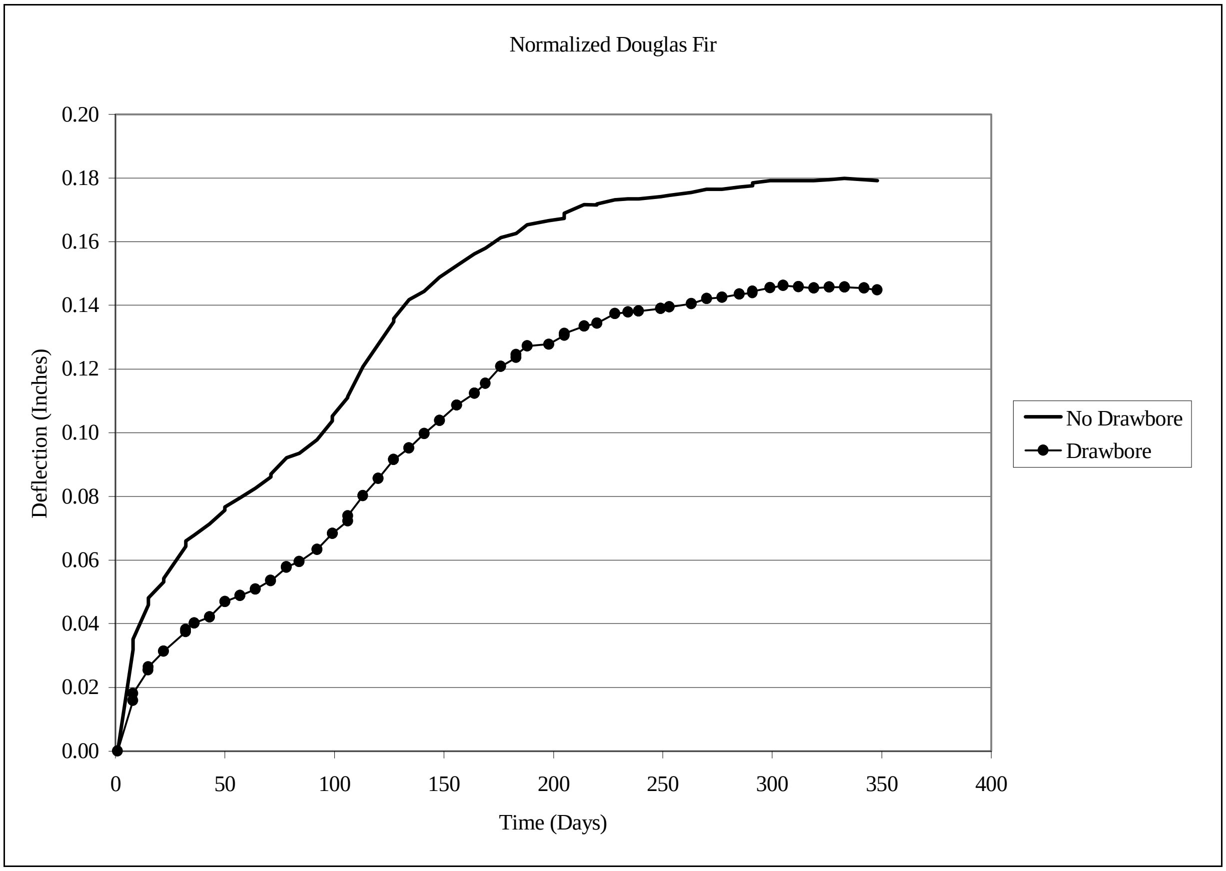


Figure 4-7 Douglas Fir Comparison

During assembly of the joints, two of the three loaded joints were damaged from drawboring. The tenon split behind both of the pegs in one joint and behind one peg in the other joint. Yet the joints were able to hold the load and the tests were continued. Reasons for the damage due to drawboring are varied. Whereas use of 1” diameter pegs is common in timber frame construction, they might be too stiff to drawbore safely. The drawbore offset (3/32” for these joints) might have been excessive. However, a drawbore of 1/8” is common for softwoods. Also possibly the tenon required more end distance to carry the increased stresses. Finally, the technique used during joint assembly might have been less precise than could be achieved by professional timber framers. In spite of the damage, drawboring did increase both the initial and long-term stiffness of the joints.

4.3. Southern Yellow Pine

Twenty-one southern yellow pine joints were contained in the load and control groups. Twelve of the joints were loaded, six at 2000 lb and six at 1000 lb. A load of 2000 lb is 40% of the mean yield value of 4960 lb found in research conducted by Schmidt and Daniels (1999); 1000 lb is 20% of the mean yield value. Schmidt and Daniels (1999) tested twelve joints, all with 1” diameter pegs. The detailing distances were 2.0D edge distance, 2.0D end distance and 3.0D spacing. In an attempt to prevent the tenon from splitting on the twelve drawbored joints, the end distance of all the drawbored joints was increased to 3.0D. Details of the individual joints are listed in Table 4-2.

4.3.1. Loading and Load Duration

Long-term load testing of the southern yellow pine joints lasted for 319 days. Twelve of the joints were drawbored by 3/32”. The drawbored joints did not develop tenon splits during construction, in constrast to two of the six Douglas fir drawbored joints.

Table 4-2 Southern Yellow Pine Long-Term Joint Parameters

Joint Number	Long Term Load (lb)	Drawbore	Peg Dia. (In)
SYP 21	1000	No	1
SYP 22	1000	No	1
SYP 23	1000	No	1
SYP 24	2000	No	1
SYP 25	2000	No	1
SYP 26	2000	No	1
SYP 27	0	No	1
SYP 28	0	No	1
SYP 29	0	No	1
SYP 30	2000	Yes	1
SYP 31	2000	Yes	1
SYP 32	2000	Yes	1
SYP 33	0	Yes	1
SYP 34	0	Yes	1
SYP 35	0	Yes	1
SYP 36	0	Yes	0.75
SYP 37	0	Yes	0.75
SYP 38	0	Yes	0.75
SYP 39	1000	Yes	0.75
SYP 40	1000	Yes	0.75
SYP 41	1000	Yes	0.75

The time-deflection plot of each joint is shown below in Figure 4-8. A normalized version of the southern yellow pine time-deflection plot, with the deflection at one day defined as the zero point, is also shown (see Figure 4-9). Comparison of the two plots reveals that again drawboring has a strong influence on initial deflection of the joints. The effect of drawboring on long-term deflection is not so obvious and is considered more closely later.

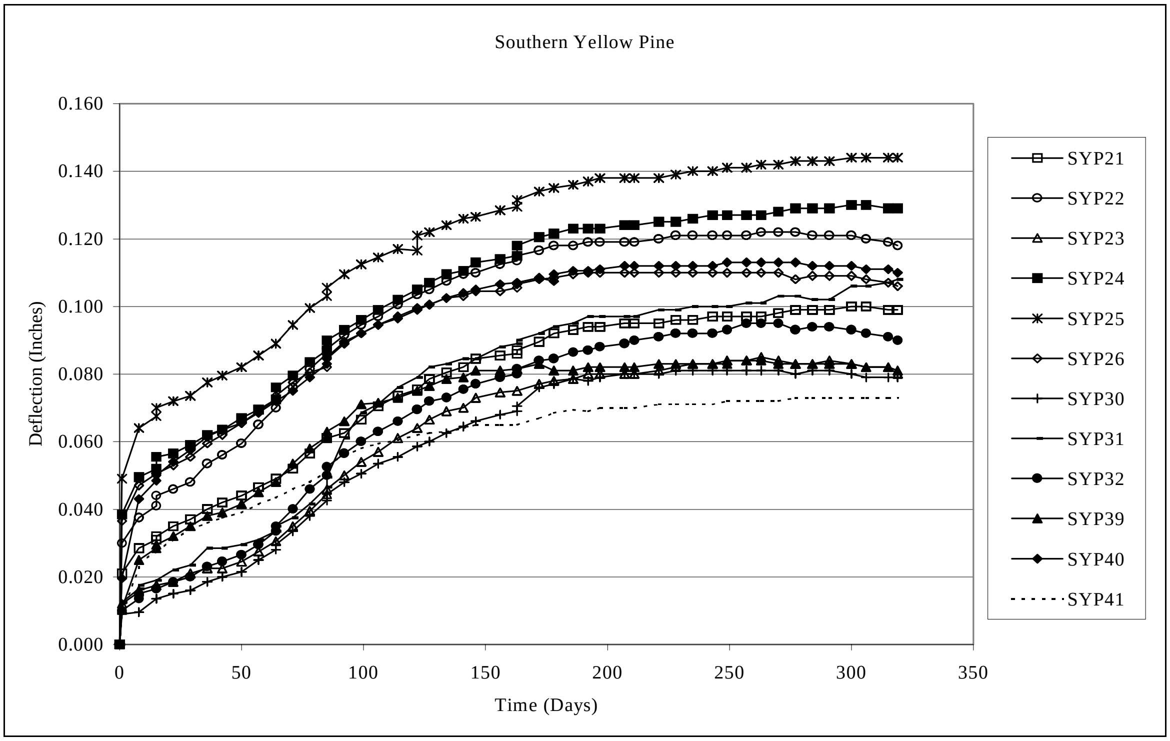


Figure 4-8 Southern Yellow Pine Joint Deflection verses Time

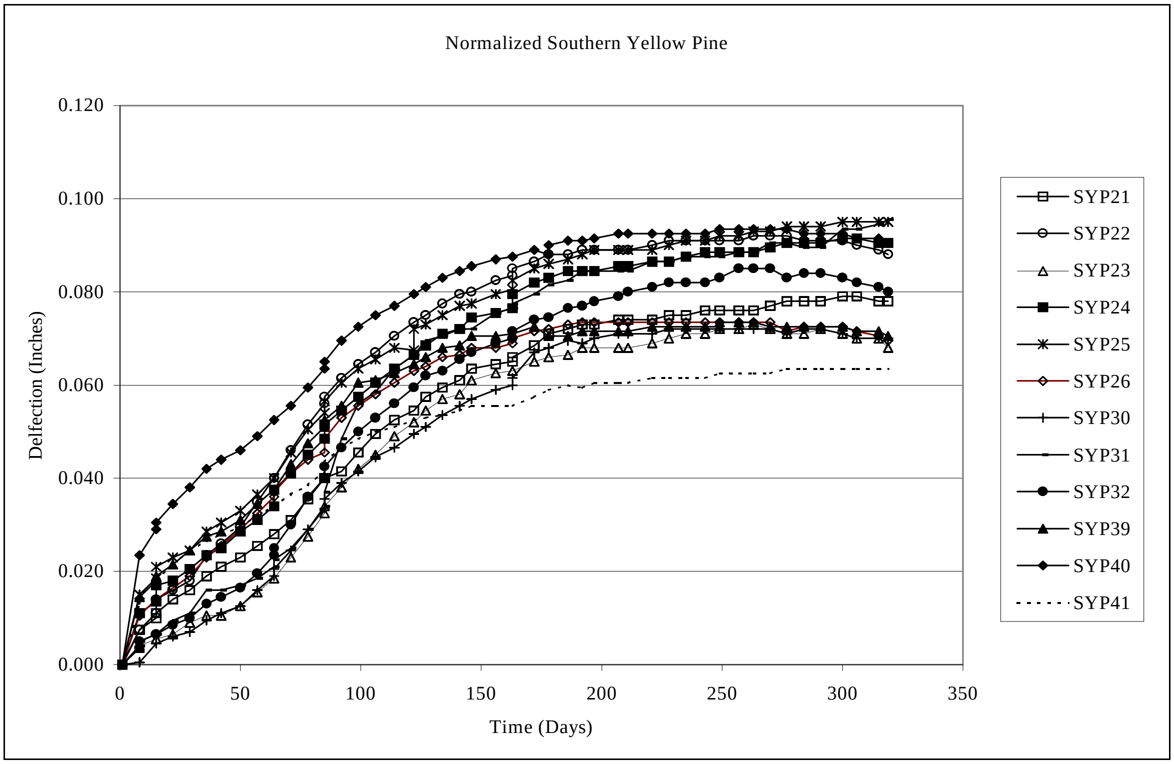


Figure 4-9 Normalized Southern Yellow Pine Deflection versus Time

Examination of the joint mean time-deflection plot (Figure 4-10) reveals that the mean creep rate slowed significantly after approximately 225 days. This is approximately the same time as for the Douglas fir joints. However in contrast, the southern yellow pine joints experienced a sizably smaller deflection than did the Douglas fir joints.

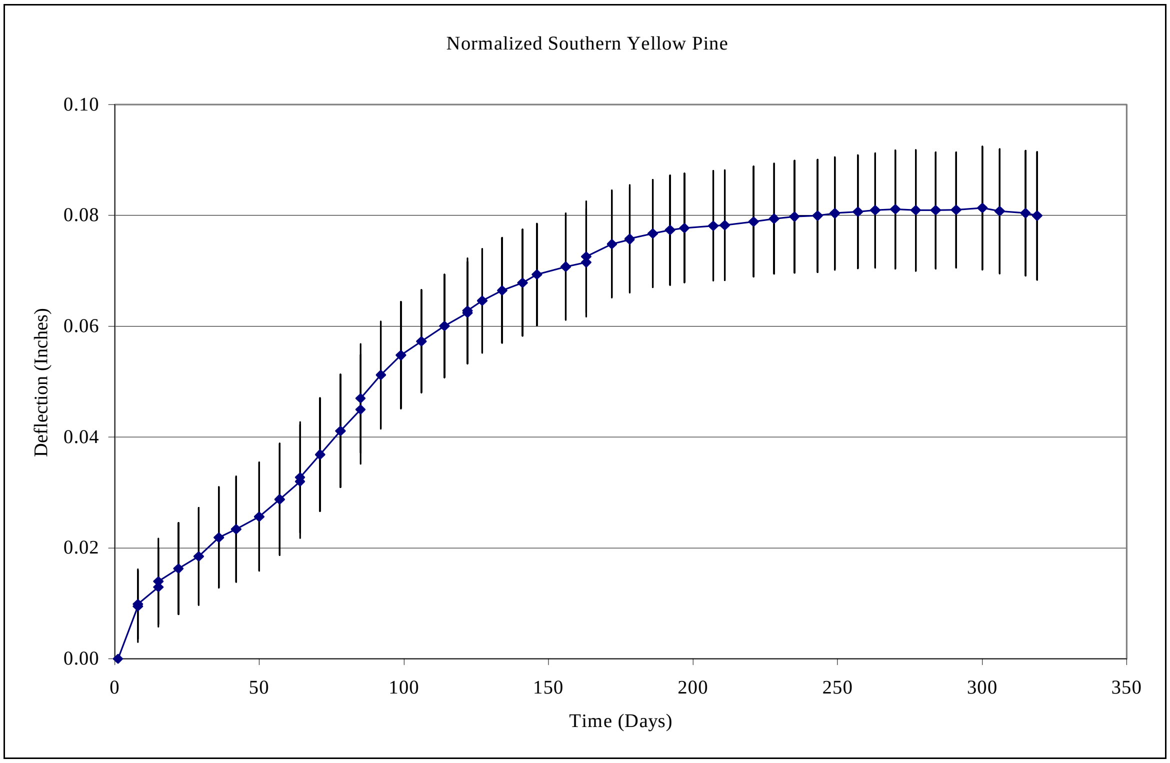


Figure 4-10 Southern Yellow Pine Mean Joint Deflection verses Time

4.3.2. Moisture Content

The mean moisture content of the control group at the start of long term testing was 13.6%. The final mean moisture content, recorded at 318 days into the test with the moisture meter, was 8.3%. The moisture content plots are shown in Figure 4-11 and Figure 4-12.

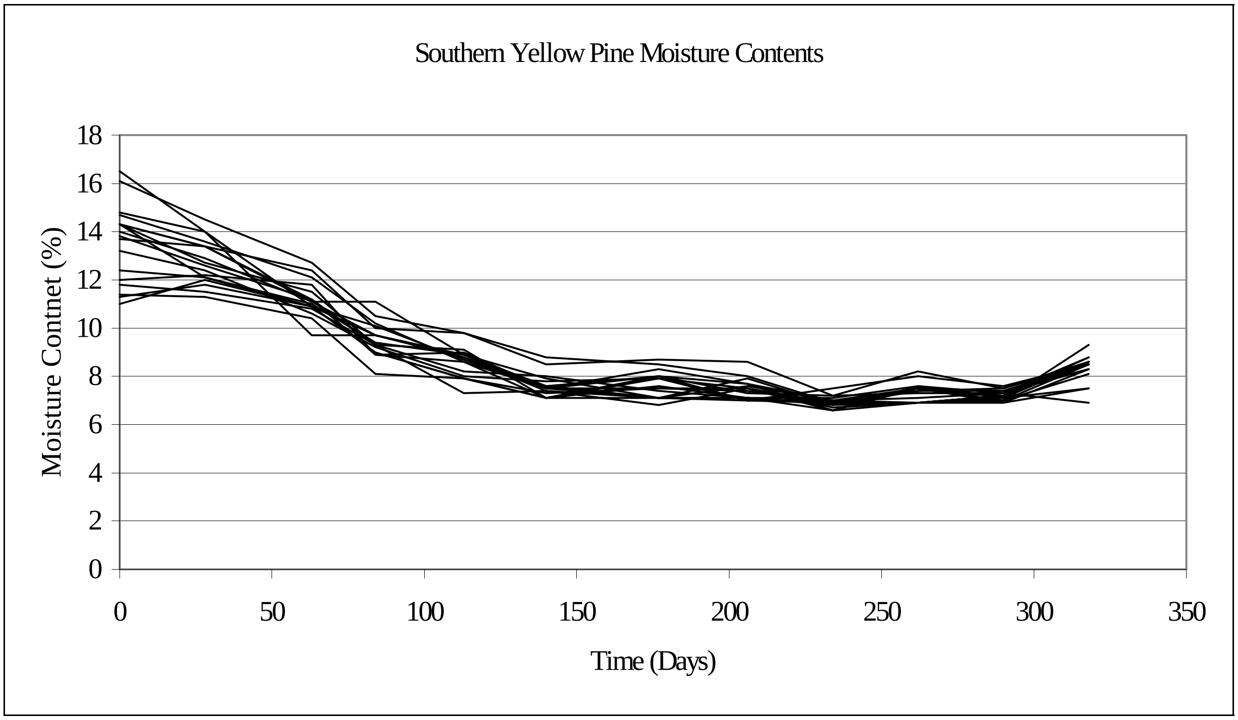


Figure 4-11 Southern Yellow Pine Moisture Content

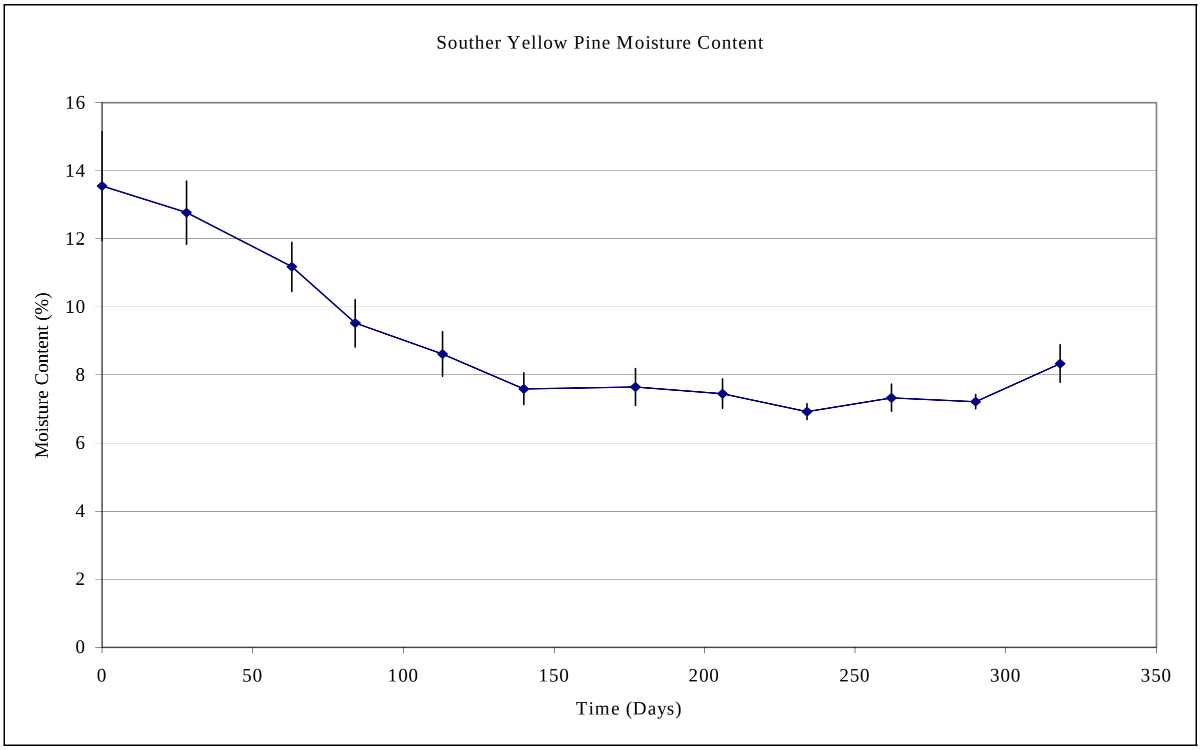


Figure 4-12 Southern Yellow Pine Mean Moisture Content

The mean moisture content increased in the final stages of load duration testing. The increase in southern yellow pine moisture content is due to the increase in relative humidity of the ambient air. The spring rains caused the increase in humidity. The effect of this moisture content increase is visible in the load-deflection plot. The increase in moisture content resulted in a slight swelling of the mortise members. The swelling of the mortise members slightly decreased the apparent deflection of the joints.

4.3.3. Results and Conclusions of Time-Deflection Behavior

Three observations are possible from the joint deflection data. The first observation is the effects of drawboring on joint deflection; Figure 4-13 illustrates these results. The creep behavior of the long-term drawbored and non-drawbored joints was approximately the same, but the initial deflection was less with the drawbored joints. This trend of less initial deflection was observed in Douglas fir testing and is repeated here.

Secondly the overall deflection was less when compared to the Douglas fir joints, the low initial moisture content and small change in moisture content explain why the deflection was less with the southern yellow pine joints.

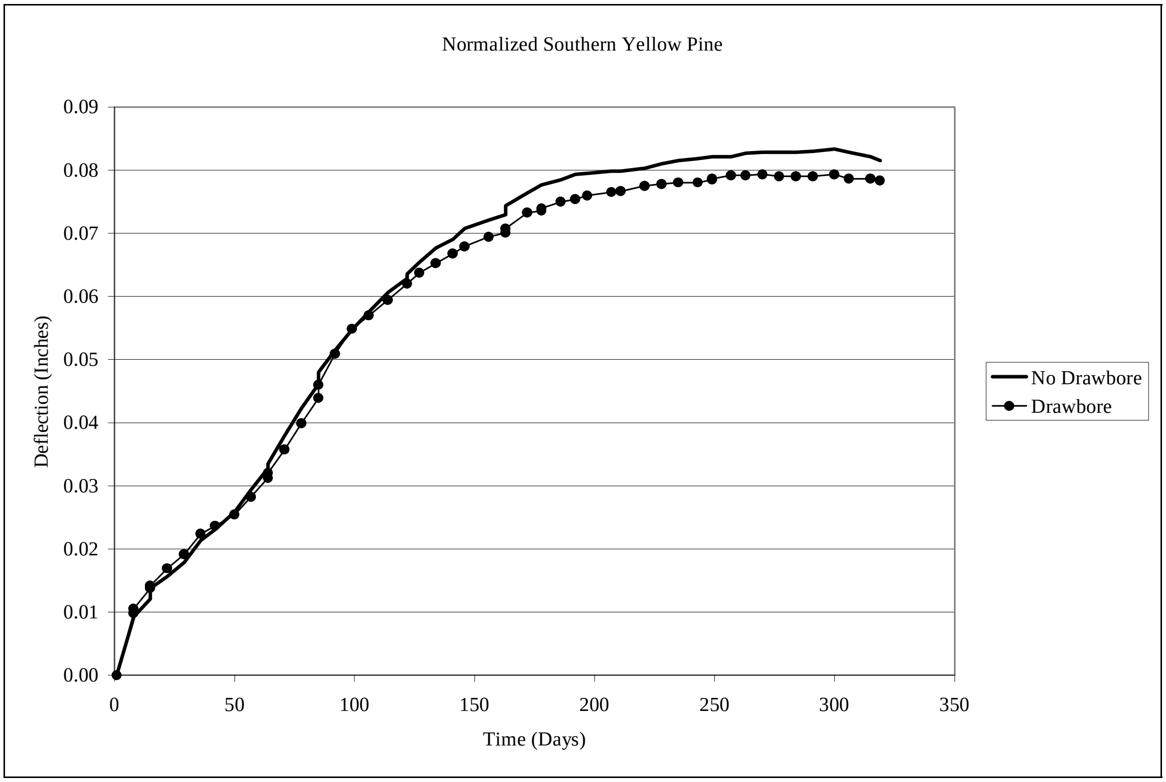


Figure 4-13 Southern Yellow Drawbore Comparison

Third, the effects of long-term load and peg diameter can be observed in the results of the southern yellow pine joint testing. A load of 2000 lb was applied to six joints and a load of 1000 lb was applied to six joints. Three from each of these load groups were drawbored. Pegs with ¾ inch diameter were used in the three joints that were drawbored with 1000 lb load. Figure 4-14 contains plots of the mean deflection of each of the three joint groups. These ¾ inch pegs showed different long-term behavior than the one inch pegs in that the deflection slowed earlier when compared to the one inch pegs. Load magnitude had only a small effect on long-term deflection, with the greater load producing slightly greater deflection.

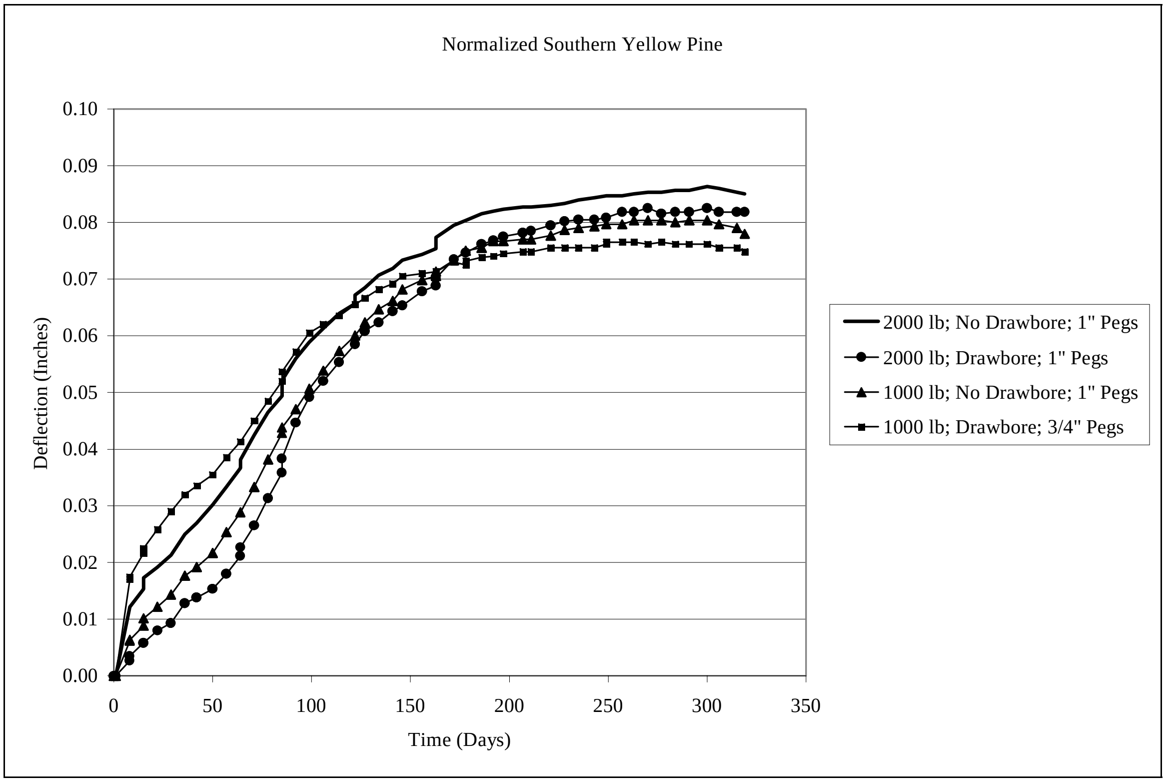


Figure 4-14 Southern Yellow Pine Comparisons

Additional general conclusions concerning the effects of drawboring, load magnitude, and peg diameter are made at the conclusion of this chapter, after a comparison of each species is made.

4.4. White Oak

White oak load duration testing was conducted on 28 joints. Variables in the white oak joints included load magnitude and the use of both white oak pegs as well as steel rods as fasteners. Joint loads were 1000 lb and 2000 lb. The average yield load reported by Schmidt and Daniels (1999) for Red Oak joints with 1” white oak pegs was 7330 lb. The loads applied to the load duration joints are 27% and 14% of this value for the 2000 lb and 1000 lb loads respectively. The joints were fabricated with 2.0D edge distance, 2.0D end distance and 2.5D spacing. Schmidt and Daniels (1999) established these end, edge and spacing distances as minimum values.

4.4.1. Loading and Load Duration

Fourteen of the 24 white oak joints were loaded for 237 days. Eight joints were loaded at 2000 lb; six joints were loaded to 1000 lb. Two of the 2000 lb joints were constructed with 1” steel rods in place of the typical white oak pegs. The 1” steel rods were used in an attempt to isolate base material behavior from peg behavior. A joint parameter table showing the joint numbers, fastener type, fastener diameter, and loading is given in Table 4-3. None of the white oak joints were drawbored.

Table 4-3 White Oak Long-Term Joint Parameters

Joint Number	Long Term Load (lb)	Drawbore	Peg Dia. (In)
WO21	2000	No	1
WO22	2000	No	1
WO23	2000	No	1
WO24	1000	No	1
WO25	1000	No	1
WO26	1000	No	1
WO27	2000	No	1
WO28	2000	No	1
WO29	2000	No	1
WO30	1000	No	1
WO31	1000	No	1
WO32	1000	No	1
WO33	0	No	1
WO34	0	No	1
WO35	0	No	1
WO36	0	No	1
WO37	0	No	1
WO38	0	No	1
WO39	0	No	1
WO40	0	No	1
WO41	0	No	1
WO42	0	No	1
WO43	2000	No	1 (Steel)
WO44	2000	No	1 (Steel)

Several of the tenons on the white oak specimens developed splits during the testing period; 19 of the 24 joints had some type of visible tenon damage during the long term testing period. The damage did not appear to be entirely the result of loading. Of the 14 joints that were loaded, 13 had tenon damage; six of the ten unloaded joints also had tenon damage. Hence, the tenon damage was more a result of shrinkage than of loading.

Damage to the tenons occurred because of differential shrinkage. The shrinkage between the two pegs is greater in the tenon than the mortise. The distance change in the tenon is due primarily to radial shrinking while the distance change in the mortised member is due to longitudinal shrinkage. The differential shrinkage therefore results in splitting of the tenon. All of the tenon damage was behind a peg or in the center of the tenon. Table 4-4 is a summary of white oak tenons that cracked during long term testing.

Table 4-4 White Oak Tenon Damage during Long Term Testing

Joint Number	Tenon Split	
	Behind One Peg	Behind Two Pegs
WO 21	x	
WO 22		x
WO 23	x	
WO 24	x	
WO 25	x	
WO 26		x
WO 27	x	
WO 28	x	
WO 29	x	
WO 30		
WO 31	x	
WO 32	x	
WO 33		
WO 34	x	
WO 35		x
WO 36		
WO 37		
WO 38		
WO 39	x	
WO 40		x
WO 41	x	
WO 42	x	
WO 43	x	
WO 44	x	

White oak joint WO21 had severe tenon damage from the long term loading. A split behind one peg developed into a block shear failure (relish failure). The tenon split was first observed approximately three weeks into the long term test; the split was noted after the joint showed considerable deflection in comparison to the other white oak joints. This relish failure substantially reduced the stiffness of the joint, since only one of the two pegs was active in carrying the 2000 lb load. Specimen WO 21 had a final deflection of 0.490", nearly twice as much any other white oak joint.

Deflection verses times curves for the 14 loaded joints are shown in Figure 4-15. Figure 4-16 shows the normalized data. Figure 4-17 is a plot of mean joint deflection with one standard deviation of all the loaded joints. The mean deflection of all the loaded joints at the conclusion of the load duration testing was 0.194". Unlike the Douglas fir and southern yellow pine joints, the mean deflection was still increasing at a steady rate when the test was stopped. An accurate prediction of if and when the joint deflection would have stopped can not be made. The joints were just at the point in time where the Douglas fir and southern yellow pine joint deflection had slowed or stopped, about 225 days. The normalized plot (Figure 4-16) of deflection verses time indicates that the white oak joint deflection did vary once the long-term test was started. This result is different than that of the Douglas fir joints, which showed little variance in deflection after the start of long-term testing.

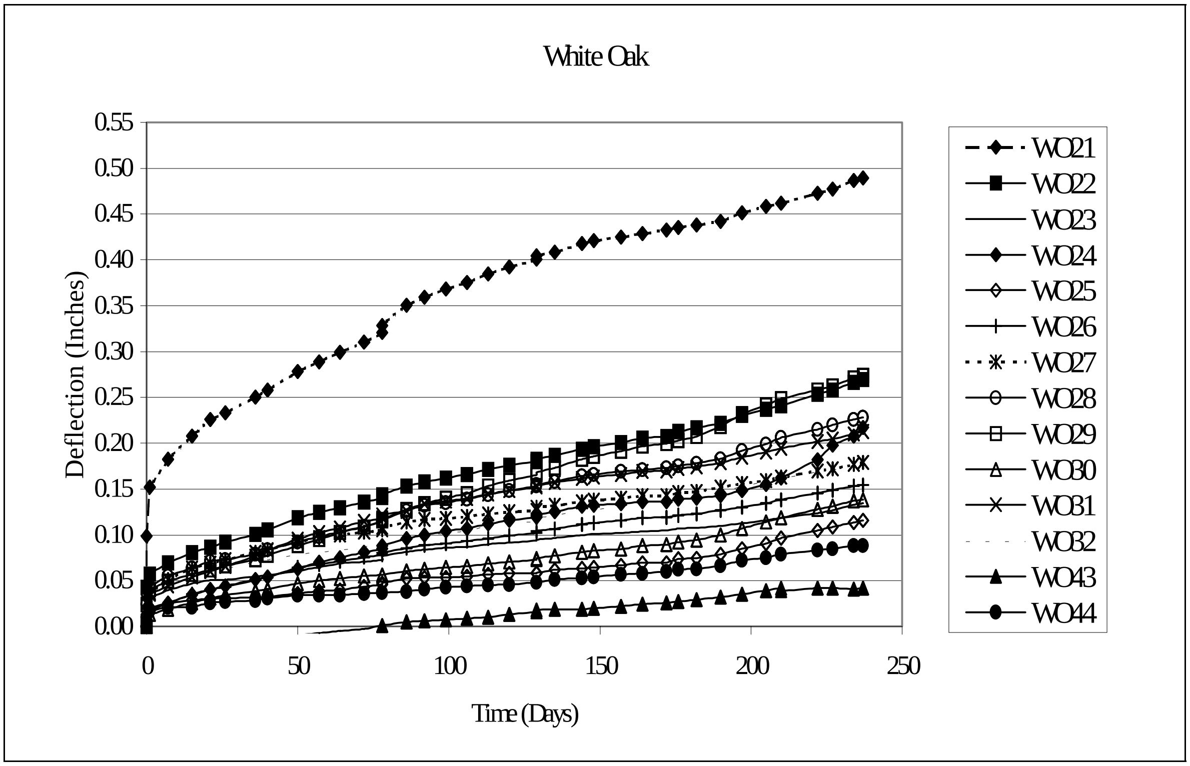


Figure 4-15 White Oak Joint Deflection verses Time

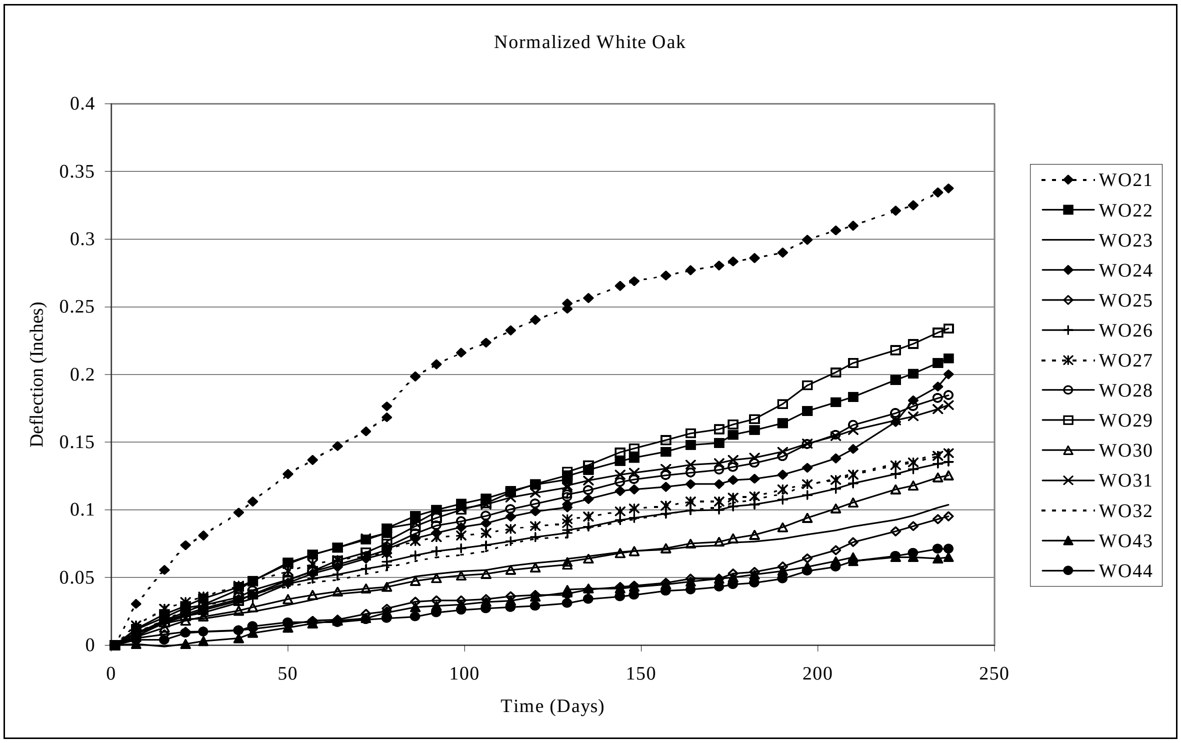


Figure 4-16 Normalized White Oak Deflection versus Time

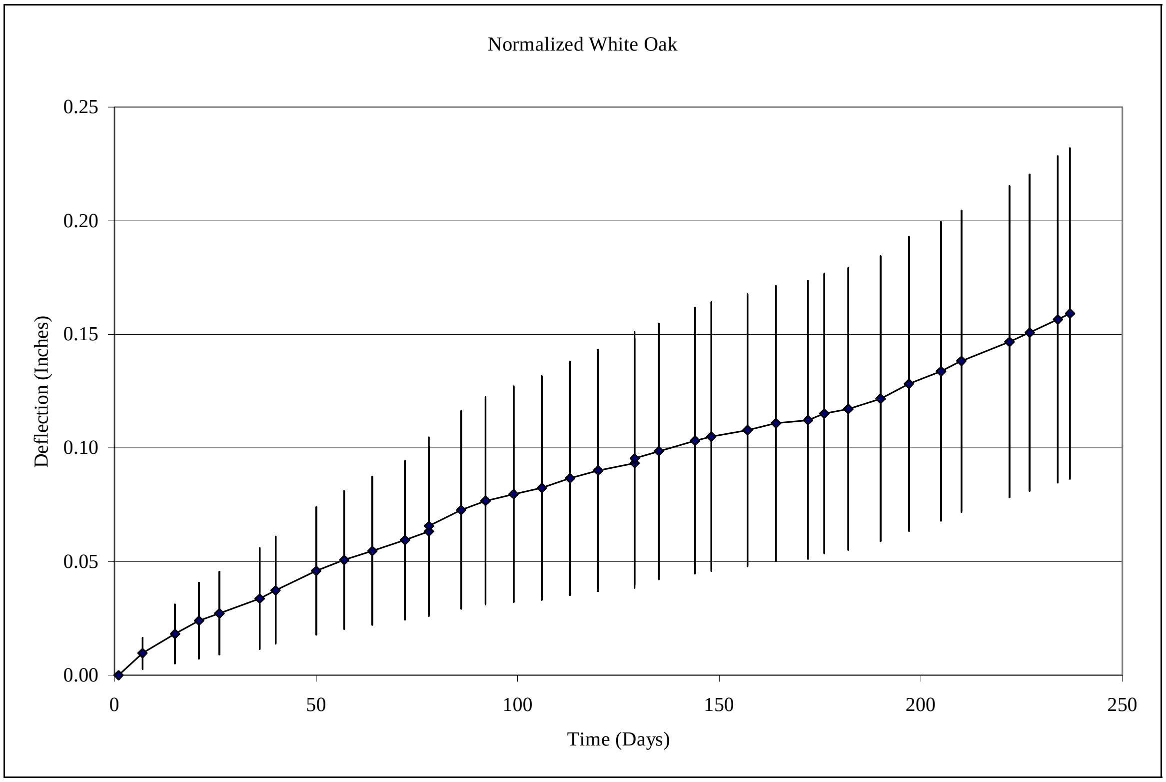


Figure 4-17 White Oak Mean Joint Deflection verses Time

4.4.2. Moisture Content

The mean moisture content of the white oak joints at the start of long term testing was the highest of any species tested. The moisture content at the start of testing averaged 33.0%. Approximately two months time passed between the time the joints were received and when testing started. During this time the joints were kept in an environmental conditioning chamber that had a high relative humidity. The objective was to prevent shrinkage of the members prior to their assembly into joints. The joints could then be loaded while they were green, so seasoning effects could be investigated. The conditioning chamber worked well; the joints remained above their fiber saturation point. The final moisture content reading of the control joints was taken at 221 days into the test. The mean moisture content at that time was 15.5%. Due to local seasonal weather conditions, the relative humidity was higher than normal while these joints were under load. A dehumidifier was used during the last 30 days of the load duration test to lower the relative humidity of the ambient air in order to speed up the seasoning (drying) process. Plots of moisture contents verses time (Figure 4-18) and mean moisture content verses time (Figure 4-19) are shown below.

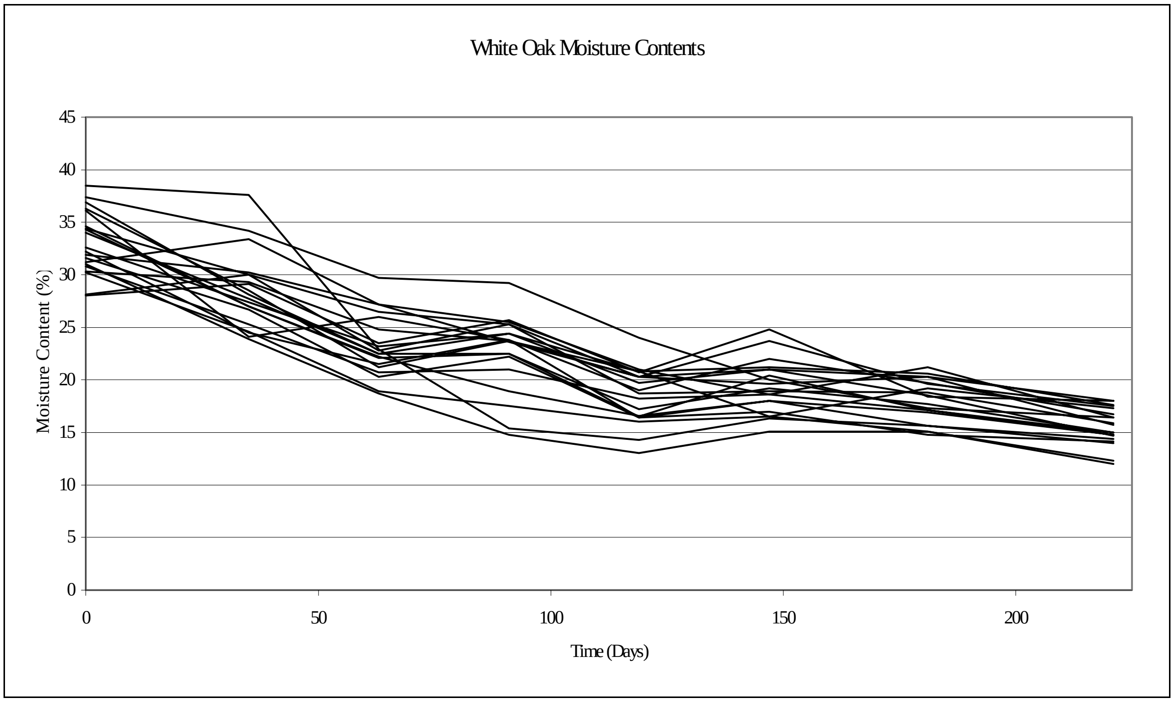


Figure 4-18 White Oak Moisture Content

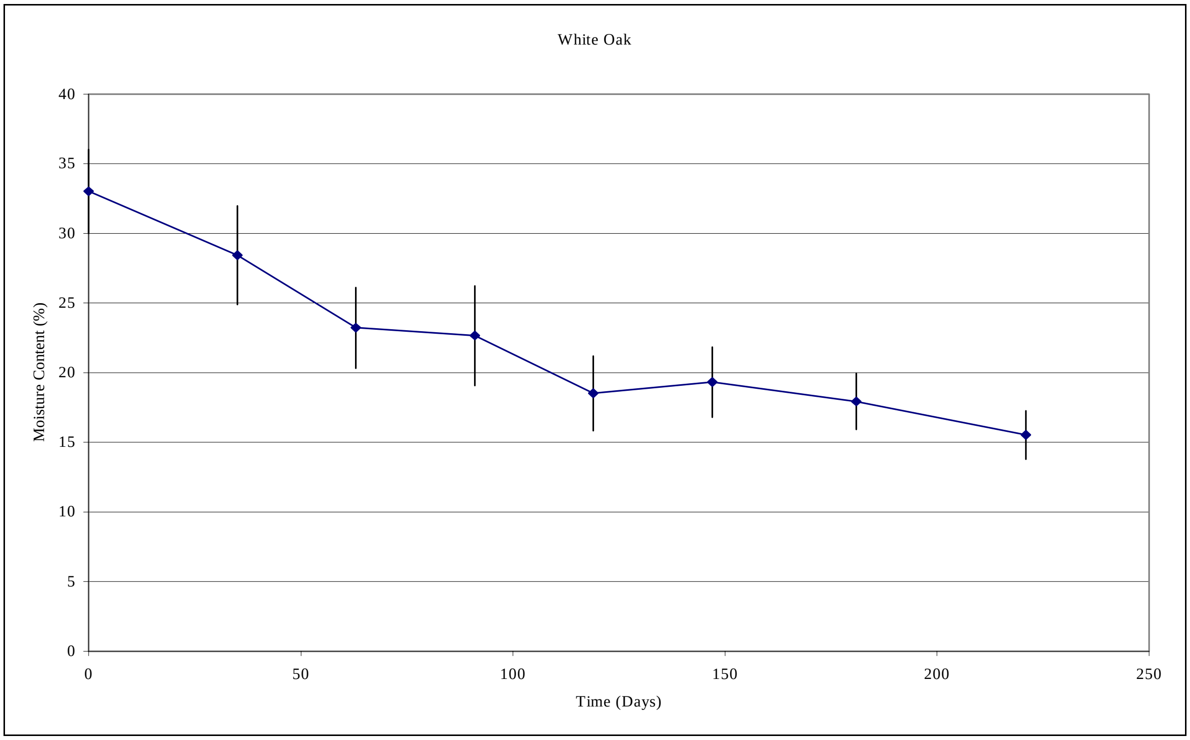


Figure 4-19 White Oak Mean Moisture Content

4.4.3. Results and Conclusions of Time-Deflection Behavior

The primary observation that can be made from analyzing the time-deflection plot is that the deflection had not stabilized in the 237 days of testing. Additional conclusions regarding the white oak load duration behavior can be made from examination of Figure 4-20. Figure 4-20 shows the normalized deflection behavior separated by load magnitude and type of fastener (1” white oak peg or 1” steel rod).

The two joints that were constructed with 1” diameter steel rods in place of white oak pegs had significantly smaller deflections than the joints that were loaded to 1000 or 2000 lb. The joint with steel rods had a negative deflection (-0.006”) at the start of testing. The negative value is due to the fact that the joints were fitted with only one dial gauge on the bottom side of the joint. When the joint was loaded the tenon member rotated slightly in the mortise, resulting in the bottom side of the tenon moving in towards the mortise. This effect was minor to the long-term behavior of the joint.

As expected the joints with 1000 lb loading had less deflection than the joints with 2000 lb loading. For joints with wood peg fasteners, the initial deflection of the 1000 lb joints was roughly half of that for the 2000 lb joints. As shown in Figure 4-20, the 1000 lb joints also experience about 25% less long-term deflection than those loaded to 2000 lb.

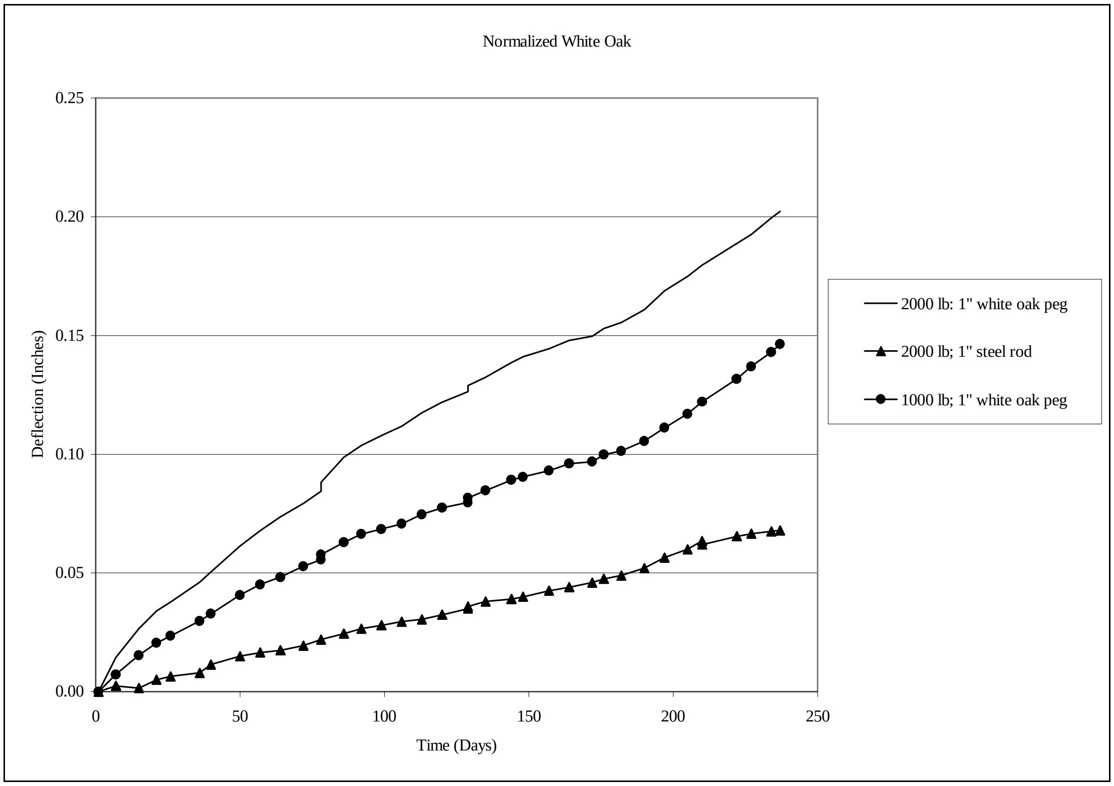


Figure 4-20 Normalized White Oak Comparison

4.5. Eastern White Pine

The final long-term test joints were constructed of eastern white pine. Twenty-eight joints were tested with 16 joints of this total loaded for 242 days. All of the joints were loaded to 1000 lb which is 16% of the mean yield value for joints with 1” pegs and 34% of the mean yield value for joints with ¾” pegs. The yield values that the previous numbers are based on are yield values for joints in which the pegs failed. The detailing distances were 4.0D edge distance, 4.0D end distance and 3.0 inches spacing. The end and edge distances for the joints with 1” pegs required the tenon member to be altered. The required tenon length for these joints was eight inches. Since the joint specimens were delivered with six-inch long tenons, the tenon shoulders were cut back an additional two inches as a part of joint preparation and assembly.

4.5.1. Loading and Load Duration

Peg diameter was the primary variable in the eastern white pine joints. Thirteen joints were constructed with ¾” white oak pegs and twelve joints with 1” white oak pegs. Steel rods replaced 1” white oak pegs in three of the loaded joints. None of the eastern white pine joints were drawbored. This test sequence is summarized in Table 4-5.

Table 4-5 Eastern White Pine Long-Term Joint Parameters

Joint Number	Long Term Load (lb)	Drawbore	Peg Dia. (In)
EWP21	1000	No	0.75
EWP22	1000	No	0.75
EWP23	1000	No	0.75
EWP24	1000	No	1
EWP25	1000	No	1
EWP26	1000	No	1
EWP27	1000	No	1 (Steel)
EWP28	1000	No	1 (Steel)
EWP29	1000	No	1 (Steel)
EWP30	1000	No	0.75
EWP31	1000	No	0.75
EWP32	1000	No	0.75
EWP33	1000	No	1
EWP34	1000	No	1
EWP35	1000	No	1
EWP36	0	No	0.75
EWP37	0	No	0.75
EWP38	0	No	0.75
EWP39	0	No	1
EWP40	0	No	1
EWP41	0	No	1
EWP42	0	No	1
EWP43	0	No	1
EWP44	0	No	1
EWP45	0	No	0.75
EWP46	0	No	0.75
EWP47	0	No	0.75
EWP48	1000	No	0.75

Joint EWP48 was first tested in the short-term testing discussed in Chapter 2. This joint was labeled as EWP09 in the earlier testing; the joint was included in the loaded group for long-term testing. The joint is also of interest due to the fact that the moisture content (9%) at the start of testing was much lower than the mean of the remaining joints (28%). This joint was not included in the mean joint deflection plot (Figure 4-23) because of the difference in initial moisture content. The moisture content of the joint will help separate the effects of moisture content from long-term load effects. However, with only one joint it is difficult to generalize any trends that could be observed from the joint. Note that joint deflection data is included in Figure 4-21.

Observation of the time-deflection plots (Figure 4-21, Figure 4-22 and Figure 4-23) show that the creep rate of the joints remained steady through the end of the long-term joint tests. This behavior was similar to that of the white oak joints. The creep of the Douglas fir and southern yellow pine joints had stopped at approximately 225 days. The white oak and eastern white pine joints continued to creep after the 225-day mark. The normalized time-deflection plot indicates that a sizable portion of the variation in the deflection was due to initial deflection. Joint EWP 35 showed a greater amount of deflection over the course of the long-term testing. This behavior is most likely due to knots in the mortise member and a check that developed between the peg holes.

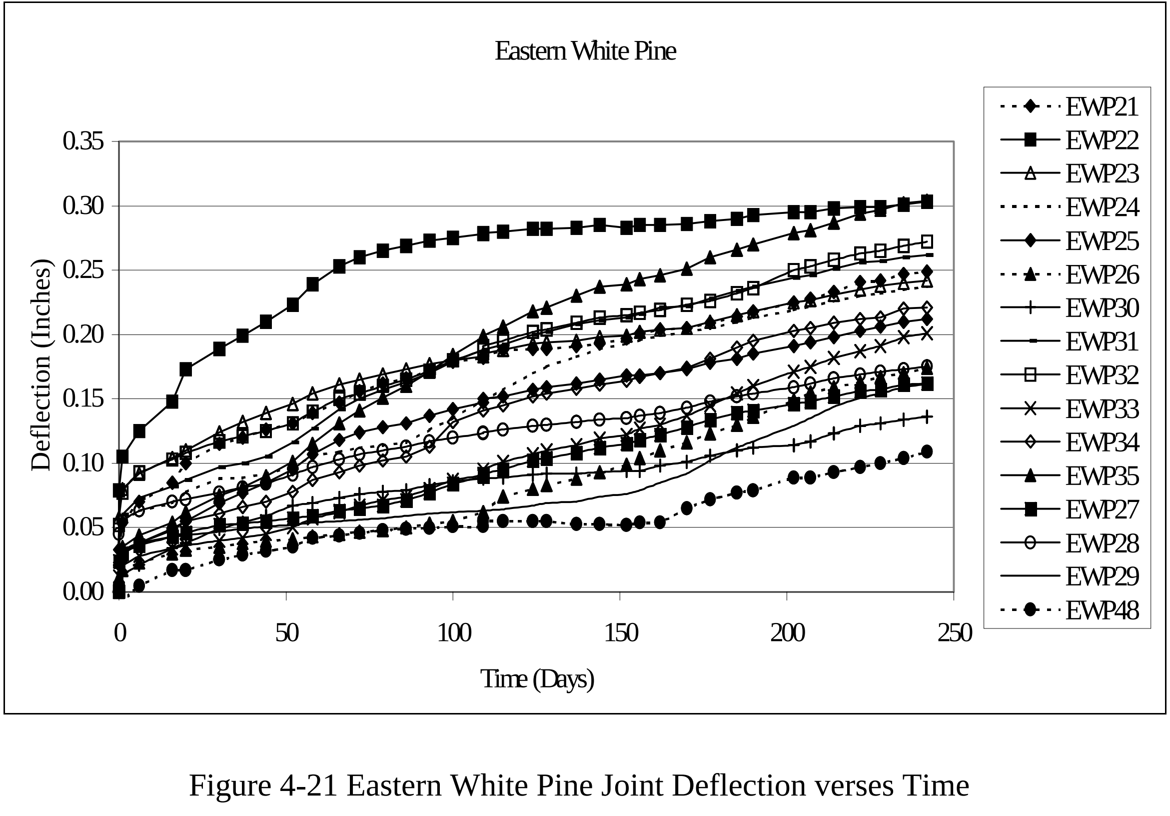


Figure 4-21 Eastern White Pine Joint Deflection verses Time

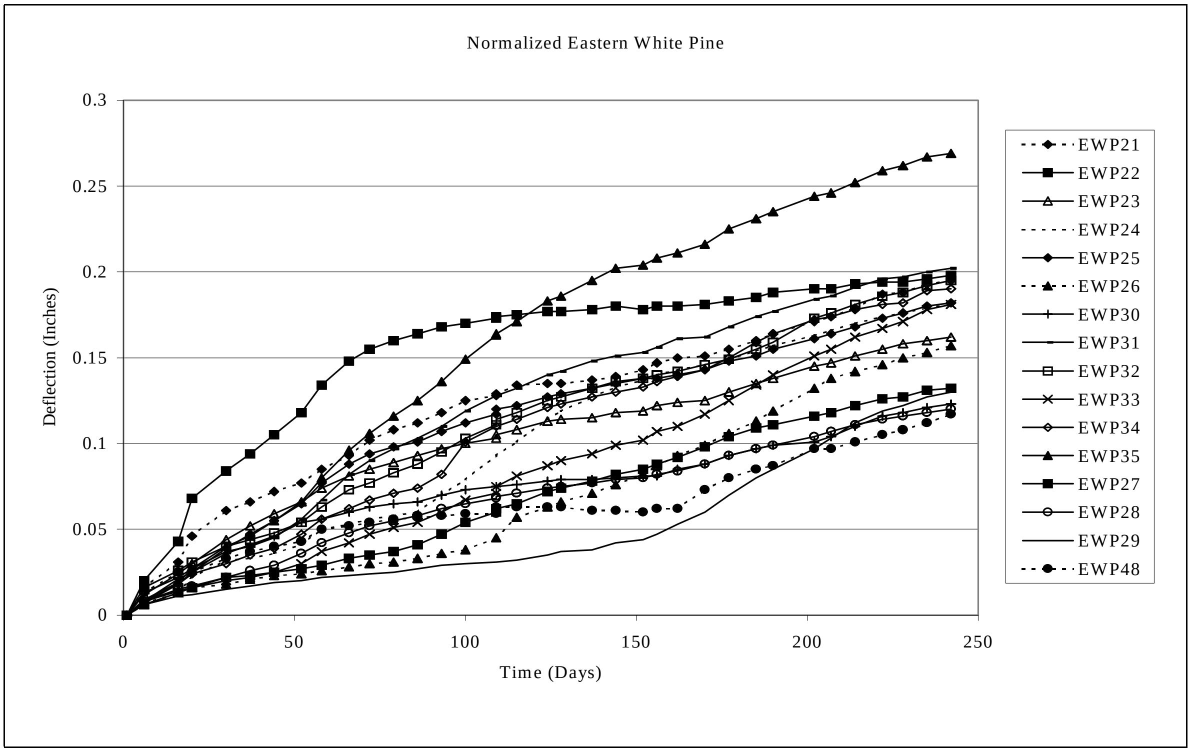


Figure 4-22 Normalized Eastern White Pine Deflection versus Time

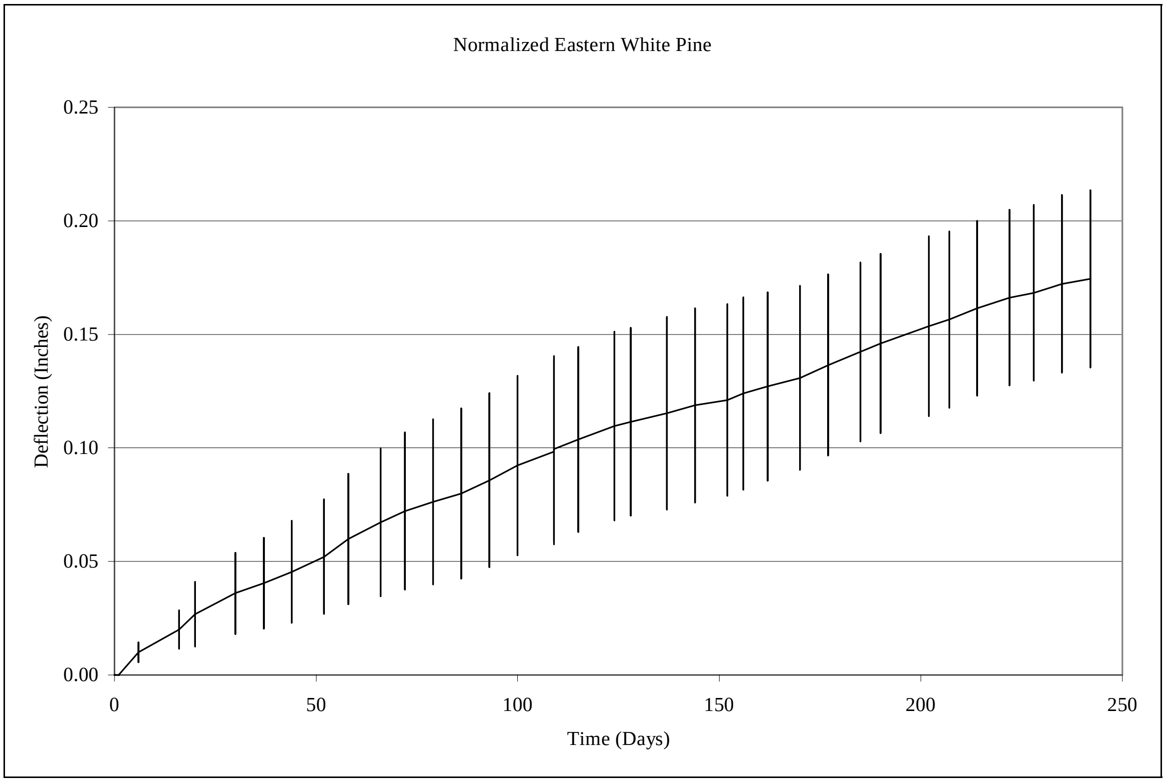


Figure 4-23 Eastern White Pine Mean Joint Deflection verses Time

4.5.2. Moisture Content

The eastern white pine joints started with a high moisture content of 28%. The joints had dried to a moisture content of 7% by the conclusion of testing. A dehumidifier was used in the final 45 days to assist in the drying process. Joint EWP48 is not included in the eastern white pine moisture calculations because it was used in prior testing and had seasoned prior to the start of long-term load testing. This joint was the recycled joint from the short term testing discussed earlier. Plots of the moisture content and mean moisture content of the eastern white pine joints are shown below in Figure 4-24 and Figure 4-25.

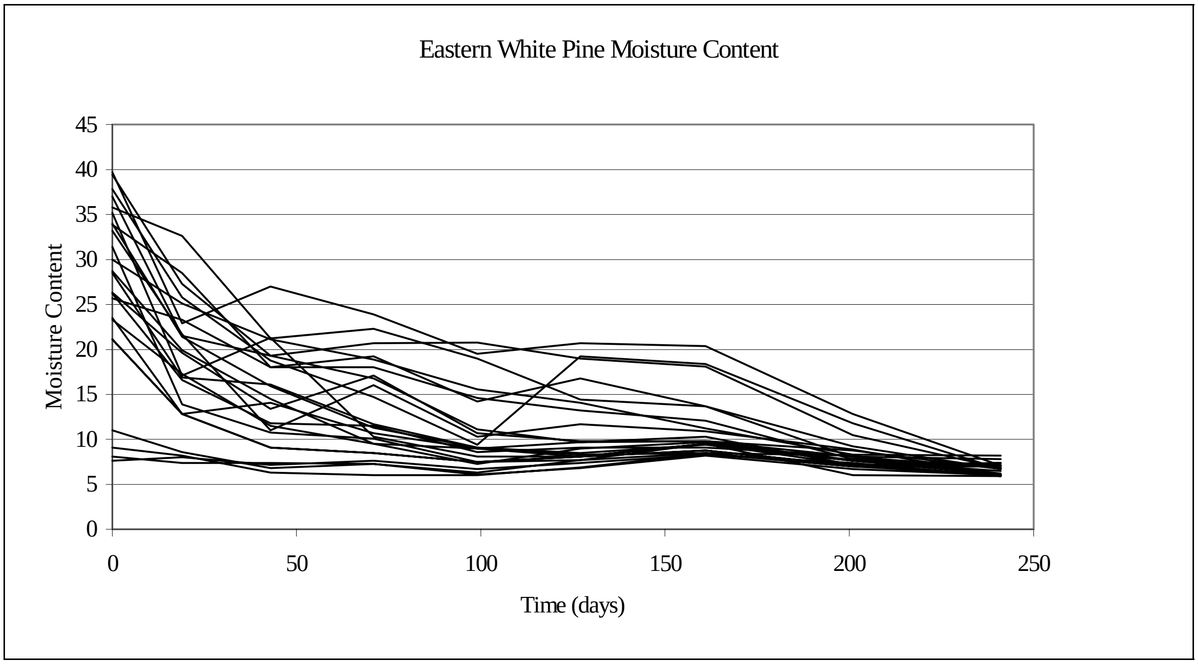


Figure 4-24 Eastern White Pine Moisture Content

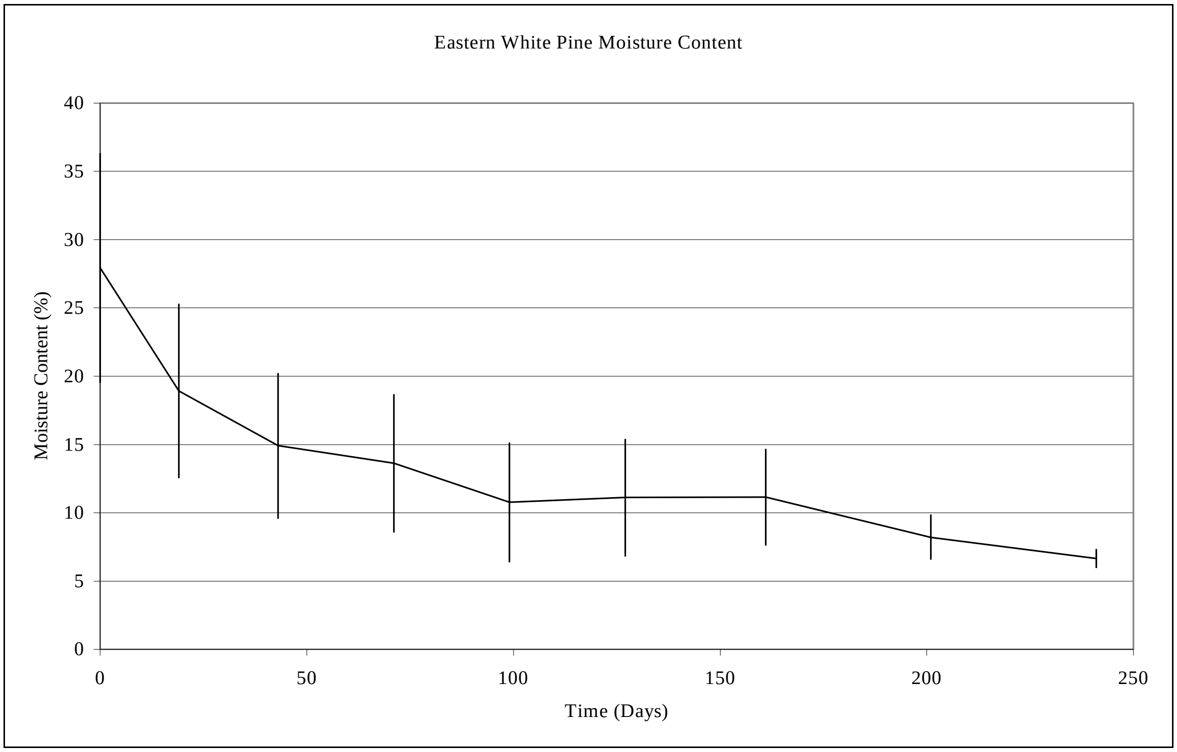


Figure 4-25 Eastern White Pine Mean Moisture Content

4.5.3. Results and Conclusions of Time-Deflection Behavior

The peg diameter and the effect of replacing 1” white oak pegs with steel rods are the variables in the eastern white pine joints. The peg diameter had only a minor effect on the joint deflection; the slopes of the time-deflection plots were within 11% of each other. The joints with the steel rods did have less deflection than the joints with white oak pegs. Figure 4-26 is illustrative of these effects. The difference in deflection rate was 29%, with respect to the white oak pegs, from the 1” white oak pegs to the 1” steel rods.

Joint EWP29 was the only eastern white pine joint which had a tenon split at the end of long term testing. The split appeared to coincide with a small knot and was not believed to be load related.

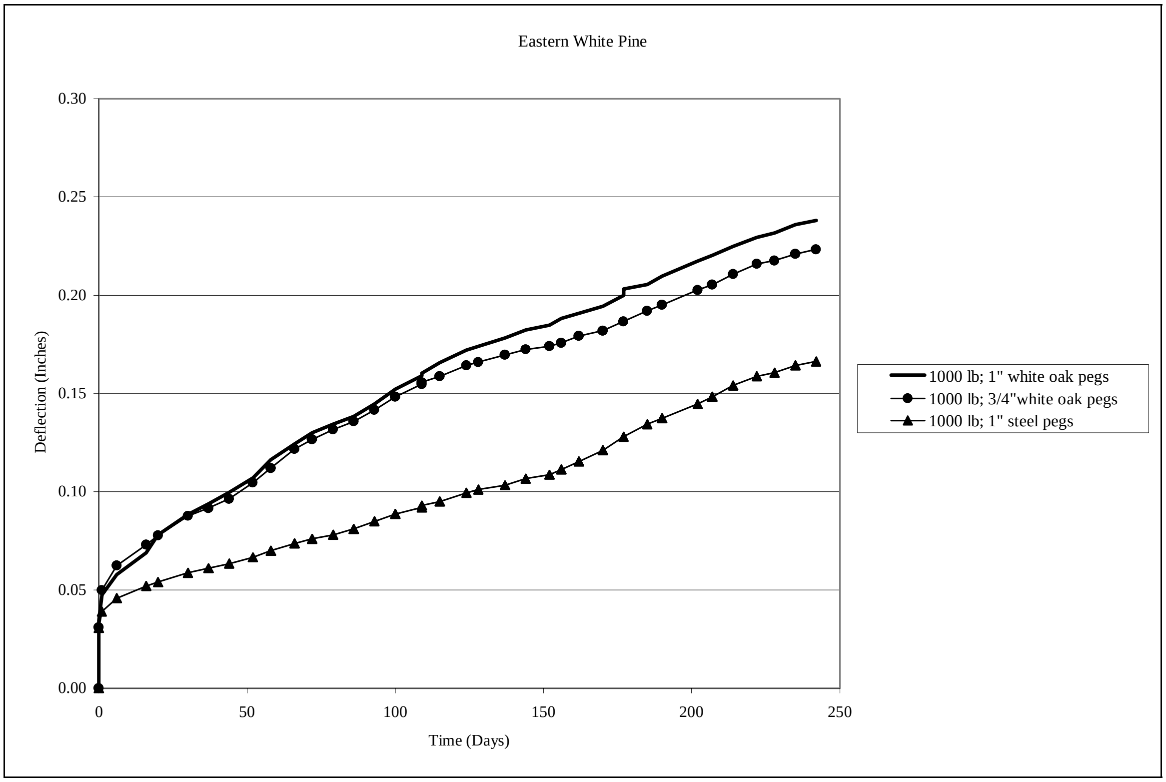


Figure 4-26 Eastern White Pine Comparison

4.6. General Long-Term Conclusions

Conclusions that can be drawn from the load duration testing are:

1. Drawboring does reduce deflection due to initial load; this was true in both the Douglas fir and southern yellow pine joints.
2. The drawboring process resulted in tenon splitting during assembly of some of the joints. Use of a longer tenon helped reduce tenon damage. It is possible that 1” diameter pegs are too stiff to drawbore safely. Smaller diameter, more flexible pegs are expected to cause less tenon damage and permit larger tolerances for fabrication. More testing is needed to validate this conclusion.

3. Drawboring also reduces the long-term creep/shrinkage deflection of the joint. This is particularly true for the Douglas fir joints. The behavior is not so evident for the southern yellow pine joints, possibly because the wood was already partially seasoned before the joints were assembled.
4. By itself, peg diameter had a minimal effect on the long-term deflection of the joints. Other influences, such as load level and use of drawboring play more significant roles than peg diameter.
5. Steel rods used as fasteners reduced both the initial deflection and the long-term deflection of the joints. The joints with steel rods were much more rigid than those with the white oak pegs. The increased rigidity has two effects. The first is that there is virtually no deflection of the steel rod itself. The second effect is that the rigid steel evenly distributes load across the thickness of the connected members. Therefore, the stress is distributed much more evenly through the joint than with a less rigid wooden peg. A more flexible wooden peg causes a higher bearing stress in the mortise and tenon members near the mortise-tenon interfaces. Hence, greater localized deflections due to dowel bearing action can be expected in these joints.
6. Tenon damage may result from differential shrinkage of the tenon relative to the mortise. This damage to the tenon is independent of load magnitude or duration.

5. Failure Testing of Long Term Specimens

5.1. Test Procedure/Analysis

To determine the effects of long-term loading on joint stiffness and strength, short-term monotonic load tests were performed on all of the joints in the load duration study. Loaded and control groups are compared; this comparison further reveals load duration effects on mortise and tenon joints. The test procedure was similar to that used earlier in this research in the testing of eastern white pine. The 5% offset method was used to find the yield values.

5.2. Douglas Fir

Twelve Douglas fir joints were tested. The Douglas fir joints all used 1” diameter white oak pegs. The test joints can be divided into four groups, combinations of loaded, unloaded, drawbored and not drawbored. Three joints were in each group.

5.2.1. Joint Properties and Results

The mean yield strength of the joints was 6120 lb, slightly higher than the mean yield strength of 5700 lb found by Schmidt and Daniels (1999) on tests of recycled Douglas fir. The mean of the loaded joints was 10% lower than the mean of the unloaded joints; 5800 lb compared to 6450 lb. Drawboring did not seem to have a significant effect on the joint strength. Joints DF27 and DF29 had tenon damage due to drawboring. Joint DF29 had the lowest yield value (4980 lbs) of any of the Douglas fir joints tested. Yet, the yield value for DF27 was 6710 lb, compared to the mean value of 6120 lb. A complete table of the yield strengths, stiffnesses and long term loading conditions for the Douglas fir joints is given in Appendix A.

The long-term load did not appear to affect the ductility of the joints. The mean displacement at ultimate load for the loaded joints was 35% greater than that of the unloaded joints, 0.229” compared to 0.170”. Not all of the Douglas Fir joints showed good ductility, several joints did not hold load after yield. However, this behavior was not limited to joints that were under long-term load. A Douglas Fir load-deflection plot is shown in Figure 5-1.

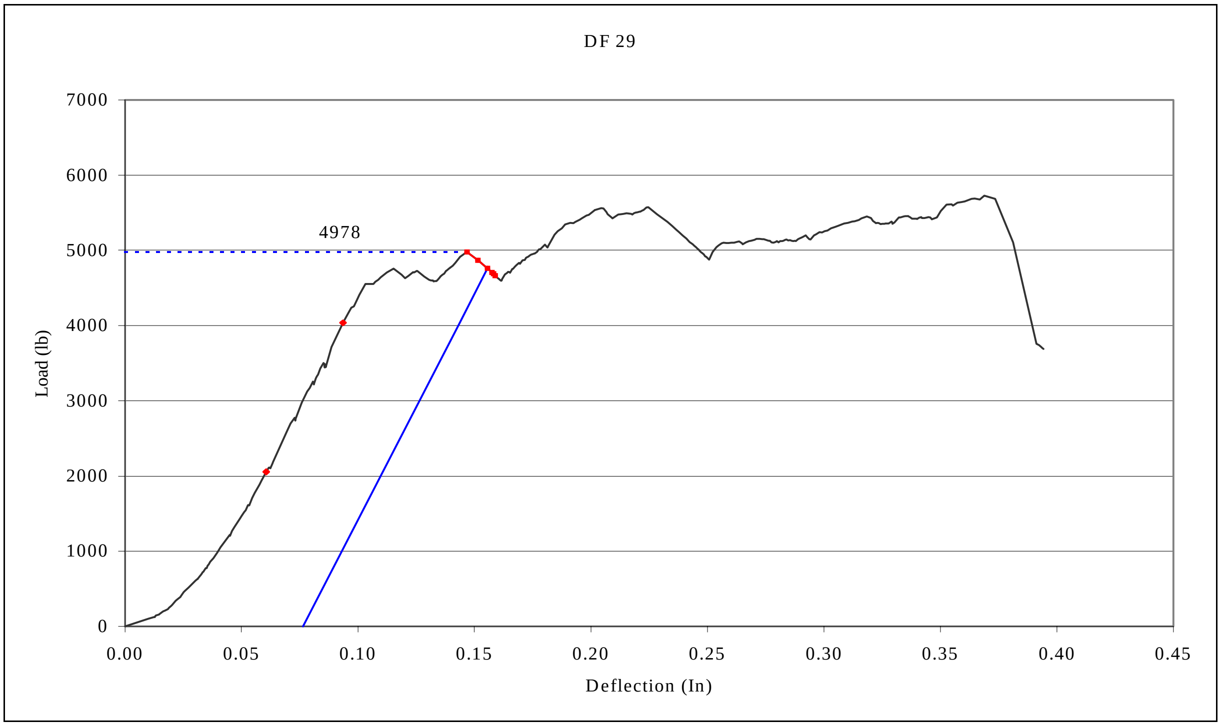


Figure 5-1 Douglas Fir Joint Test

5.2.2. Material Properties (Dowel Bearing Strength and MC)

The joints were disassembled after testing and two dowel bearing test samples were cut from each mortise member and each tenon member. The mortise member dowel bearing test samples were oriented in such a way that the applied load was perpendicular to the grain of the specimen. The tenon dowel bearing specimen was oriented with load applied parallel to the grain. A table of the dowel bearing test results is in Appendix A. That data is summarized in Table 5-1.

Table 5-1 Douglas Fir Dowel Bearing Test Summary

Mortise Samples (24 specimens)		
Statistic	Yield Strength (lb/in^2)	Stiffness (lb/in^3)
Mean	2,730	32,500
St. Dev.	630	12,750
5% Exclusion	1,530	8,260
COV	0.231	0.392
K	1.901	1.901

Tenon Samples (24 specimens)		
Statistic	Yield Strength (lb/in^2)	Stiffness (lb/in^3)
Mean	6,820	162,300
St. Dev.	690	21,550
5% Exclusion	5,500	121,330
COV	0.101	0.133
K	1.901	1.901

Moisture content was taken from each mortise member and each tenon member; the mean moisture content for the Douglas fir joints was 8.9%. The reported values are not adjusted for moisture content. Specific gravity tests were also performed on a sample taken from the member. The mean specific gravity of the Douglas fir joints was 0.478. A complete listing of each member's specific gravity is given in Appendix A.

5.3. Southern Yellow Pine

Twenty-one southern yellow pine joints were tested. The variables of the test joints included peg diameter, load magnitude and drawbore. Test joints had ¾" and 1" diameter pegs white oak pegs, the joints were loaded at 1000 and 2000 lb. The joints that were constructed with drawbore had the same amount of drawbore as the Douglas fir joints, 3/32".

5.3.1. Joint Properties

The mean yield strength of the joint tests was 7090 lb. The mean strength found by Schmidt and Daniels (1999) of comparable southern yellow pine joints was 4960 lbs.

The difference between these is likely due to the differences in moisture content of the joints at the time of testing. For this research, the joints averaged 8.3% moisture content, whereas those tested by Schmidt and Daniels averaged 17%.

The southern yellow pine joints without load did have a higher mean strength than the joints loaded to 2000 lb, 7580 lb compared to 6640 lb. The joints with 1000 lb loading were in between the two groups at 7040 lb. The stiffness of the joints with 2000 lb loading was 14% higher than the joints loaded at 1000 lb and 19% higher than the unloaded joints. The load deflection plot of the joints with 2000 lb loading was extremely steep up to 2000 lb of load. If the remainder of the plot was not linear, the stiffness reported is from the linear portion of the plot from 0 to 2000 lbs. The joints tended to soften after the 2000 lb load point.

Drawboring did not have a significant impact on joint strength. The mean strength of the loaded joints with 1" diameter pegs was slightly lower than that of the unloaded joints. The 1" diameter peg joints with 2000 lb long-term loading had the lowest mean yield value of the 1" diameter peg group, the joints with 1000 lb long-term load were slightly higher, and the joints without long-term load had the greatest mean yield value of the 1" diameter peg group. The joints with ¾" diameter pegs did not follow the trend of the 1" diameter pegs. However the total number of joints with ¾" diameter pegs was limited to six. The yield values of the joints were scattered; the loaded joints had a higher mean yield value than the unloaded joints. The long-term load did not seem to have a negative effect on the joint strength. A table of each joint's yield strength and stiffness along with mean values for loaded and unloaded and drawbored and non-drawbored joints is included in Appendix B.

5.3.2. Material Properties (Dowel Bearing Strength and MC)

Two dowel bearing tests were performed on each mortise and each tenon from the southern yellow pine long-term joints. The specimens were loaded in the typical manner. A summary table is provided below, the complete results of each test are given in Appendix B. The mean moisture content of the southern yellow pine members was 9.1%; the specific gravity was 0.454.

Table 5-2 Southern Yellow Pine Dowel Bearing Test Summary

Mortise Samples (42 specimens)		
Statistic	Yield Strength (lb/in^2)	Stiffness (lb/in^3)
Mean	2,350	28,730
St. Dev.	310	5,190
5% Exclusion	1,790	19,230
COV	0.130	0.181
K	1.829	1.829

Tenon Samples (42 specimens)		
Statistic	Yield Strength (lb/in^2)	Stiffness (lb/in^3)
Mean	6,820	162,300
St. Dev.	690	21,550
5% Exclusion	5,500	121,330
COV	0.101	0.133
K	1.901	1.901

5.4. White Oak

The white oak joints performed well despite damage to many tenons during long term testing. The joint variables in the long-term white oak joints were load magnitude, and peg type. All of the joints were constructed with 1” diameter fasteners, two joints with steel rods and the remainder with white oak pegs. Tenon damage was not limited to any group of variables.

5.4.1. Joint Properties

The mean yield value of the white oak joints, excluding the two joints with steel rods, was 5860 lb. This can be compared to a mean yield value of 7330 lb of the red oak joints tested by Schmidt and Daniels (1999).

The white oak joints did not follow the trend of the Douglas fir and southern yellow pine joints. The joints loaded with 2000 lb had a higher mean yield value (6410 lb) than the joints with no long-term load (5980 lb); the joints with steel rods were excluded from this compison. The joints with a 1000 lb long term load had the lowest mean yield value of any group at 5110 lb. This result is unexpected for two reasons. First the trend of the previous two species test results was not followed. Secondly a higher percentage of loaded joints had tenon damage from the long term testing.

The two joints with 1" steel round stock used for fasteners had a much larger yield value (11,300 lb) than any of the joints with white oak pegs. Stiffness was also greatly increased with the steel rods. A brittle tenon failure occurred in both of these joints.

5.4.2. Material Properties (Dowel Bearing Strength and MC)

Dowel bearing, moisture content, and specific gravity tests were performed on the white oak joint members. The dowel bearing tests results are summarized in Table 5-3. The mean moisture content was 11.9% and the mean specific gravity was 0.678. Tables of the white oak properties are given in Appendix C.

Table 5-3 White Oak Dowel Bearing Test Summary

Mortise Samples (48 specimens)		
Statistic	Yield Strength (lb/in^2)	Stiffness (lb/in^3)
Mean	4,730	40,700
St. Dev.	860	7,110
5% Exclusion	3,170	27,810
COV	0.182	0.175
K	1.815	1.815

Tenon Samples (47 specimens)		
Statistic	Yield Strength (lb/in^2)	Stiffness (lb/in^3)
Mean	7,070	165,400
St. Dev.	1,210	32,150
5% Exclusion	4,860	106,990
COV	0.172	0.194
K	1.818	1.818

5.5. Eastern White Pine

Eastern white pine tests joints composed the remainder of the long-term tests. Twenty-eight joints were tested, 15 with 1" diameter fasteners. For these fifteen joints, twelve joints had white oak pegs and three had steel rods. The remaining thirteen joints used ¾” diameter white oak pegs for fasteners. Sixteen joints were loaded during long-term testing, all at a load of 1000 lb. One joint (EWP48) was recycled from the short-term tests discussed earlier.

5.5.1. Joint Properties

The mean strength of the eastern white pine joints with 1” diameter pegs was 5530 lb. This is 13% less than the mean yield value of 6270 lb for the two comparable tests discussed in Chapter 2. Once again the loaded joints with 1” pegs had a higher mean yield value (5310 lb) than the 1” diameter unloaded joints (4710 lb) with 1” diameter pegs.

As expected the joints with 1” diameter steel rods were stronger and stiffer than the joints with white oak pegs. Stiffness was more and double: a mean stiffness of 101,000 lb/in with 1” diameter steel rods compared to 43,600 lb/in with 1” diameter white oak pegs. The mean strength of the three joints with 1” steel rods was 7560 lb, 2500 lb higher than the mean of comparable joints with white oak pegs.

The joints with ¾” diameter pegs had the same mean strength for both the loaded and unloaded groups. Reaffirming the trend that long-term loading has no well–defined effect on joint strength.

The recycled joint (EWP48) was excluded from the calculations for mean yield value. This recycled joint had a yield value of 2710 lb, 27% lower than the mean of 3710 lb. Many factors may have influenced the yield value of the recycled joint. Previous tests being the primary factor in the properties of that joint.

5.5.2. Material Properties (Dowel Bearing Strength and MC)

Dowel bearing tests were performed on each eastern white pine mortise and tenon member. The mortise and tenon samples were prepared and loaded in a manner consistent with previous tests in this chapter. Summaries of the material properties are found in Table 5-4. The mean moisture content of the eastern white pine joints was 7.1% with a corresponding specific gravity of 0.349.

Table 5-4 Eastern White Pine Dowel Bearing Test Summary

Mortise Samples (56 specimens)		
Statistic	Yield Strength (lb/in^2)	Stiffness (lb/in^3)
Mean	1,890	16,700
St. Dev.	290	3,800
5% Exclusion	1,360	9,800
COV	0.156	0.229
K	1.801	1.801

Tenon Samples (54 specimens)		
Statistic	Yield Strength (lb/in^2)	Stiffness (lb/in^3)
Mean	6,820	162,300
St. Dev.	690	21,550
5% Exclusion	5,500	121,330
COV	0.101	0.133
K	1.901	1.901

5.6. Conclusions

The results did not indicate any trends that were followed consistently for each of the species of joints tested. The Douglas fir and southern yellow pine joints that were loaded had lower yield values than the unloaded joints. In direct contrast, the white oak and eastern white pine joints that were loaded had a higher mean yield value than the joints that were not loaded. None of the species showed a consistent significant difference between the loaded and unloaded joints.

A similar situation occurred with the drawbored joints. Drawboring did not effect the strength of the joint. There was not a consistent trend toward drawboring either increasing or decreasing joint strength.

A trend that is visible is that the loaded joints are stiffer up to the corresponding long-term load. This conclusion is difficult to quantify numerically; however it is visible in the load-deflection plots shown in the appendices. Long-term loading did not effect the ductility of the joints. The displacements at yield and at ultimate joint capacity were not affected by long-term load.

A second trend was found with the steel rods, the steel rods resulted in significantly higher mean yield values and a much stiffer joint. All of the joints with steel rods had brittle failures with no warning of impending failure. This brittle failure is typical of conventional wood connections with large diameter dowel fasteners.

6. Analysis, Summary and Conclusions

6.1. Correlation (MC-SG-Strength-Stiffness)

The possibility of a correlation between base material specific gravity and joint yield strength was discussed in Chapter 2. A plot was made using data from the short-term eastern white pine joint tests and tests reported by Schmidt and Daniels (1999). In an attempt to develop the correlation, the long-term test results were added to the plot, shown in Figure 6-1. The plot uses peg shear stress on four shear planes for a comparison of joint strength just as with the previous plot in Chapter 2.

A satisfactory correlation was not achieved. In particular, the long-term test points were scattered and without definite trends.

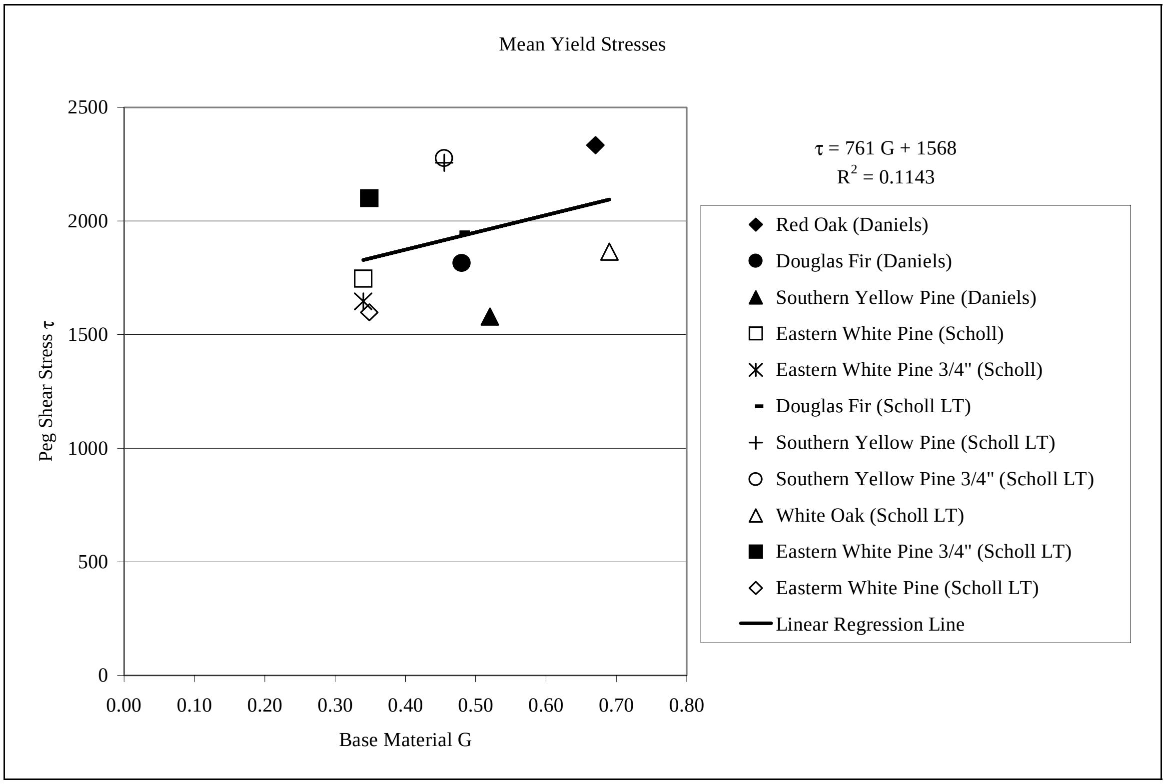


Figure 6-1 Base Material Specific Gravity-Joint Strength Correlation Plot

6.2. Modification to minimum end and edge distance, due to seasoning/creep/load duration

The minimum detailing distances were obtained primarily from test results from Schmidt and Daniels (1999); eastern white pine tests were conducted in this research to find the minimum allowable end and edge distances for that species. Table 6-1 shows the detailing and distances used for the long-term tests.

Table 6-1 Detailing Distances for Long-Term Test Joints

Species	End (D)	Edge (D)	Spacing (D)
Douglas Fir	2	2.5	2.5
Eastern White Pine	4	4	3*
Red/White Oak	2	2	2.5
Southern Yellow Pine	2**	2	3

*A constant value of 3" was used for testing
**3D with drawbore

The joints performed reasonably well. Tenon damage occurred in the Douglas fir joints with drawbore. The tenon ends split shortly after construction due to the stress from drawboring. An extra peg diameter was then added to the end distance of the southern yellow pine joints to reduce tenon splitting. The additional tenon length was successful in prevention of tenon splitting.

The white oak joints had tenon damage in nearly all of the loaded joints and over half of the joints with no long-term load. The mean moisture content of the white oak joints was 33% at the beginning of the test sequence. As the joints dried the tenons shrank radially and tangentially at a greater rate than the mortise members shrank longitudinally, resulting in tenon splitting. The stress imposed by the long-term load added to the stress from the differential shrinkage, resulting in more of the loaded joints having tenon damage. Therefore shrinkage was the primary cause of tenon damage; the long-term load on the joint was a secondary cause. However, to prevent tenon damage due to differential shrinkage a minimum end distance of 3.0D may be appropriate. This problem is not unexpected since it is well known that hardwoods generally shrink more than softwoods.

The eastern white pine joints performed well and without damage from the long term loading. Tenon damage was not present in the eastern white pine joints for two reasons. First, the initial moisture content was not as high as the other species tested. Secondly and most importantly the end distance of the joints was twice that of the white oak. The extra end distance gave the tenon added strength to overcome the effects of differential shrinkage.

The modified minimum detailing distances after changes due to long-term test findings are presented in Table 6-2 below. Note the white oak end distance is changed and all end distances are increased by 1D for drawbored joints.

Table 6-2 Modified Minimum Detailing Distances

Species	End (D)*	Edge (D)
Douglas Fir	2	2.5
Eastern White Pine	4	4
Red/White Oak	3	2
Southern Yellow Pine	2	2

*Add 1D with Drawbore

6.3. Load duration factor

Current design practice involves use of a load duration factor based upon the Madison Curve for connection design. However, mean yield values of joints that were loaded in long-term tests did not consistently have lower yield values than the joints that remained unloaded. Therefore, the Madison Curve does not appear to represent the behavior of mortise and tenon joints in tension.

The mean yield values of the joint species tested were in the realm of the mean yield values found in research by Schmidt and Daniels (1999). With this in mind, it can be concluded that design of mortise and tenon joints for long-term load is a serviceability concern rather than a strength issue.

Serviceability of joints is related to the joint behavior under typical loading (working level loads). Deflections of the joints with typical loading should be kept within reasonable limits established by the design engineer. These limits are imposed on design to assure a structure that will remain serviceable. Serviceability limits control non-strength related effects such as excessive gaps in joints, drywall cracking and floor vibration. In addition, large long-term deflections due to creep and shrinkage can result in load redistribution in indeterminate structures. The consequences of such behavior must be considered individually for each structure.

6.4. Design Values

The minimum detailing distances are given in Table 6-2. Drawboring and higher moisture content were causes of joint damage during the long-term testing. The joints that had tenon damage after long-term load application performed better than expected in the failure test. The joints with tenon damage had yield strengths and stiffnesses comparable to the joints that were undamaged. Confinement of the tenon by the mortise aided by not allowing the tenon split to open when load was applied.

In this research drawboring joints resulted in a high potential for tenon damage. It is therefore recommended to drawbore joints with smaller diameter pegs; ¾” white oak pegs performed well in drawbored joints. Drawboring of joints with 1” or larger diameter pegs should be done with caution and a realization of a high probability of tenon splitting. If drawboring is performed, the end distance should be increased by a minimum of one peg diameter.

6.5. Need for future work

Future research needs to be conducted involving drawboring. A substantial percentage (33%) of the joints that were drawbored without an increase in end distance experienced tenon splitting shortly after construction. Increasing end distance eliminated tenon splitting; additional research is needed in this area to validate this recommendation.

Further research is also needed before the total deflection of a joint under sustained load can accurately be predicted. The conclusion that load duration does not affect the yield strength of the jointis consistent with the results found by previous researchers (Wilkinson, 1988; Fridley & Rosowsky, 1998; Rosowsky & Reinhold, 1999). Further study should also involve the effect of moisture content on tenon splitting. The difference in shrinkage rates will be present in all joints; an allowable maximum moisture content should be found.

7. References

AFPA, (1997). "National Design Specification for Wood Construction," American Forest & Paper Association (AFPA), Washington, DC.

ASTM, (1999). *1999 Annual Book of ASTM Standards, 04.10 Wood,* Philadelphia, PA.

Borchers, M. (1999), "Alternate Mode III Failure of Bolted Connections", Diploma Thesis, Institut fuer Baukonstruktion und Holzbau, Technische Universitaet Braunschweig, Germany.

Breyer, D.E., Fridley, K. J., and Cobeen, K. E. (1999). "Design of Wood Structures" Fourth Edition, McGraw Hill Inc. New York

Fridley, K. J. and Rosowsky, D. V. (1998) "Time-Dependent Behavior of Nailed Connections in Direct Shear," *Proceedings, World Conference on Timber . Engineering,* Vol. 1, pp. 369-374. Montreux, Switzerland, August 17-20

Reid, E.H. (1997). "Behavior of Wood Pegs in Traditional Timber Frame Connections" M.S. Thesis, Michigan Technological University.

Rosowsky, D. V. and Reinhold, T. A. (1999) "Rate-of-Load and Duration-of-Load Effects for Wood Fasteners," *Journal of Structural Engineering,* 125(7):719-724.

Sandberg, L. B., Bulleit, W. M., and Reid, E. H. (2000) "Strength and Stiffness of Oak Pegs in Traditional Timber-Frame Joints, *Journal of Structural Engineering,* 126(6):717

Schmidt, R.J. and Daniels, C.E. (1999). "Design Considerations for Mortise and Tenon Connections" Research Report, Department of Civil and Architectural Engineering, University of Wyoming, Laramie, WY 82070, April 1999.

Schmidt, R.J. and MacKay, R.B. (1997). "Timber Frame Tension Joinery" Research Report, Department of Civil and Architectural Engineering, University of Wyoming, Laramie, WY 82070, October 1997.

Wilkinson, T. L. (1988) "Duration of Load on Bolted Joints: A Pilot Study." No. FPL-RP-488, Forest Products Laboratory, U.S. Dept of Agriculture, Washington, DC.

Appendices

Appendix A (Douglas fir)

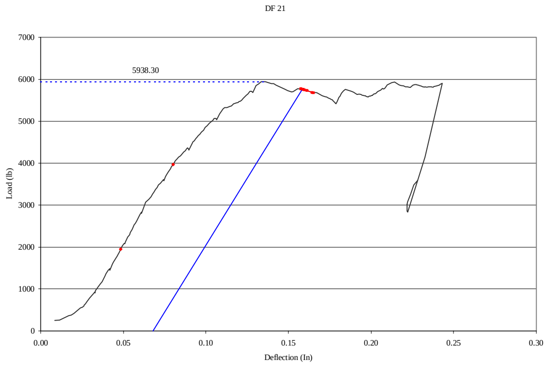
Joint Test Results

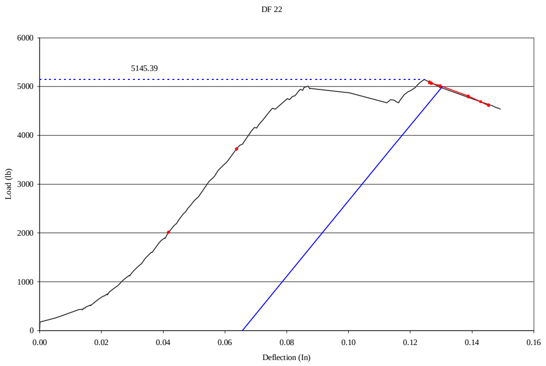
Joint Number	Long Term Load (lb)	Drawbore	Peg Dia. (In)	Yield Disp. (In)	Yield Load (lb)	Stiffness (lb/in)	Ult. Disp (in)	Ult. Load (lb)
DF21	2000	No	1	0.136	5,940	63,800	0.136	5940
DF22	2000	No	1	0.124	5,150	77,200	0.124	5150
DF23	2000	No	1	0.135	4,920	43,700	0.208	4980
DF24	0	No	1	0.180	5,880	47,400	0.180	5880
DF25	0	No	1	0.204	7,510	45,500	0.204	7510
DF26	0	No	1	0.200	6,500	32,200	0.200	6500
DF27	2000	Yes	1	0.156	6,710	77,800	0.230	7350
DF28	2000	Yes	1	0.113	7,110	100,300	0.307	7130
DF29	2000	Yes	1	0.147	4,980	60,100	0.369	5720
DF30	0	Yes	1	0.137	6,040	64,900	0.137	6040
DF31	0	Yes	1	0.131	5,900	70,200	0.131	5900
DF32	0	Yes	1	0.171	6,850	64,400	0.171	6850

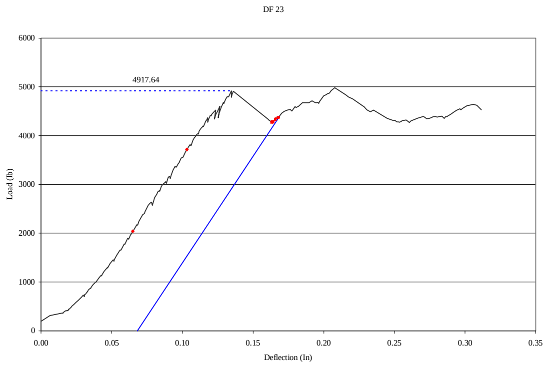
Mean	0.153	6,120	62,300	0.200	6200
Loaded	0.135	5,800	70,500	0.229	6000
Unloaded	0.168	6,450	54,100	0.170	6400
Drawbore	0.143	6,260	73,000	0.224	6500
No Drawbore	0.163	5,980	51,600	0.175	6000
No Drawbore; No Load	0.194	6,630	41,700	0.194	6600
No Drawbore; 2000 lb Load	0.132	5,330	61,600	0.156	5400
Drawbore; No Load	0.146	6,260	66,500	0.146	6300
Drawbore Load	0.139	6,260	79,400	0.302	6700

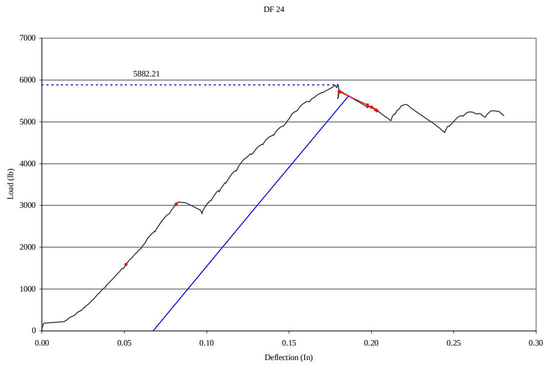
Load-Deflection Plots

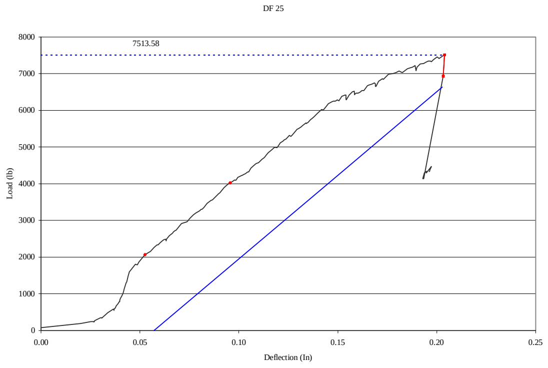
Douglas Fir

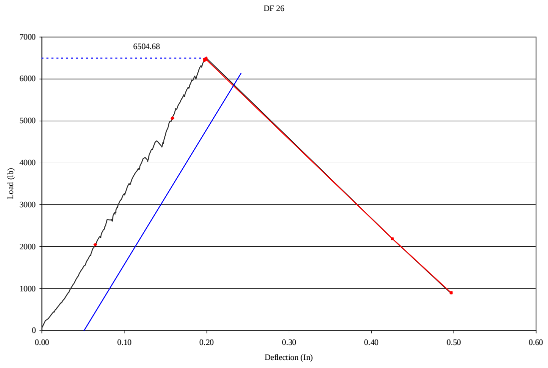


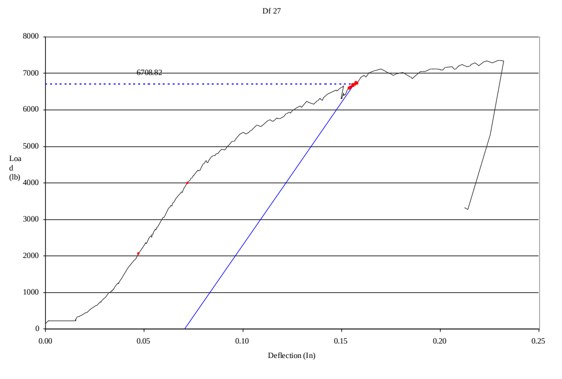











Df 27
6708.82
Load (lb)
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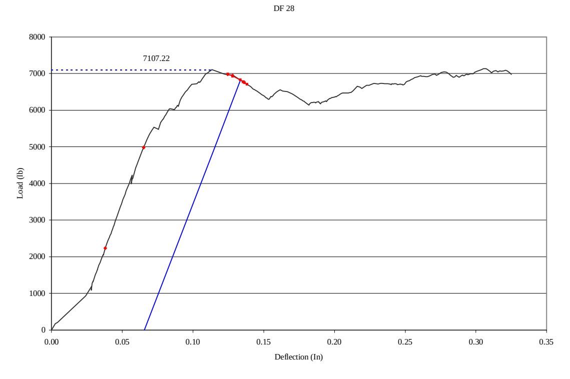
7000
6000
5000
4000
3000
2000
1000
0
0.00
0.05
0.10
0.15
0.20
0.25
Deflection (In)


DF 28
7107.22
Load (lb)
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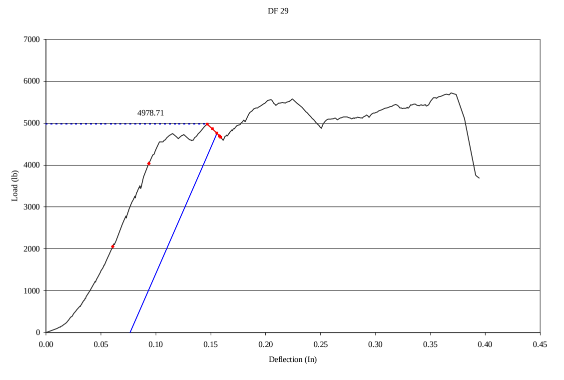
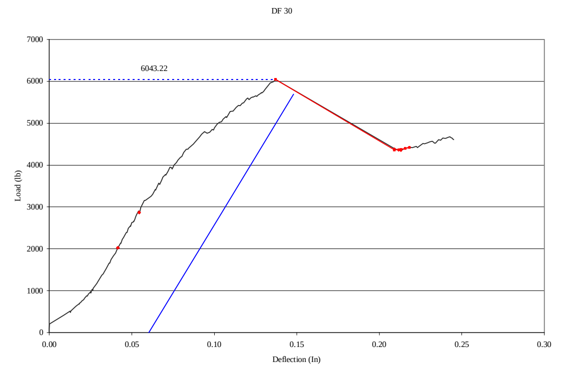
7000
6000
5000
4000
3000
2000
1000
0
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
Deflection (In)


DF 29
4978.71
Load (lb)
7000
6000
5000
4000
3000
2000
1000
0
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
Deflection (In)


DF 30
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Load (lb)
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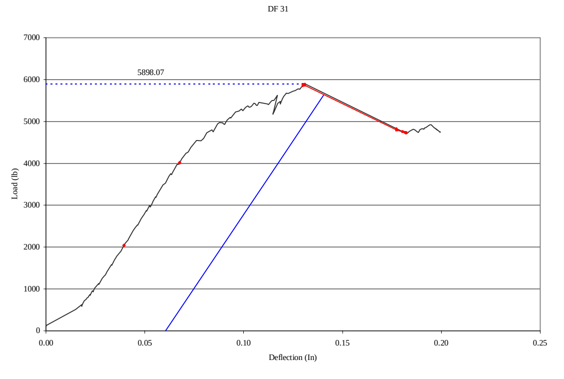
Deflection (In)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Load (lb)
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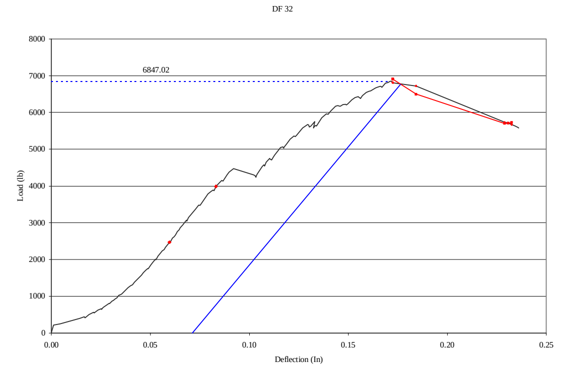
0.25
Deflection (In)


DF 32
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Load (lb)
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6000
5000
4000
3000
2000
1000
0
0.00
0.05
0.10
0.15
0.20
0.25
Deflection (In)

Dowel Bearing Test Results

Test Number	Yield Value (lb/in2)	Stiffness (lb/in3)
DF21M1	3,800	57,500
DF21M2	3,630	57,100
DF22M1	4,280	59,800
DF22M2	4,110	60,300
DF23M1	2,660	36,200
DF23M2	2,620	26,100
DF24M1	2,680	23,600
DF24M2	2,850	28,100
DF25M1	2,190	27,100
DF25M2	2,760	33,600
DF26M1	1,820	20,300
DF26M2	1,930	21,300
DF27M1	2,230	20,400
DF27M2	2,560	21,300
DF28M1	2,580	31,400
DF28M2	2,370	34,000
DF29M1	2,570	27,800
DF29M2	2,570	27,500
DF30M1	2,550	31,000
DF30M2	3,030	31,500
DF31M1	2,640	27,900
DF31M2	2,720	30,000
DF32M1	2,330	24,300
DF32M2	2,110	22,000
Mean	2,730	32,500
St. Dev.	630	12,750
5% Exclusion	1,530	8,260
COV	0.231	0.392
K	1.901	1.901

Test Number	Yield Value (lb/in2)	Stiffness (lb/in3)
DF21T1	7,210	191,500
DF21T2	7,770	149,200
DF22T1	6,430	133,600
DF22T2	6,600	147,500
DF23T1	5,780	160,300
DF23T2	6,670	179,100
DF24T1	6,730	146,900
DF24T2	5,080	148,700
DF25T1	7,370	162,600
DF25T2	7,040	171,600
DF26T1	7,100	193,100
DF26T2	7,000	184,200
DF27T1	7,610	140,700
DF27T2	8,510	179,900
DF28T1	6,400	153,700
DF28T2	6,190	169,400
DF29T1	7,220	145,400
DF29T2	7,430	179,100
DF30T1	6,590	175,800
DF30T2	6,970	180,900
DF31T1	6,750	189,200
DF31T2	6,450	172,400
DF32T1	6,540	124,400
DF32T2	6,240	116,100
Mean	6,820	162,300
St. Dev.	690	21,550
5% Exclusion	5,500	121,330
COV	0.101	0.133
K	1.901	1.901

Specific Gravity and Moisture Contents at the Conclusion of Testing

Member	Moisture Content	S.G.	Member	Moisture Content	S.G.
DF 21 M	8.6%	0.456	DF 21 T	9.8%	0.494
DF 22 M	9.7%	0.503	DF 22 T	8.6%	0.469
DF 23 M	8.3%	0.491	DF 23 T	9.4%	0.568
DF 24 M	9.4%	0.521	DF 24 T	8.4%	0.473
DF 25 M	8.3%	0.448	DF 25 T	8.9%	0.481
DF 26 M	9.6%	0.492	DF 26 T	8.3%	0.474
DF 27 M	9.6%	0.473	DF 27 T	9.3%	0.493
DF 28 M	10.3%	0.499	DF 28 T	9.6%	0.445
DF 29 M	8.6%	0.482	DF 29 T	6.7%	0.522
DF 30 M	9.4%	0.415	DF 30 T	8.5%	0.478
DF 31 M	8.2%	0.487	DF 31 T	7.5%	0.479
DF 32 M	9.3%	0.416	DF 32 T	10.2%	0.419
Mean	9.1%	0.474	Mean	8.8%	0.483
St. Dev.	0.67%	0.033	St. Dev.	0.99%	0.037
COV	0.074	0.071	COV	0.113	0.077

Peg Specific Gravity and Moisture Contents at the Conclusion of Testing

	Peg 1		Peg 2		Average	
Joint	Moisture Content	Specific Gravity	Moisture Content	Specific Gravity	Moisture Content	Specific Gravity
DF 21	12.7%	0.57	9.8%	0.58	11.3%	0.58
DF 22	10.6%	0.60	12.2%	0.60	11.4%	0.60
DF 23	10.0%	0.57	12.4%	0.62	11.2%	0.59
DF 24	9.9%	0.59	10.3%	0.59	10.1%	0.59
DF 25	10.8%	0.74	11.4%	0.71	11.1%	0.72
DF 26	9.4%	0.73	9.8%	0.73	9.6%	0.73
DF 27	8.0%	0.69	9.7%	0.73	8.8%	0.71
DF 28	10.3%	0.71	10.6%	0.77	10.4%	0.74
DF 29	9.4%	0.65	10.4%	0.68	9.9%	0.66
DF 30	11.6%	0.67	9.8%	0.65	10.7%	0.66
DF 31	12.1%	0.64	10.9%	0.65	11.5%	0.65
DF 32	8.7%	0.68	9.6%	0.69	9.1%	0.68
				Mean	10.4%	0.66

Appendix B (southern yellow pine)

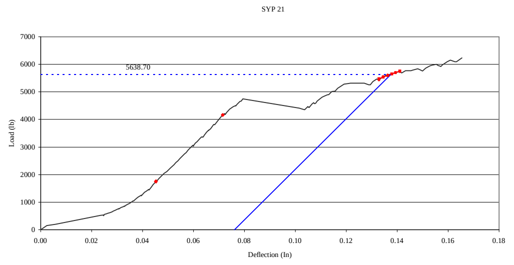
Joint Test Results

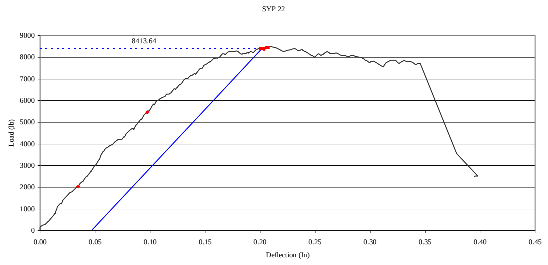
Joint Number	Long Term Load (lb)	Drawbore	Peg Dia. (In)	Yield Disp. (In)	Yield Load (lb)	Stiffness (lb/in)	Ult. Disp (in)	Ult. Load (lb)
SYP 21	1000	No	1	0.136	5,640	91,800	0.165	6,230
SYP 22	1000	No	1	0.200	8,410	54,300	0.207	8,480
SYP 23	1000	No	1	0.117	7,050	91,500	0.117	7,050
SYP 24	2000	No	1	0.112	7,490	105,800	0.112	7,490
SYP 25	2000	No	1	0.151	6,660	76,400	0.151	6,660
SYP 26	2000	No	1	0.135	6,670	70,100	0.135	6,670
SYP 27	0	No	1	0.133	6,960	81,600	0.133	6,960
SYP 28	0	No	1	0.172	8,060	64,800	0.172	8,060
SYP 29	0	No	1	0.134	7,750	75,100	0.134	7,750
SYP 30	2000	Yes	1	0.096	6,510	107,800	0.096	6,510
SYP 31	2000	Yes	1	0.183	5,000	49,800	0.197	5,150
SYP 32	2000	Yes	1	0.104	7,520	144,400	0.114	7,630
SYP 33	0	Yes	1	0.102	7,420	104,100	0.102	7,420
SYP 34	0	Yes	1	0.144	7,540	59,500	0.144	7,540
SYP 35	0	Yes	1	0.134	7,740	65,400	0.134	7,740
SYP 36	0	Yes	0.75	0.101	3,760	57,900	0.101	3,760
SYP 37	0	Yes	0.75	0.100	3,480	49,600	0.218	3,780
SYP 38	0	Yes	0.75	0.106	4,560	56,100	0.157	4,570
SYP 39	1000	Yes	0.75	0.088	4,490	80,600	0.117	4,540
SYP 40	1000	Yes	0.75	0.117	3,560	39,900	0.251	3,880
SYP 41	1000	Yes	0.75	0.092	4,280	65,400	0.092	4,280

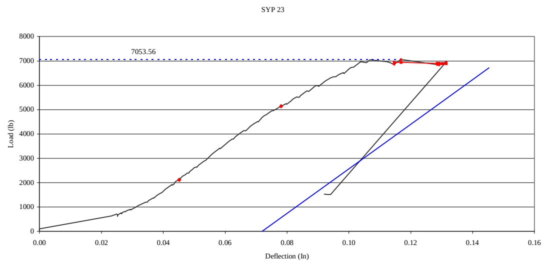
1"					
Mean	0.137	7,090	82,800	0.141	7,160
	0.000				
Loaded 1000 lb	0.151	7,040	79,200	0.163	7,250
Loaded 2000 lb	0.130	6,640	92,400	0.134	6,680
Unloaded	0.137	7,580	75,100	0.137	7,580
Drawbore	0.127	6,950	88,500	0.131	7,000
No Drawbore	0.143	7,190	79,000	0.147	7,260
No Drawbore No Load	0.146	7,590	73,800	0.146	7,590
No Drawbore 1000 lb Load	0.151	7,040	79,200	0.163	7,250
No Drawbore 2000 lb Load	0.133	6,940	84,100	0.133	6,940
Drawbore No Load	0.127	7,560	76,300	0.127	7,560
Drawbore 2000 lb Load	0.128	6,340	100,700	0.135	6,430
3/4"					
Mean	0.101	4,020	58,300	0.156	4,100
Loaded 1000	0.099	4,110	62,000	0.153	4,200
Unloaded	0.102	3,930	54,600	0.159	4,000

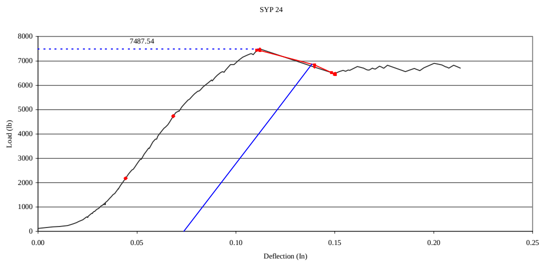
Load-Deflection Plots

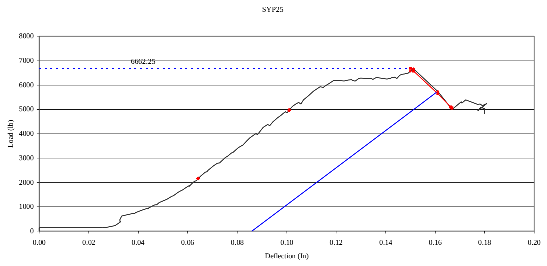
Southern Yellow Pine

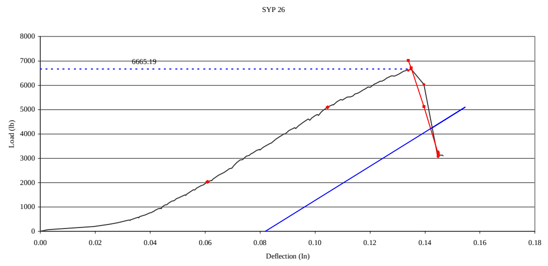


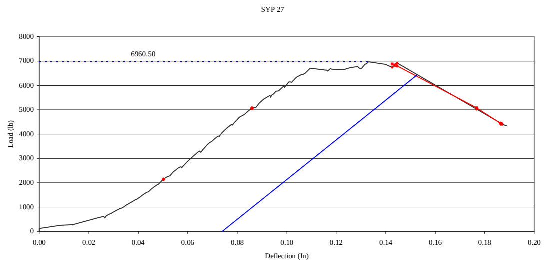








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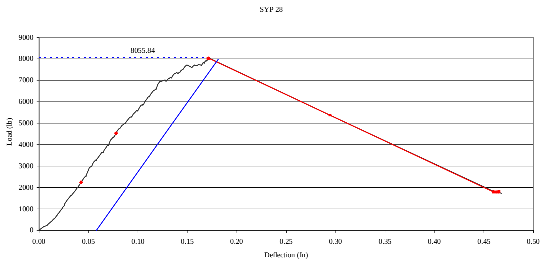
8000
7000
6000
5000
4000
3000
2000
1000
0
Load (lb)
0.00
0.02
0.04
0.06
0.08
0.10
0.12
0.14
0.16
0.18
0.20
Deflection (In)


SYP 28
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9000
8000
7000
6000
5000
4000
3000
2000
1000
0
Load (lb)
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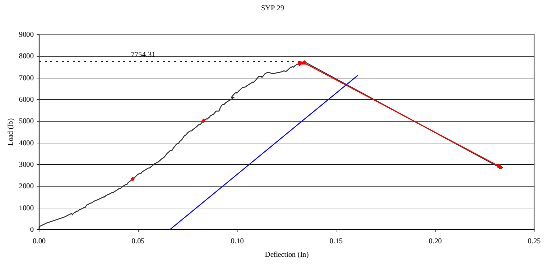
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
Deflection (In)


SYP 29
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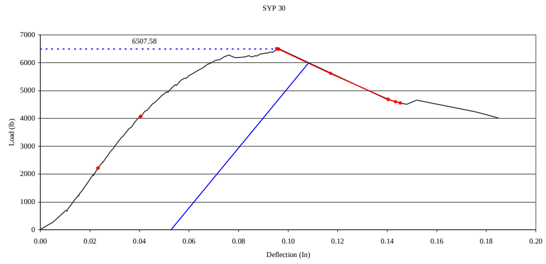
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
Load (lb)
0.00
0.05
0.10
0.15
0.20
0.25
Deflection (In)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7000
6000
5000
4000
3000
2000
1000
0
Load (lb)
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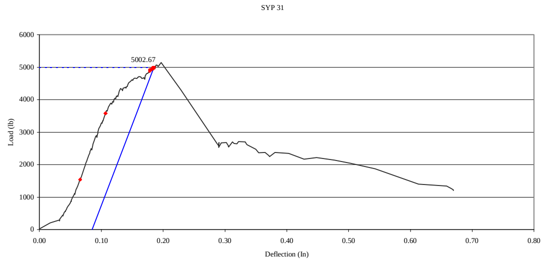
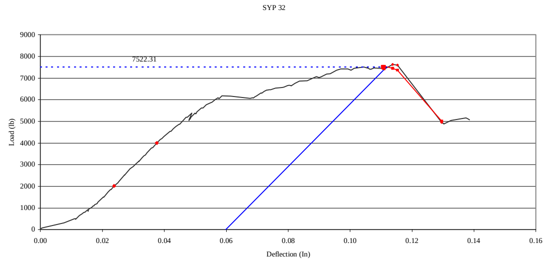
0.02
0.04
0.06
0.08
0.10
0.12
0.14
0.16
0.18
0.20
Deflection (In)


SYP 31
5002.67
6000
5000
4000
3000
2000
1000
0
Load (lb)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
Deflection (In)


SYP 32
7522.31
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
Load (lb)
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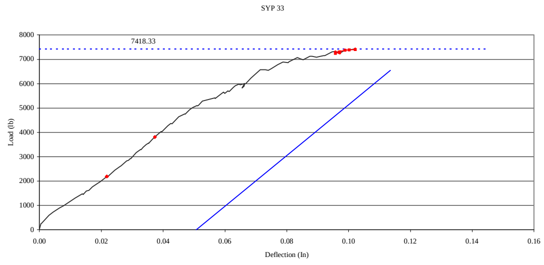
0.02
0.04
0.06
0.08
0.10
0.12
0.14
0.16
Deflection (In)


SYP 33
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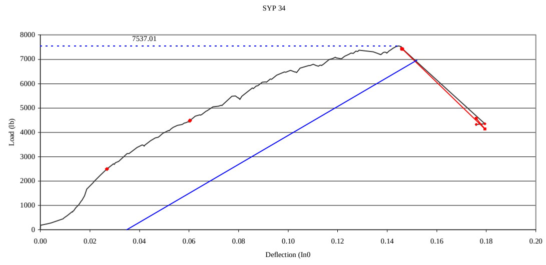
8000
7000
6000
5000
4000
3000
2000
1000
0
Load (lb)
0.00
0.02
0.04
0.06
0.08
0.10
0.12
0.14
0.16
Deflection (In)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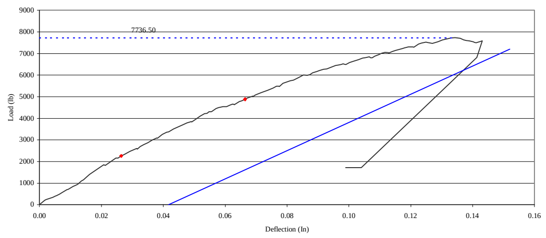
8000
7000
6000
5000
4000
3000
2000
1000
0
Load (lb)
0.00
0.02
0.04
0.06
0.08
0.10
0.12
0.14
0.16
0.18
0.20
Deflection (In0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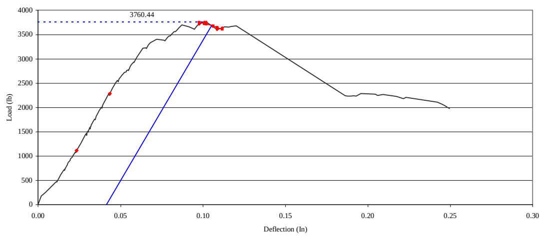
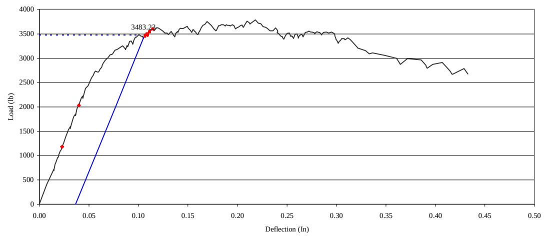

SYP 36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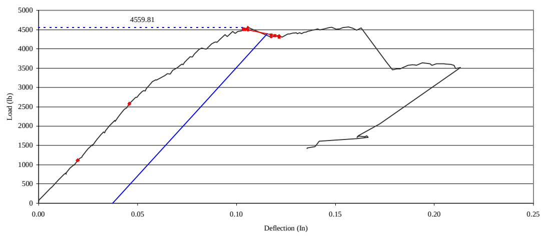
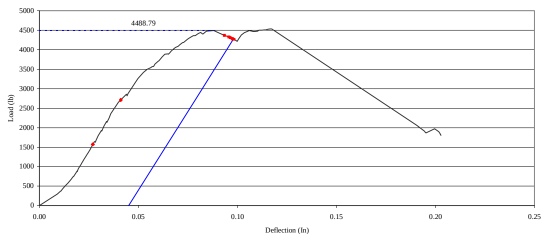
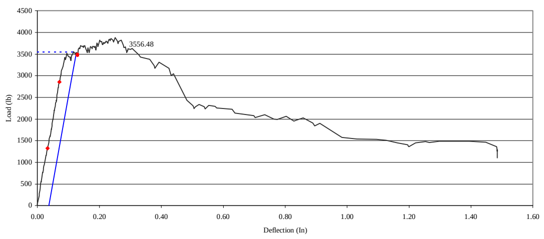
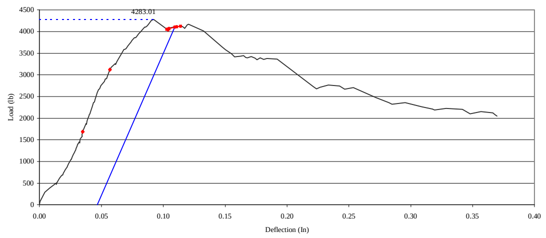

SYP 38


SYP 39


SYP 40


SYP 41

Dowel Bearing Test Results

Test Number	Yield Value (lb/in2)	Stiffness (lb/in3)
SYP21M1	2,570	31,400
SYP21M2	2,290	32,600
SYP22M1	2,310	19,300
SYP22M2	2,220	24,300
SYP23M1	2,030	24,500
SYP23M2	2,150	29,000
SYP24M1	2,270	24,700
STP24M2	2,250	29,800
SYP25M1	2,550	35,000
SYP25M2	2,510	30,200
SYP26M1	2,630	33,000
SYP26M2	2,470	26,600
SYP27M1	2,950	31,300
SYP27M2	2,940	35,900
SYP28M1	2,860	45,300
SYP28M2	2,310	26,700
SYP29M1	2,690	31,800
SYP29M2	2,880	35,300
SYP30M1	2,300	28,300
SYP30M2	2,000	19,200
SYP31M1	2,290	30,600
SYP31M2	2,120	26,400
SYP32M1	2,300	30,600
SYP32M2	2,330	31,000
SYP33M1	2,130	24,100
SYP33M2	2,190	26,100
SYP34M1	2,610	30,700
SYP34M2	2,830	33,200
SYP35M1	1,790	21,300
SYP35M2	1,730	19,400
SYP36M1	2,300	27,300
SYP36M2	2,150	27,500
SYP37M1	2,160	27,400
SYP37M2	2,060	25,300
SYP38M1	2,720	34,000
SYP38M2	2,550	31,000
SYP39M1	2,610	36,700
SYP39M2	2,500	30,900
SYP40M1	2,050	24,200
SYP40M2	2,060	24,700
SYP41M1	2,060	24,800
SYP41M2	1,960	25,200
Mean	2,350	28,730
St. Dev.	310	5,190
5% Exclusion	1,790	19,230
COV	0.130	0.181
K	1.829	1.829

Test Number	Yield Value (lb/in2)	Stiffness (lb/in3)
SYP21T1	5,210	78,600
SYP21T2	5,350	94,800
SYP22T1	4,820	142,100
SYP22T2	5,040	143,200
SYP23T1	5,780	116,600
SYP23T2	5,370	144,200
SYP24T1	5,620	135,900
STP24T2	5,940	133,400
SYP25T1	5,940	139,200
SYP25T2	5,470	165,800
SYP26T1	4,820	122,200
SYP26T2	5,350	134,400
SYP27T1	5,490	128,500
SYP27T2	5,330	147,300
SYP28T1	5,350	94,200
SYP28T2	5,660	102,600
SYP29T1	5,100	118,100
SYP29T2	5,270	125,600
SYP30T1	5,430	134,100
SYP30T2	5,640	141,100
SYP31T1	4,730	96,900
SYP31T2	5,720	83,600
SYP32T1	4,720	119,600
SYP32T2	5,140	126,200
SYP33T1	6,260	162,500
SYP33T2	6,820	135,600
SYP34T1	4,910	107,600
SYP34T2	4,710	119,300
SYP35T1	4,240	82,700
SYP35T2	4,690	88,300
SYP36T1	4,420	108,600
SYP36T2	4,260	92,000
SYP37T1	4,040	133,500
SYP37T2	4,570	148,300
SYP38T1	4,650	100,800
SYP38T2	4,810	132,300
SYP39T1	5,330	127,700
SYP39T2	5,040	152,500
SYP40T1	4,490	100,900
SYP40T2	4,550	100,200
SYP41T1	4,340	97,000
SYP41T2	4,560	110,400
Mean	5,120	120,700
St. Dev.	590	22,580
5% Exclusion	4,040	79,360
COV	0.115	0.187
K	1.829	1.829

Specific Gravity and Moisture Contents at the Conclusion of Testing

Member	Moisture Content	S.G.
SYP 21 M	8.6%	0.486
SYP 22 M	9.6%	0.454
SYP 23 M	9.1%	0.458
SYP 24 M	8.5%	0.458
SYP 25 M	11.3%	0.480
SYP 26 M	8.6%	0.390
SYP 27 M	9.4%	0.448
SYP 28 M	10.4%	0.458
SYP 29 M	9.3%	0.364
SYP 30 M	8.2%	0.488
SYP 31 M	7.7%	0.398
SYP 32 M	11.1%	0.436
SYP 33 M	8.2%	0.472
SYP 34 M	9.0%	0.421
SYP 35 M	8.0%	0.484
SYP 36 M	6.5%	0.400
SYP 37 M	6.3%	0.515
SYP 38 M	6.2%	0.414
SYP 39 M	10.9%	0.457
SYP 40 M	7.4%	0.447
SYP 41 M	10.5%	0.479
Mean	8.8%	0.45
St. Dev.	1.5%	0.04
COV	0.17	0.09

Member	Moisture Content	S.G.
SYP 21 T	9.9%	0.492
SYP 22 T	9.5%	0.422
SYP 23 T	8.0%	0.411
SYP 24 T	8.8%	0.441
SYP 25 T	8.8%	0.457
SYP 26 T	10.1%	0.532
SYP 27 T	9.4%	0.457
SYP 28 T	8.7%	0.467
SYP 29 T	7.8%	0.464
SYP 30 T	10.0%	0.420
SYP 31 T	9.3%	0.397
SYP 32 T	10.1%	0.421
SYP 33 T	8.3%	0.441
SYP 34 T	11.7%	0.470
SYP 35 T	7.7%	0.390
SYP 36 T	11.4%	0.485
SYP 37 T	8.7%	0.463
SYP 38 T	11.3%	0.495
SYP 39 T	8.6%	0.593
SYP 40 T	10.4%	0.482
SYP 41 T	7.6%	0.457
Mean	9.3%	0.46
St.Dev.	1.2%	0.05
COV	0.13	0.10

Peg Specific Gravity and Moisture Contents at the Conclusion of Testing

	Peg 1		Peg 2		Average	
Joint	Moisture Content	Specific Gravity	Moisture Content	Specific Gravity	Moisture Content	Specific Gravity
SYP 21	9.4%	0.74	11.6%	0.74	10.5%	0.74
SYP 22	10.5%	0.77	9.3%	0.80	9.9%	0.78
SYP 23	11.4%	0.73	9.9%	0.80	10.6%	0.76
SYP 24	9.9%	0.72	11.0%	0.72	10.5%	0.72
SYP 25	8.7%	0.70	8.7%	0.74	8.7%	0.72
SYP 26	9.6%	0.78	11.5%	0.76	10.5%	0.77
SYP 27	10.4%	0.79	8.0%	0.75	9.2%	0.77
SYP 28	11.2%	0.74	9.8%	0.75	10.5%	0.74
SYP 29	9.6%	0.77	9.0%	0.79	9.3%	0.78
SYP 30	12.5%	0.66	9.7%	0.66	11.1%	0.66
SYP 31	9.2%	0.64	11.8%	0.67	10.5%	0.66
SYP 32	9.2%	0.82	15.6%	0.77	12.4%	0.79
SYP 33	9.2%	0.80	13.9%	0.77	11.6%	0.79
SYP 34	8.4%	0.80	13.2%	0.80	10.8%	0.80
SYP 35	12.5%	0.74	10.5%	0.79	11.5%	0.76
SYP 36	13.0%	0.62	12.1%	0.64	12.6%	0.63
SYP 37	7.2%	0.62	9.4%	0.63	8.3%	0.62
SYP 38	11.9%	0.74	12.3%	0.73	12.1%	0.74
SYP 39	19.7%	0.66	12.5%	0.69	16.1%	0.67
SYP 40	9.1%	0.58	19.7%	0.58	14.4%	0.58
SYP 41	11.9%	0.61	10.3%	0.65	11.1%	0.63
				Mean	11.1%	0.72

Appendix C (white oak)

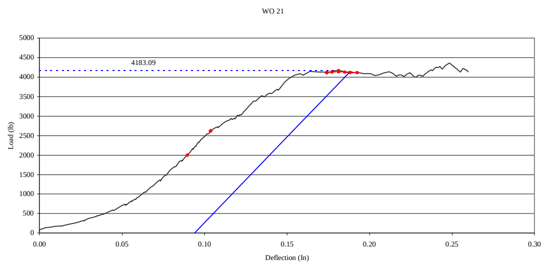
7.1.1. Joint Test Results

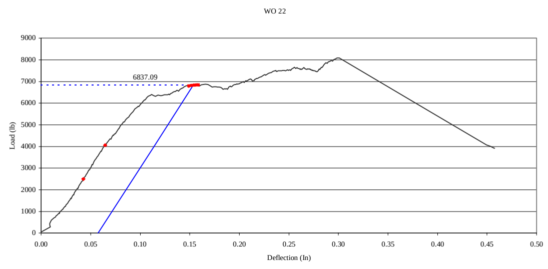
Joint Number	Long Term Load (lb)	Drawbore	Peg Dia. (In)	Yield Disp. (In)	Yield Load (lb)	Stiffness (lb/in)	Ult. Disp (in)	Ult. Load (lb)
WO21	2000	No	1	0.181	4,180	43,900	0.248	4,360
WO22	2000	No	1	0.153	6,840	71,100	0.301	8,090
WO23	2000	No	1	0.108	8,810	129,500	0.108	8,810
WO24	1000	No	1	0.116	5,160	77,300	0.249	6,220
WO25	1000	No	1	0.128	5,010	66,900	0.432	7,260
WO26	1000	No	1	0.095	3,470	106,000	0.284	4,830
WO27	2000	No	1	0.087	6,470	81,100	0.087	6,470
WO28	2000	No	1	0.139	6,260	74,300	0.376	7,950
WO29	2000	No	1	0.178	5,890	62,800	0.464	7,750
WO30	1000	No	1	0.112	5,890	86,600	0.118	6,070
WO31	1000	No	1	0.118	4,720	44,200	0.118	4,720
WO32	1000	No	1	0.105	6,380	64,300	0.105	6,380
WO33	0	No	1	0.118	6,840	80,300	0.182	7,540
WO34	0	No	1	0.136	5,900	61,700	0.501	6,470
WO35	0	No	1	0.150	5,800	52,600	0.309	6,830
WO36	0	No	1	0.112	6,580	87,900	0.187	6,920
WO37	0	No	1	0.137	6,020	71,700	0.270	6,630
WO38	0	No	1	0.106	6,140	89,600	0.218	6,730
WO39	0	No	1	0.143	6,390	66,200	0.362	7,210
WO40	0	No	1	0.057	3,830	80,700	0.257	4,120
WO41	0	No	1	0.117	6,310	95,900	0.190	7,070
WO42	0	No	1	0.083	6,477	52,165	0.154	6,536
WO43	2000	No	1 (Steel)	0.027	8,280	262,800	0.027	8,280
WO44	2000	No	1 (Steel)	0.052	14,410	333,900	0.052	14,410

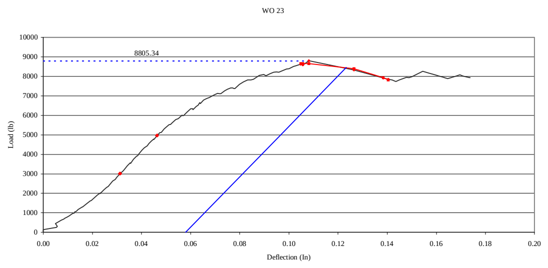
Mean	0.114	6,310	116,000	0.233	6,990
Loaded 1000 lb White Oak Pegs	0.112	5,110	74,200	0.218	5,920
Loaded 2000 lb White Oak Pegs	0.141	6,410	77,100	0.264	7,240
Unloaded White Oak Pegs	0.115	5,980	128,000	0.263	6,610
Loaded 2000 lb Steel Pegs	0.039	11,340	298,300	0.039	11,340

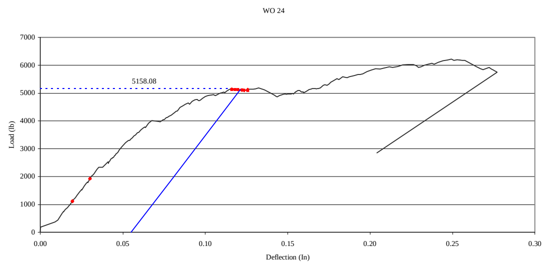
Load-Deflection Plots

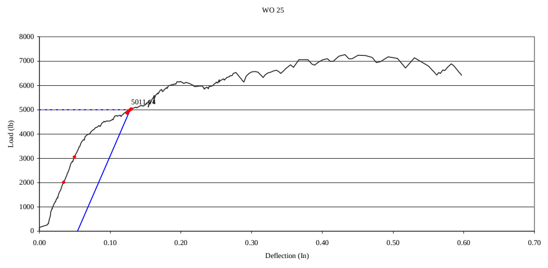
White Oak

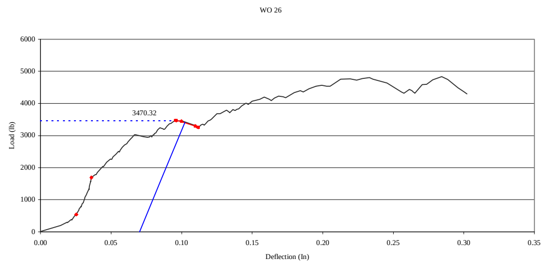


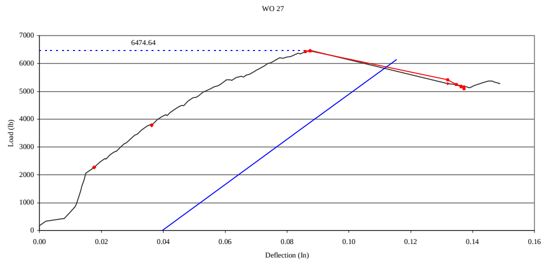








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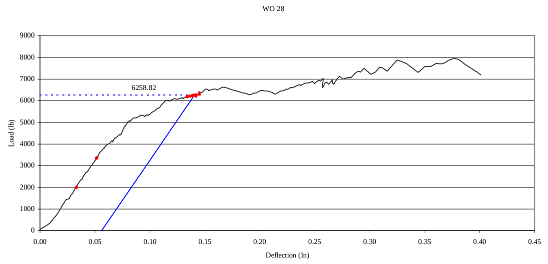
6474.64
Load (lb)
0
1000
2000
3000
4000
5000
6000
7000
0.00
0.02
0.04
0.06
0.08
0.10
0.12
0.14
0.16
Deflection (In)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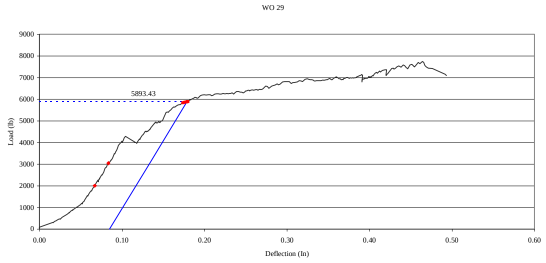
6258.82
Load (lb)
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
Deflection (In)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5893.43
Load (lb)
0
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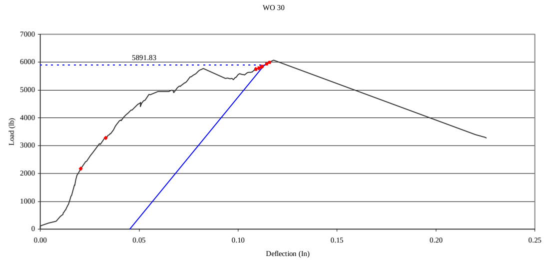
2000
3000
4000
5000
6000
7000
8000
9000
0.00
0.10
0.20
0.30
0.40
0.50
0.60
Deflection (In)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5891.83
Load (lb)
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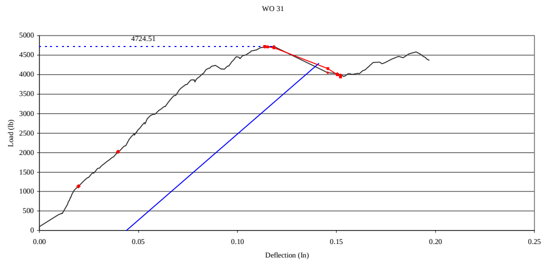
2000
3000
4000
5000
6000
7000
0.00
0.05
0.10
0.15
0.20
0.25
Deflection (In)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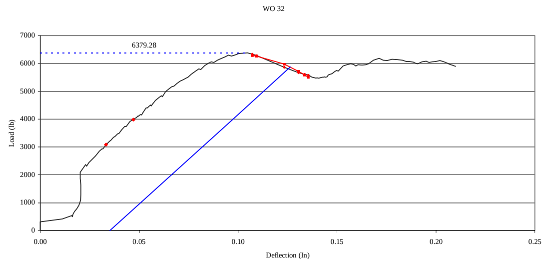
Load (lb)
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
0.00
0.05
0.10
0.15
0.20
0.25
Deflection (In)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6379.28
Load (lb)
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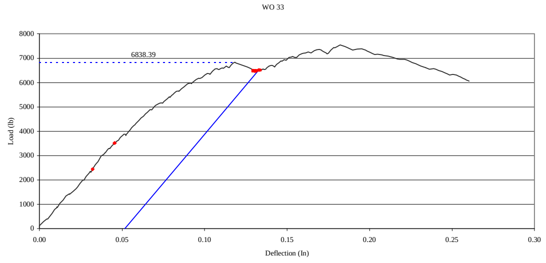
2000
3000
4000
5000
6000
7000
0.00
0.05
0.10
0.15
0.20
0.25
Deflection (In)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6838.39
Load (lb)
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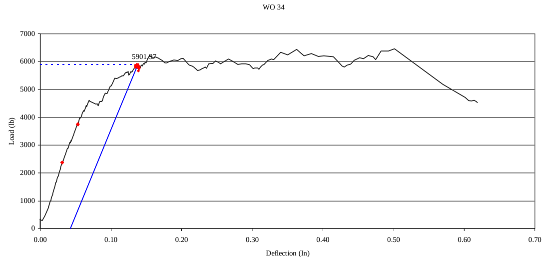
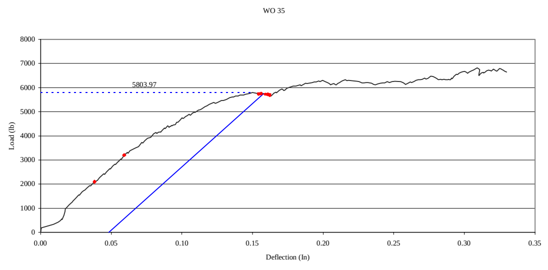
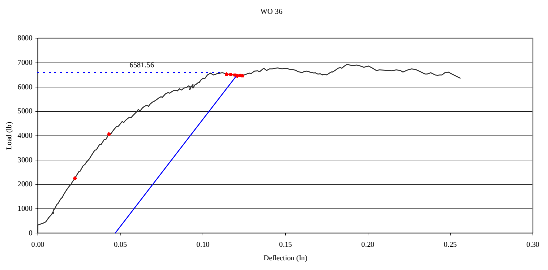
1000
2000
3000
4000
5000
6000
7000
8000
0.00
0.05
0.10
0.15
0.20
0.25
0.30
Deflection (In)


WO 34
5901.57
Load (lb)
0
1000
2000
3000
4000
5000
6000
7000
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
Deflection (In)


WO 35
5803.97
Load (lb)
0
1000
2000
3000
4000
5000
6000
7000
8000
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
Deflection (In)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6581.56
Load (lb)
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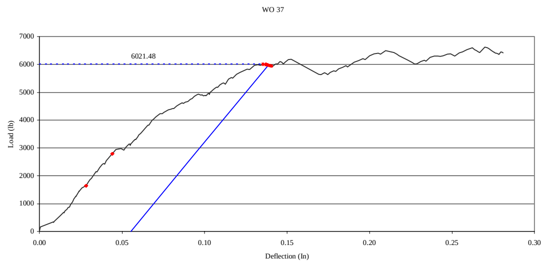
2000
3000
4000
5000
6000
7000
8000
0.00
0.05
0.10
0.15
0.20
0.25
0.30
Deflection (In)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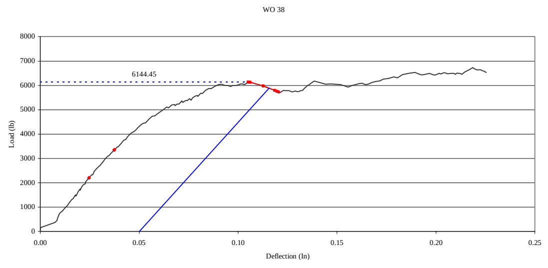
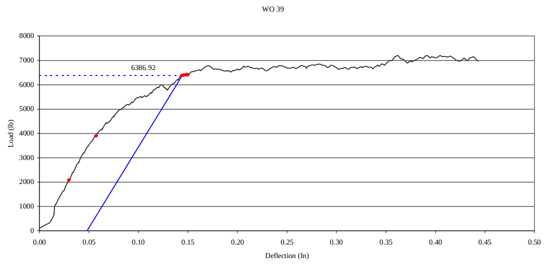
6021.48
Load (lb)
0
1000
2000
3000
4000
5000
6000
7000
0.00
0.05
0.10
0.15
0.20
0.25
0.30
Deflection (In)


WO 38
6144.45
Load (lb)
0
1000
2000
3000
4000
5000
6000
7000
8000
0.00
0.05
0.10
0.15
0.20
0.25
Deflection (In)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6386.92
Load (lb)
0
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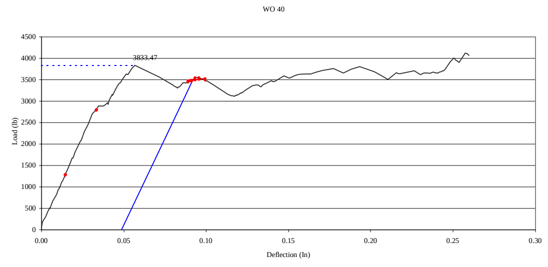
2000
3000
4000
5000
6000
7000
8000
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
Deflection (In)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3833.47
Load (lb)
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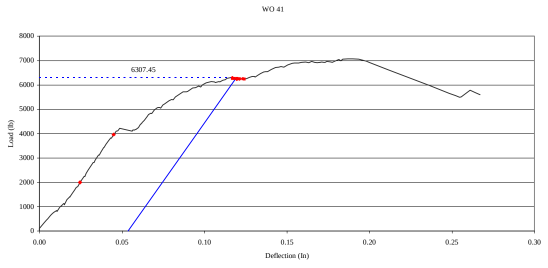
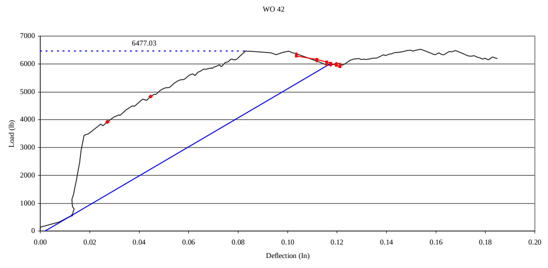
1000
1500
2000
2500
3000
3500
4000
4500
0.00
0.05
0.10
0.15
0.20
0.25
0.30
Deflection (In)


WO 41
6307.45
Load (lb)
0
1000
2000
3000
4000
5000
6000
7000
8000
0.00
0.05
0.10
0.15
0.20
0.25
0.30
Deflection (In)


WO 42
6477.03
Load (lb)
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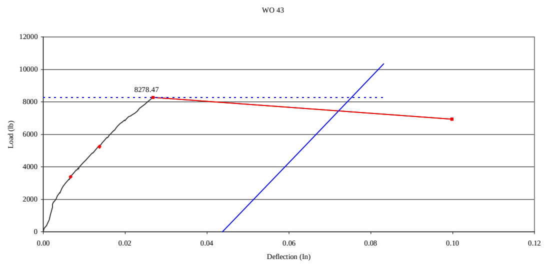
2000
3000
4000
5000
6000
7000
0.00
0.02
0.04
0.06
0.08
0.10
0.12
0.14
0.16
0.18
0.20
Deflection (In)


WO 43
12000
10000
8000
6000
4000
2000
0
Load (lb)
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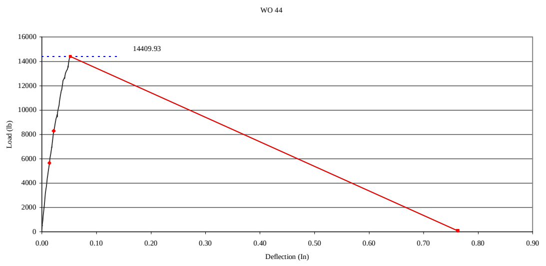
0.02
0.04
0.06
0.08
0.10
0.12
Deflection (In)


WO 44
16000
14000
12000
10000
8000
6000
4000
2000
0
Load (lb)
14409.93
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
Deflection (In)

Dowel Bearing Test Results

Test Number	Yield Value (lb/in^2)	Stiffness (lb/in^3)	Test Number	Yield Value (lb/in^2)	Stiffness (lb/in^3)
WO21 M1	5,710	46,600	WO21 T1	6,560	191,300
WO21 M2	6,140	45,000	WO21 T2	6,140	173,400
WO22 M1	4,350	38,100	WO22 T1	7,430	208,800
WO22 M2	4,230	38,100	WO22 T2	8,020	230,000
WO23 M1	6,200	51,300	WO23 T1	7,400	140,100
WO23 M2	5,750	59,200	WO23 T2	7,320	197,800
WO24 M1	4,210	34,200	WO24 T1	5,790	158,100
WO24 M2	4,510	38,600	WO24 T2	5,950	153,300
WO25 M1	3,600	37,400	WO25 T1	6,050	170,400
WO25 M2	3,660	40,600	WO25 T2	6,160	155,600
WO26 M1	3,920	37,700	WO26 T1	8,700	215,200
WO26 M2	4,670	34,500	WO26 T2	8,320	196,800
WO27 M1	5,080	39,200	WO27 T1	9,660	222,800
WO27 M2	5,150	37,100	WO27 T2	9,270	226,000
WO28 M1	4,170	37,600	WO28 T1	-	-
WO28 M2	3,530	34,000	WO28 T2	6,890	140,500
WO29 M1	3,650	39,100	WO29 T1	5,430	111,100
WO29 M2	3,880	38,100	WO29 T2	5,770	123,900
WO30 M1	3,670	30,600	WO30 T1	8,760	183,100
WO30 M2	3,670	34,000	WO30 T2	8,150	149,200
WO31 M1	4,410	42,500	WO31 T1	5,890	128,600
WO31 M2	4,620	53,900	WO31 T2	6,230	122,800
WO32 M1	7,130	42,600	WO32 T1	10,060	207,000
WO32 M2	7,110	43,400	WO32 T2	8,970	187,500
WO33 M1	4,760	38,200	WO33 T1	6,550	157,100
WO33 M2	4,870	47,600	WO33 T2	6,500	145,600
WO34 M1	4,680	52,700	WO34 T1	5,330	133,200
WO34 M2	4,890	46,300	WO34 T2	5,710	129,800
WO35 M1	5,600	36,800	WO35 T1	7,320	157,000
WO35 M2	5,180	41,900	WO35 T2	7,430	187,200
WO36 M1	5,530	43,300	WO36 T1	7,160	152,200
WO36 M2	5,550	28,600	WO36 T2	7,130	163,100
WO37 M1	4,540	37,700	WO37 T1	5,170	118,300
WO37 M2	4,250	43,500	WO37 T2	5,610	120,900
WO38 M1	4,460	34,800	WO38 T1	7,410	167,100
WO38 M2	4,250	41,000	WO38 T2	7,220	172,500
WO39 M1	4,110	29,000	WO39 T1	7,200	150,200
WO39 M2	4,240	36,600	WO39 T2	7,000	181,200
WO40 M1	5,450	49,800	WO40 T1	6,240	114,300
WO40 M2	5,580	53,000	WO40 T2	6,490	158,100
WO41 M1	4,500	39,600	WO41 T1	6,710	150,600
WO41 M2	4,740	32,900	WO41 T2	6,410	163,100
WO42 M1	3,380	27,200	WO42 T1	8,630	201,600
WO42 M2	4,190	37,500	WO42 T2	7,530	204,900
WO43 M1	4,720	52,500	WO43 T1	6,150	142,300
WO43 M2	4,790	49,800	WO43 T2	5,730	129,900
WO44 M1	4,870	37,100	WO44 T1	8,290	191,300
WO44 M2	4,920	43,400	WO44 T2	8,390	190,200
Mean	4,730	40,700	Mean	7,070	165,400
St. Dev.	860	7,110	St. Dev.	1,210	32,150
5% Exclusion	3,170	27,810	5% Exclusion	4,860	106,990
COV	0.182	0.175	COV	0.172	0.194
K	1.815	1.815	K	1.818	1.818

Specific Gravity and Moisture Contents at the Conclusion of Testing

Member	Moisture Content	S.G.
WO 21 M	11.7%	0.763
WO 22 M	11.4%	0.611
WO 23 M	13.7%	0.767
WO 24 M	14.5%	0.712
WO 25 M	11.9%	0.619
WO 26 M	13.0%	0.651
WO 27 M	12.3%	0.675
WO 28 M	10.1%	0.712
WO 29 M	11.3%	0.570
WO 30 M	14.5%	0.593
WO 31 M	12.4%	0.690
WO 32 M	7.0%	0.793
WO 33 M	11.6%	0.712
WO 34 M	10.5%	0.695
WO 35 M	11.6%	0.695
WO 36 M	12.0%	0.710
WO 37 M	13.3%	0.727
WO 38 M	13.9%	0.650
WO 39 M	13.6%	0.746
WO 40 M	14.4%	0.791
WO 41 M	13.3%	0.671
WO 42 M	14.1%	0.697
WO 43 M	11.1%	0.688
WO 44 M	12.8%	0.610
Mean	12.3%	0.69
St. Dev.	1.7%	0.06
COV	0.14	0.09

Member	Moisture Content	S.G.
WO 21 T	11.3%	0.601
WO 22 T	10.9%	0.768
WO 23 T	12.5%	0.640
WO 24 T	12.0%	0.625
WO 25 T	13.0%	0.619
WO 26 T	13.2%	0.778
WO 27 T	9.8%	0.758
WO 28 T	13.1%	0.601
WO 29 T	9.6%	0.574
WO 30 T	10.8%	0.762
WO 31 T	13.0%	0.585
WO 32 T	9.6%	0.796
WO 33 T	15.3%	0.702
WO 34 T	11.2%	0.574
WO 35 T	10.9%	0.715
WO 36 T	12.6%	0.709
WO 37 T	10.9%	0.546
WO 38 T	10.8%	0.624
WO 39 T	10.7%	0.650
WO 40 T	11.0%	0.652
WO 41 T	10.9%	0.657
WO 42 T	9.9%	0.740
WO 43 T	10.1%	0.700
WO 44 T	12.0%	0.637
Mean	11.5%	0.67
St. Dev.	1.4%	0.07
COV	0.12	0.11

Peg Specific Gravity and Moisture Contents at the Conclusion of Testing

	Peg 1		Peg 2		Average	
Joint	Moisture Content	Specific Gravity	Moisture Content	Specific Gravity	Moisture Content	Specific Gravity
WO 21	5.4%	0.648	8.3%	0.619	6.8%	0.633
WO 22	7.7%	0.824	6.8%	0.831	7.2%	0.827
WO 23	6.0%	0.829	8.5%	0.808	7.2%	0.818
WO 24	15.7%	0.583	6.6%	0.613	11.1%	0.598
WO 25	5.3%	0.634	4.8%	0.620	5.0%	0.627
WO 26	6.0%	0.637	5.4%	0.626	5.7%	0.632
WO 27	6.5%	0.674	16.7%	0.555	11.6%	0.615
WO 28	4.2%	0.679	7.9%	0.670	6.1%	0.674
WO 29	6.1%	0.659	11.9%	0.613	9.0%	0.636
WO 30	7.5%	0.671	5.9%	0.645	6.7%	0.658
WO 31	9.7%	0.646	9.2%	0.670	9.5%	0.658
WO 32	3.6%	0.627	7.8%	0.675	5.7%	0.651
WO 33	8.5%	0.640	6.0%	0.667	7.2%	0.653
WO 34	7.1%	0.599	5.7%	0.593	6.4%	0.596
WO 35	5.1%	0.677	17.0%	0.576	11.0%	0.626
WO 36	8.3%	0.674	7.8%	0.646	8.0%	0.660
WO 37	5.0%	0.662	4.7%	0.621	4.9%	0.641
WO 38	7.4%	0.658	4.7%	0.609	6.1%	0.633
WO 39	8.2%	0.619	8.8%	0.628	8.5%	0.624
WO 40	10.5%	0.614	8.0%	0.663	9.2%	0.638
WO 41	7.3%	0.628	7.0%	0.664	7.1%	0.646
WO 42	5.6%	0.604	6.7%	0.656	6.1%	0.630
				Mean	7.6%	0.653

Appendix D (eastern white pine)

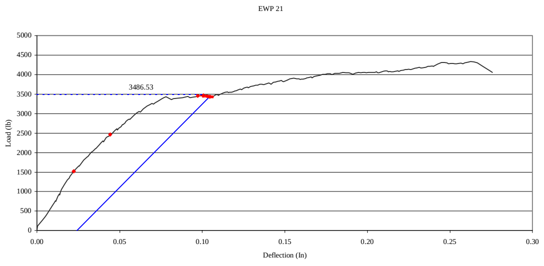
Joint Test Results

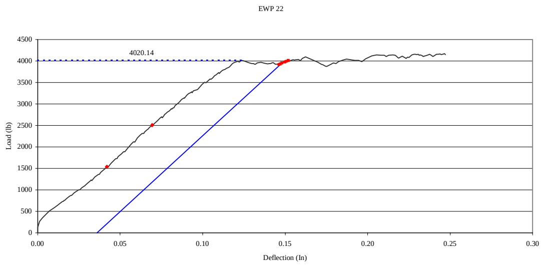
Joint Number	Long Term Load (lb)	Drawbore	Peg Dia. (In)	Yield Disp. (In)	Yield Load (lb)	Stiffness (lb/in)	Ult. Disp (in)	Ult. Load (lb)
EWP21	1000	No	0.75	0.101	3,490	42,700	0.262	4,340
EWP22	1000	No	0.75	0.123	4,020	35,300	0.247	4,170
EWP23	1000	No	0.75	0.118	3,760	35,200	0.327	4,140
EWP24	1000	No	1	0.100	4,480	45,400	0.202	6,340
EWP25	1000	No	1	0.182	5,630	46,900	0.236	6,240
EWP26	1000	No	1	0.176	6,780	65,300	0.220	7,230
EWP27	1000	No	1 (Steel)	0.114	8,070	119,300	0.225	10,120
EWP28	1000	No	1 (Steel)	0.104	7,580	91,400	0.104	7,580
EWP29	1000	No	1 (Steel)	0.091	7,030	92,200	0.091	7,030
EWP30	1000	No	0.75	0.145	4,190	40,700	0.176	4,310
EWP31	1000	No	0.75	0.128	2,740	37,400	0.233	3,440
EWP32	1000	No	0.75	0.140	4,090	36,200	0.160	4,140
EWP33	1000	No	1	0.163	5,400	47,600	0.272	6,500
EWP34	1000	No	1	0.150	5,090	51,700	0.305	7,860
EWP35	1000	No	1	0.129	4,500	47,800	0.346	7,330
EWP36	0	No	0.75	0.119	3,480	38,300	0.142	3,510
EWP37	0	No	0.75	0.112	3,880	35,600	0.307	4,360
EWP38	0	No	0.75	0.107	3,620	38,200	0.321	4,010
EWP39	0	No	1	0.141	4,820	37,500	0.481	8,240
EWP40	0	No	1	0.190	4,500	30,600	0.282	5,650
EWP41	0	No	1	0.163	4,770	46,000	0.371	6,280
EWP42	0	No	1	0.169	6,200	44,900	0.169	6,200
EWP43	0	No	1	0.130	3,910	27,800	0.446	7,910
EWP44	0	No	1	0.153	4,160	32,300	0.229	4,750
EWP45	0	No	0.75	0.131	3,630	38,000	0.138	3,690
EWP46	0	No	0.75	0.162	4,060	34,400	0.247	4,370
EWP47	0	No	0.75	0.130	3,610	29,800	0.130	3,610
EWP48	1000	No	0.75	0.194	2,770	15,600	0.437	3,990

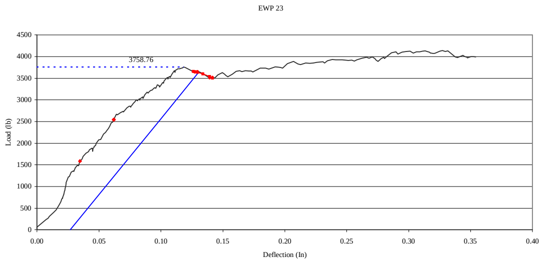
1"					
Mean	0.144	5,530	55,100	0.265	7,000
		0			
Loaded 1000 White Oak Pegs	0.150	5,310	50,800	0.264	6,900
Unloaded White Oak Pegs	0.158	4,730	36,500	0.330	6,500
Loaded 1000 Steel Pegs	0.103	7,560	101,000	0.140	8,200
		0			
Steel	0.103	7,560	101,000	0.140	8,200
White Oak	0.154	5,020	43,600	0.297	6,700
3/4" (EWP 48 is excluded from the following mean calculations)					
Mean	0.126	3,710	36,800	0.224	4,000
Loaded 1000	0.126	3,710	37,900	0.234	4,100
Unloaded	0.127	3,710	35,700	0.214	3,900
mean 1	0.154	5,020	43,600	0.297	6,700
mean 3/4	0.126	3,710	36,800	0.224	4,000

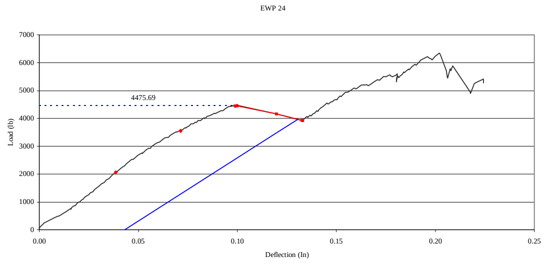
Load-Deflection Plots

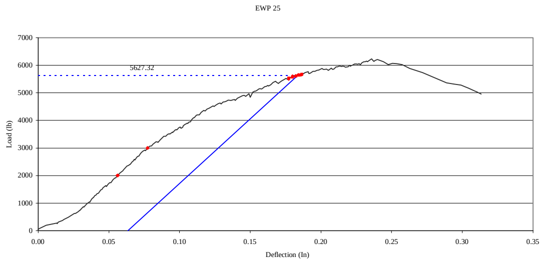
Eastern White Pine

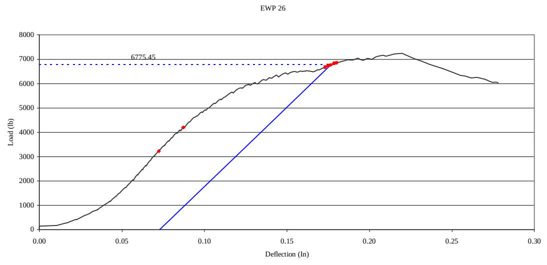


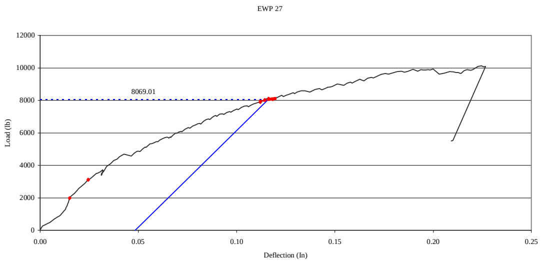











EWP 27
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6000
4000
2000
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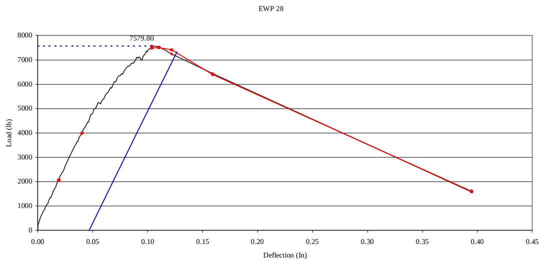
0.00
0.05
0.10
0.15
0.20
0.25
Deflection (In)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2000
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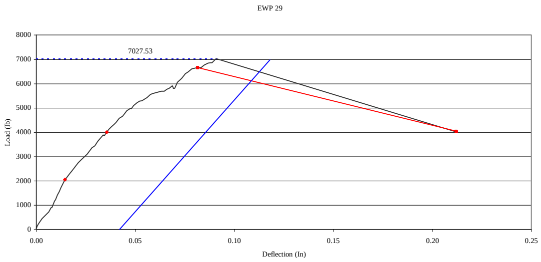
0.20
0.25
0.30
0.35
0.40
0.45
Deflection (In)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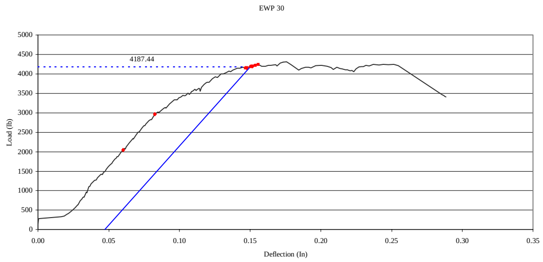
0.25
Deflection (In)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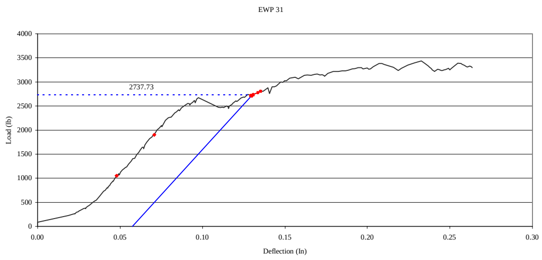
0.15
0.20
0.25
0.30
0.35
Deflection (In)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3000
2500
2000
1500
1000
500
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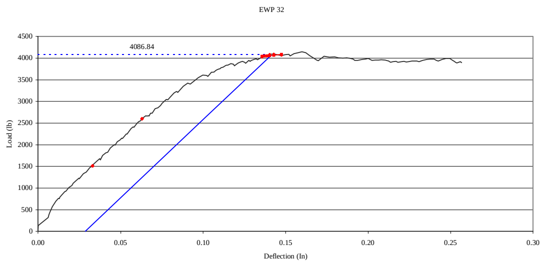
0.20
0.25
0.30
Deflection (In)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2000
1500
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500
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0.05
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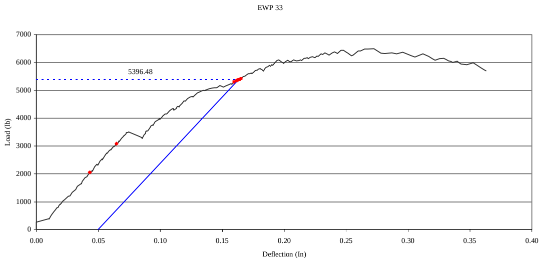
0.15
0.20
0.25
0.30
Deflection (In)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4000
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1000
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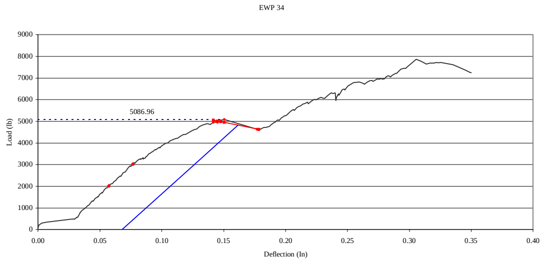
0.20
0.25
0.30
0.35
0.40
Deflection (In)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7000
6000
5000
4000
3000
2000
1000
0
Load (lb)
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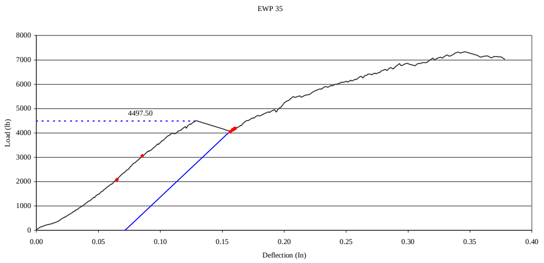
0.15
0.20
0.25
0.30
0.35
0.40
Deflection (In)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2000
3000
4000
5000
6000
7000
8000
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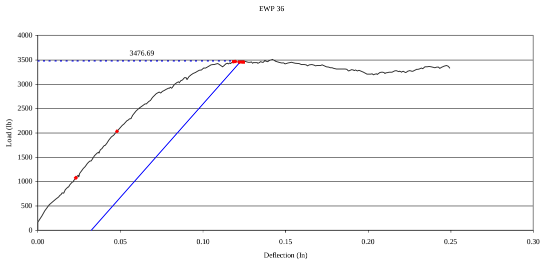
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
Deflection (In)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2500
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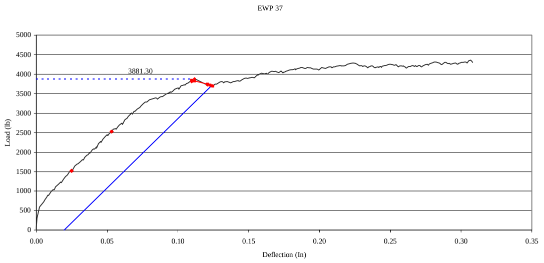
3500
4000
0.00
0.05
0.10
0.15
0.20
0.25
0.30
Deflection (In)


EWP 37
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Load (lb)
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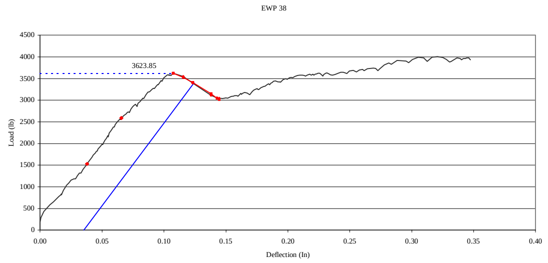
1000
1500
2000
2500
3000
3500
4000
4500
5000
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
Deflection (In)


EWP 38
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Load (lb)
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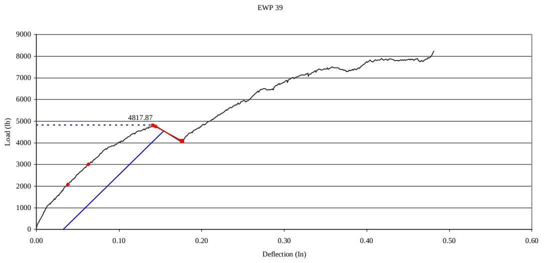
1000
1500
2000
2500
3000
3500
4000
4500
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
Deflection (In)


EWP 39
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Load (lb)
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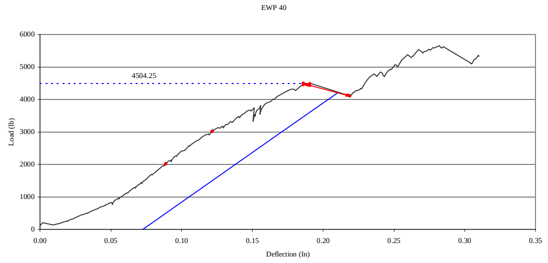
1000
2000
3000
4000
5000
6000
7000
8000
9000
0.00
0.10
0.20
0.30
0.40
0.50
0.60
Deflection (In)


EWP 40
4504.25
Load (lb)
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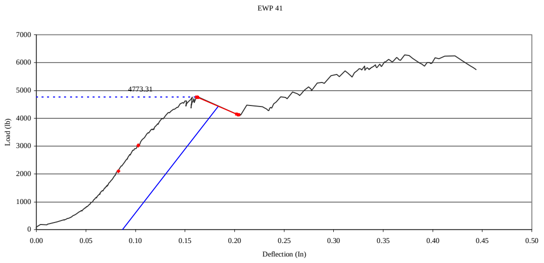
1000
2000
3000
4000
5000
6000
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
Deflection (In)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Load (lb)
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4000
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7000
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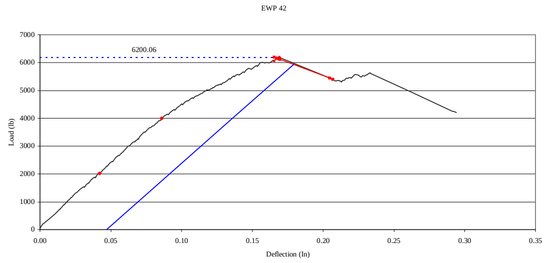
0.20
0.25
0.30
0.35
0.40
0.45
0.50
Deflection (In)


EWP 42
6200.06
Load (lb)
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1000
2000
3000
4000
5000
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7000
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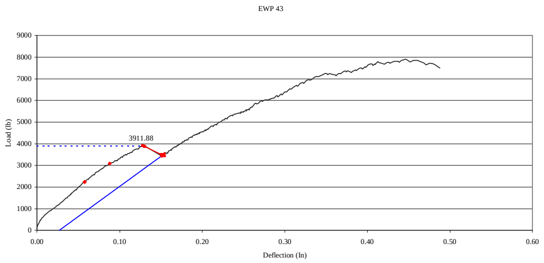
0.15
0.20
0.25
0.30
0.35
Deflection (In)


EWP 43
3911.88
Load (lb)
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1000
2000
3000
4000
5000
6000
7000
8000
9000
Deflection (In)
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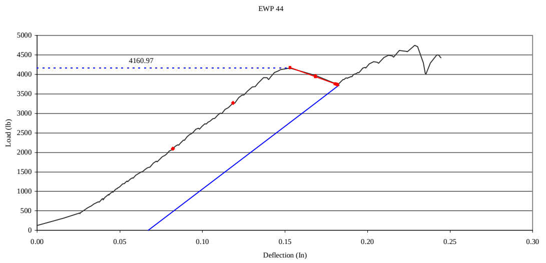
0.10
0.20
0.30
0.40
0.50
0.60


EWP 44
4160.97
Load (lb)
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500
1000
1500
2000
2500
3000
3500
4000
4500
5000
Deflection (In)
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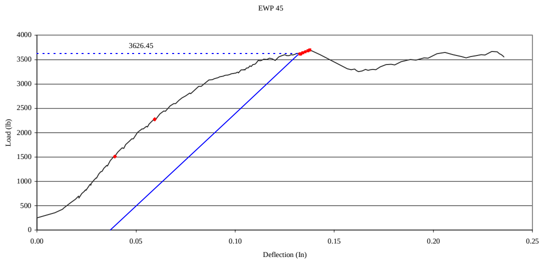
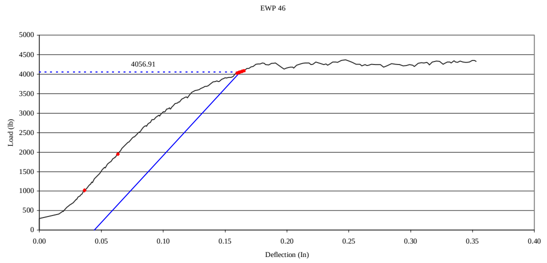
0.05
0.10
0.15
0.20
0.25
0.30


EWP 45
3626.45
Load (lb)
0
500
1000
1500
2000
2500
3000
3500
4000
Deflection (In)
0.00
0.05
0.10
0.15
0.20
0.25


EWP 46
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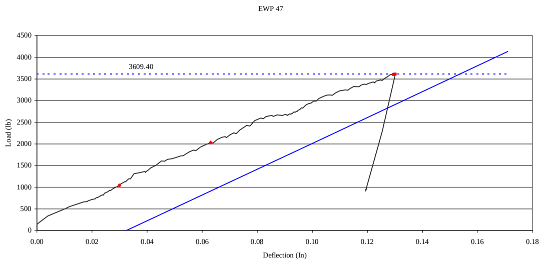
Load (lb)
0
500
1000
1500
2000
2500
3000
3500
4000
4500
5000
Deflection (In)
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40


EWP 47
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Load (lb)
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500
1000
1500
2000
2500
3000
3500
4000
4500
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0.02
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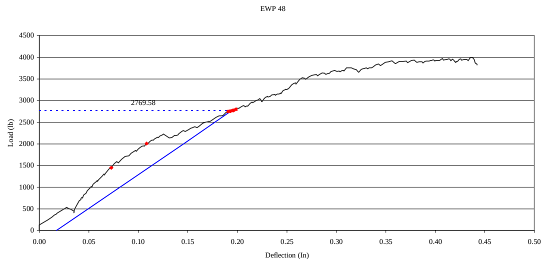
0.06
0.08
0.10
0.12
0.14
0.16
0.18


EWP 48
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Load (lb)
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500
1000
1500
2000
2500
3000
3500
4000
4500
Deflection (In)
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50

Dowel Bearing Test Results

Test Number	Yield Value (lb/in^2)	Stiffness (lb/in^3)
EWP21 M1	1,940	17,100
EWP21 M2	1,840	16,800
EWP22 M1	1,760	13,400
EWP22 M2	1,610	13,600
EWP23 M1	1,930	22,700
EWP23 M2	2,160	22,000
EWP24 M1	1,380	15,500
EWP24 M2	1,420	15,700
EWP25 M1	1,850	15,000
EWP25 M2	1,680	15,800
EWP26 M1	2,040	14,600
EWP26 M2	1,840	14,500
EWP27 M1	1,510	12,600
EWP27 M2	1,640	10,000
EWP28 M1	1,590	14,000
EWP28 M2	1,660	13,900
EWP29 M1	2,110	15,700
EWP29 M2	2,250	14,900
EWP30 M1	2,100	21,500
EWP30 M2	2,120	20,000
EWP31 M1	1,640	15,100
EWP31 M2	1,690	12,300
EWP32 M1	1,630	12,100
EWP32 M2	1,560	12,400
EWP33 M1	2,060	18,700
EWP33 M2	2,020	16,800
EWP34 M1	2,320	29,400
EWP34 M2	1,800	28,100
EWP35 M1	1,930	13,600
EWP35 M2	1,820	13,000
EWP36 M1	1,920	18,500
EWP36 M2	1,940	17,000
EWP37 M1	1,680	14,100
EWP37 M2	1,580	15,600
EWP38 M1	1,590	13,000
EWP38 M2	1,700	14,700
EWP39 M1	2,130	18,600
EWP39 M2	2,090	16,400
EWP40 M1	2,070	16,400
EWP40 M2	1,840	15,100
EWP41 M1	1,440	12,700
EWP41 M2	1,920	15,100
EWP42 M1	1,710	21,100
EWP42 M2	2,570	21,500
EWP43 M1	2,100	20,200
EWP43 M2	2,180	18,500
EWP44 M1	2,070	18,600
EWP44 M2	1,770	16,500
EWP45 M1	2,470	17,100
EWP45 M2	2,430	19,800
EWP46 M1	2,370	22,000
EWP46 M2	2,360	22,700
EWP47 M1	1,400	15,100
EWP47 M2	1,410	13,700
EWP48 M1	1,840	14,200
EWP48 M2	2,050	16,600
Mean	1,890	16,700
St. Dev.	290	3,800
5% Exclusion	1,360	9,800
COV	0.156	0.229
K	1.801	1.801

Test Number	Yield Value (lb/in^2)	Stiffness (lb/in^3)
EWP21 T1	3,410	50,100
EWP21 T2	3,700	56,700
EWP22 T1	4,420	73,800
EWP22 T2	3,920	72,600
EWP23 T1	4,100	67,600
EWP23 T2	4,440	71,000
EWP24 T1	3,580	43,300
EWP24 T2	3,710	57,300
EWP25 T1	Error	-
EWP25 T2	4,520	134,600
EWP26 T1	5,320	94,500
EWP26 T2	4,620	69,300
EWP27 T1	4,530	75,800
EWP27 T2	4,820	76,700
EWP28 T1	Error	-
EWP28 T2	4,940	67,500
EWP29 T1	3,370	57,600
EWP29 T2	2,910	41,700
EWP30 T1	4,780	80,400
EWP30 T2	4,910	94,800
EWP31 T1	4,720	84,500
EWP31 T2	4,420	81,300
EWP32 T1	5,140	76,400
EWP32 T2	Error	-
EWP33 T1	4,770	79,100
EWP33 T2	5,290	91,400
EWP34 T1	3,770	73,300
EWP34 T2	2,950	79,700
EWP35 T1	4,410	65,700
EWP35 T2	4,660	67,200
EWP36 T1	4,040	59,800
EWP36 T2	4,170	53,000
EWP37 T1	4,900	82,700
EWP37 T2	4,610	74,300
EWP38 T1	4,340	69,600
EWP38 T2	5,420	91,300
EWP39 T1	3,830	60,200
EWP39 T2	4,070	57,400
EWP40 T1	4,110	64,800
EWP40 T2	3,890	59,500
EWP41 T1	5,760	90,500
EWP41 T2	4,800	72,700
EWP42 T1	4,530	68,500
EWP42 T2	4,460	71,000
EWP43 T1	4,210	73,700
EWP43 T2	4,100	61,700
EWP44 T1	4,890	77,900
EWP44 T2	4,690	67,600
EWP45 T1	5,590	85,000
EWP45 T2	5,820	88,300
EWP46 T1	7,040	177,200
EWP46 T2	6,660	91,900
EWP47 T1	4,580	72,900
EWP47 T2	4,930	84,100
EWP48 T1	4,040	109,100
EWP48 T2	4,380	92,000
Mean	4540	76200
St. Dev.	780	21200
5% Exclusion	3130	38000
COV	0.172	0.278
K	1.805	1.805

Specific Gravity and Moisture Contents at the Conclusion of Testing

Member	Moisture Content	S.G.
EWP 21 M	7.6%	0.324
EWP 22 M	6.5%	0.320
EWP 23 M	5.3%	0.360
EWP 24 M	7.2%	0.359
EWP 25 M	3.9%	0.346
EWP 26 M	5.8%	0.384
EWP 27 M	6.0%	0.347
EWP 28 M	8.0%	0.326
EWP 29 M	7.3%	0.386
EWP 30 M	6.5%	0.405
EWP 31 M	7.1%	0.322
EWP 32 M	9.0%	0.357
EWP 33 M	7.8%	0.376
EWP 34 M	7.8%	0.341
EWP 35 M	5.1%	0.314
EWP 36 M	6.5%	0.372
EWP 37 M	6.3%	0.360
EWP 38 M	5.8%	0.311
EWP 39 M	7.0%	0.361
EWP 40 M	8.6%	0.337
EWP 41 M	7.8%	0.371
EWP 42 M	7.0%	0.425
EWP 43 M	7.2%	0.359
EWP 44 M	6.1%	0.315
EWP 45 M	7.9%	0.412
EWP 46 M	9.0%	0.359
EWP 47 M	5.9%	0.298
EWP 48 M	7.0%	0.329
Mean	6.9%	0.35
St. Dev.	1.2%	0.03
COV	0.17	0.09

Member	Moisture Content	S.G.
EWP 21 T	5.3%	0.312
EWP 22 T	7.5%	0.356
EWP 23 T	7.0%	0.331
EWP 24 T	5.3%	0.282
EWP 25 T	8.0%	0.390
EWP 26 T	9.0%	0.331
EWP 27 T	7.7%	0.390
EWP 28 T	7.5%	0.308
EWP 29 T	9.1%	0.289
EWP 30 T	7.4%	0.309
EWP 31 T	7.8%	0.335
EWP 32 T	8.1%	0.348
EWP 33 T	6.6%	0.401
EWP 34 T	6.4%	0.310
EWP 35 T	9.4%	0.333
EWP 36 T	10.4%	0.334
EWP 37 T	9.5%	0.364
EWP 38 T	6.4%	0.355
EWP 39 T	5.7%	0.286
EWP 40 T	7.3%	0.315
EWP 41 T	7.3%	0.428
EWP 42 T	6.7%	0.344
EWP 43 T	5.7%	0.350
EWP 44 T	6.3%	0.342
EWP 45 T	6.9%	0.382
EWP 46 T	6.2%	0.393
EWP 47 T	7.0%	0.399
EWP 48 T	7.7%	0.340
Mean	7.3%	0.34
St. Dev.	1.3%	0.04
COV	0.18	0.11

Peg Specific Gravity and Moisture Contents at the Conclusion of Testing

	Peg 1		Peg 2		Average	
Joint	Moisture Content	Specific Gravity	Moisture Content	Specific Gravity	Moisture Content	Specific Gravity
EWP 21	7.7%	0.61	5.4%	0.73	6.5%	0.67
EWP 22	8.1%	0.65	5.5%	0.79	6.8%	0.72
EWP 23	3.1%	0.63	4.6%	0.69	3.8%	0.66
EWP 24	10.7%	0.77	9.1%	0.79	9.9%	0.78
EWP 25	9.3%	0.63	9.2%	0.80	9.3%	0.71
EWP 26	8.4%	0.79	8.2%	0.80	8.3%	0.79
EWP 30	6.1%	0.62	7.6%	0.73	6.8%	0.68
EWP 31	5.5%	0.71	4.9%	0.84	5.2%	0.77
EWP 32	8.1%	0.62	8.2%	0.61	8.1%	0.62
EWP 33	8.9%	0.62	5.8%	0.84	7.4%	0.73
EWP 34	4.0%	0.86	8.1%	0.78	6.1%	0.82
EWP 35	7.0%	0.81	4.5%	0.87	5.8%	0.84
EWP 36	6.1%	0.61	3.2%	0.64	4.6%	0.62
EWP 37	4.1%	0.72	5.6%	0.74	4.8%	0.73
EWP 38	6.5%	0.73	6.9%	0.70	6.7%	0.72
EWP 39	7.4%	0.77	5.3%	0.77	6.3%	0.77
EWP 40	7.5%	0.63	5.7%	0.74	6.6%	0.68
EWP 41	7.5%	0.75	7.6%	0.79	7.5%	0.77
EWP 42	8.8%	0.65	6.2%	0.81	7.5%	0.73
EWP 43	4.9%	0.78	6.4%	0.81	5.7%	0.79
EWP 44	6.1%	0.78	7.2%	0.84	6.6%	0.81
EWP 45	4.3%	0.67	4.7%	0.61	4.5%	0.64
EWP 46	5.0%	0.63	6.8%	0.82	5.9%	0.72
EWP 47	9.4%	0.66	7.5%	0.82	8.4%	0.74
EWP 48	4.1%	0.68	5.8%	0.70	5.0%	0.69
				Mean	6.6%	0.73