PB2000-107443

A Center of Excellence for Rural and Intermodal Transportation

MPC REPORT NO. 00-112

Composite Repair of Timber Structures

D.W. Radford
M.L. Peterson
D. VanGoethem

June 2000

Mountain-Plains Consortium

Colorado State University
Fort Collins, Colorado

North Dakota State University
Fargo, North Dakota

University of Wyoming
Laramie, Wyoming

Utah State University
Logan, Utah

COMPOSITE REPAIR OF TIMBER STRUCTURES

D.W. Radford
M.L. Peterson
D. VanGoethem

Department of Mechanical Engineering
Colorado State University
Fort Collins, Colorado

June 2000

ACKNOWLEDGMENT

This report has been prepared with funds provided by the United States Department of Transportation to the Mountain-Plains Consortium (MPC). The authors also would like to acknowledge the support of the Structural Engineering Laboratory and the Composite Materials, Manufacture and Structures Laboratory at Colorado State University for the use of test and measurement equipment critical to the collection and evaluation of the data presented in this report. Finally, the authors would like to thank Shell Resins and PPG Fiberglass for material donations used in the preparation of test specimens.

DISCLAIMER

ABSTRACT

Existing applications of composite materials technology to the repair of timber structures were evaluated. Based on the prior state of the art, it was determined that a new composites-based repair approach was necessary if the material was to be used to its fullest and if ease of field application was a criterion. The shear spiking approach that has been developed draws heavily on an approach coming into favor in laminated composite aerospace structures, known as "Z"-spiking. As in "Z"-spiking, this new approach to timber repair relies on rods, which in this case are made of pultruded fiberglass, inserted vertically through the beam to enhance the structure's shear performance. Sub-scale tests of this concept have been performed, and stiffness and strength have been monitored. The results indicate that significant damage can be overcome through the insertion of the pultruded fiberglass rods. The results also indicate that localized repair is possible with this approach and that stiffness can be adjusted by varying the number and position of the transverse composite rods.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

EXECUTIVE SUMMARY

Timber bridge infrastructure used throughout the United States is at a stage where the demand for maintenance, repair, and refit is occurring faster than ever before. Nationwide nearly 50 percent of existing timber bridges are classified as structurally deficient. Replacing bridge timbers, especially in railroad applications, means expensive downtime and the large timbers required for repair are becoming evermore scarce. Thus, technological approaches to in-service repair of timber bridge structures is becoming a critical infrastructure issue.

A variety of repair approaches have been suggested over the past decades, with many of these approaches employing modern fiber reinforced composite materials. Unfortunately, the approaches proposed to-date are difficult, if not impractical, to apply in the field during service. Such approaches have included the addition of bonded fiberglass shear plates to the sides of span timbers, encasement of the span timbers in shells and overwraps, and rejuvenation approaches based on the injection of epoxies into the damage. These approaches each bring a drawback, but globally they are impractical to apply to all span timbers without removal from the bridge structure.

The present research suggests a new approach to the use of composite materials technology for repair of timber structural members. Due to the small aspect ratio of the typical railroad bridge span timbers, stiffness degradation can be related to decrease in the beam's shear performance. To rejuvenate the beam the proposed approach focuses on overcoming the loss of shear properties by inserting fiberglass rods from the bottom to the top of the beam, through the areas of damage. The concept includes the injection of an adhesive during the process of insertion, which not only bonds the reinforcing rods in-place, but also fills adjacent cracks. To support this rejuvenation approach, scale beam testing was carried out with a variety of reinforcement cases being evaluated. The overall result is extremely positive, with test beams showing strong recovery of flexural properties through the addition of fiberglass shear spikes. Thus, these results suggest that further, full-scale testing should be undertaken to fully develop the repair approach.

1 INTRODUCTION

While timber is ubiquitous as a building material, it is commonly associated with lightweight residential construction. However, in a number of areas of the United States and in a number of applications, timber construction is an important part of the infrastructure. Bridges make an interesting and important point in this regard. In the National Bridge Inventory (NBI), 41,743 timber bridges are included in the inventory with an additional 42,102 steel bridges with timber decks. The number of timber bridges also is a significant percentage of the total bridge inventory in many states. Colorado, Nebraska, Montana, and North Dakota are the four western states with 20 percent or more of the bridge inventory built with timber main spans. In the southern portion of the country, Louisiana is notable with 42.1 percent of the bridge inventory built of timber with Arkansas, Alabama and Mississippi with 20 percent to 35 percent of the bridge inventory with timber main spans. [1]

Other important applications of timber are in the utility and railroads where much of the infrastructure uses timber. Telephone poles are a familiar application of large timber sections and represent a significant installed investment base in a critical application. Similarly timber railroad bridges are common throughout the country for most of the short and many of the longer bridges. Timber bridges are used on all types of rail lines including important main transcontinental rail lines. The large number of these structures makes it critical that the limited maintenance funds available are targeted to structures in greatest need of repair. In both cases many of the installations are between 50 and 100 years old, and are still necessary for daily operation. Replacement of the structures with concrete or steel is not cost effective and repair is becoming increasingly difficult because of limited availability of the large timber sections required for these structures. Repair or replacement-for-cause provides a means with which to maintain safety and reliability of the infrastructure while the railroads and utilities compete in an unregulated environment.[2]

Of the number of total number of timber bridges, more than 47 percent are classified as structurally deficient in the NBI. Based on the methodology of the NBI, deficiency of these bridges is based on visual inspection and may overstate the significance of visual defects. In spite of the reduced load ratings of many of these bridges, the rural locations of them increase the likelihood that overloaded trucks use the bridges on a regular basis. Thus, the lack of failures may indicate excessive conservatism in the evaluation of some of the structures.

1.1 Timber Rail Bridge Structures

While timber is an uncommon material for critical highway bridge structures, rail traffic depends on a large number of long and short timber bridges. Some of the bridges are nearly 100 years old. They have been in continuous use although they typically have been either partially or extensively rebuilt at periodic intervals. On spur lines, maintenance has been less consistent with the result that equipment and personnel may be at risk. Recent changes in the business environment of the railroads have resulted in operational changes, which significantly affect the use of timber railroad bridges. These changes have made evaluation of the safety and reliability of timber rail structures critical to the efficient operation of railroads. A similar situation also exists outside the United States, with raised inspection concerns regarding older timber railroad bridges.[3]

The most important operational change affecting timber has been an increase in the allowable axle loadings for railcars. For many years axle loadings remained constant at 30 tons. Double stack container trains now regularly run 35.7 ton axle loadings with an additional 10-20 percent increase in axle loadings projected by the turn of the century.[4] This increase is due to the competitive environment in which railroads function and due to changes in rail dynamics technology that have allowed trains to operate safely with higher axle loadings. The result is that many aging rail structures undergo a dynamic proof test each time a piece of modern rail equipment passes over a timber bridge. Safety concerns would dictate the replacement of many of these bridges; however, maintenance funding is affected by the same competitive environment, which affects the rest of railroad operations.

Sufficiently conservative bridge design and maintenance could ensure the reliability of these aging timber rail structures. However since this level of conservatism in bridge design and maintenance is unsustainable in the current business climate, alternative strategies that ensure operational safety of the railroad must be identified. The strategies must center around a repair-for-cause approach to bridge maintenance. Structures can no longer be scheduled for a complete rebuild at a predetermined time, instead repair and replacement must be scheduled based on a priority-based decision making process. The bridges, which are unsafe or have significant deterioration, must be identified in a low cost and efficient manner based on a sound technological basis, modern inspection, and evaluation. When repairs are made, critical components of the bridge which have deteriorated must be identified for replacement or retrofit. Since the large timbers required for replacement of these railroad structural members are becoming less readily available and the replacement of these timbers is a time consuming and costly task, in-situ repair strategies are of great interest. Thus, technologies that can enable repair without timber removal are highly desirable. This preliminary study begins to address these needs by considering in-situ repair strategies, utilizing composite materials, for timber rail bridge components.

2 DRAWBACKS OF PRIOR REPAIR STRATEGIES

Several approaches previously have been applied to the repair of wooden timbers. However, these approaches have not been specifically developed for in-situ application. Procedures include the replacement of severely degraded timbers, epoxy repair approaches, the addition of reinforcing plates to the sides of exposed timbers, and the addition of a fiberglass wrap (bandage) around damage locations.[5,6,7,8,9] Unfortunately, the reinforcing plates and the wrap can only be readily applied, in-situ, to exposed timbers which means that many of the structural timbers must be removed prior to repair. Figure 2.1 shows a laboratory single-span timber bridge. The span timbers are seen directly below the ties and are the focus of this research project. Five parallel span timbers are shown in this case, of which only the outside of the outer two timbers would be readily field assessable for the application of shear patches. This accessibility issue would negatively also effect the application of on-site applied composite wraps, and the central timbers would have to be removed for access or replacement.

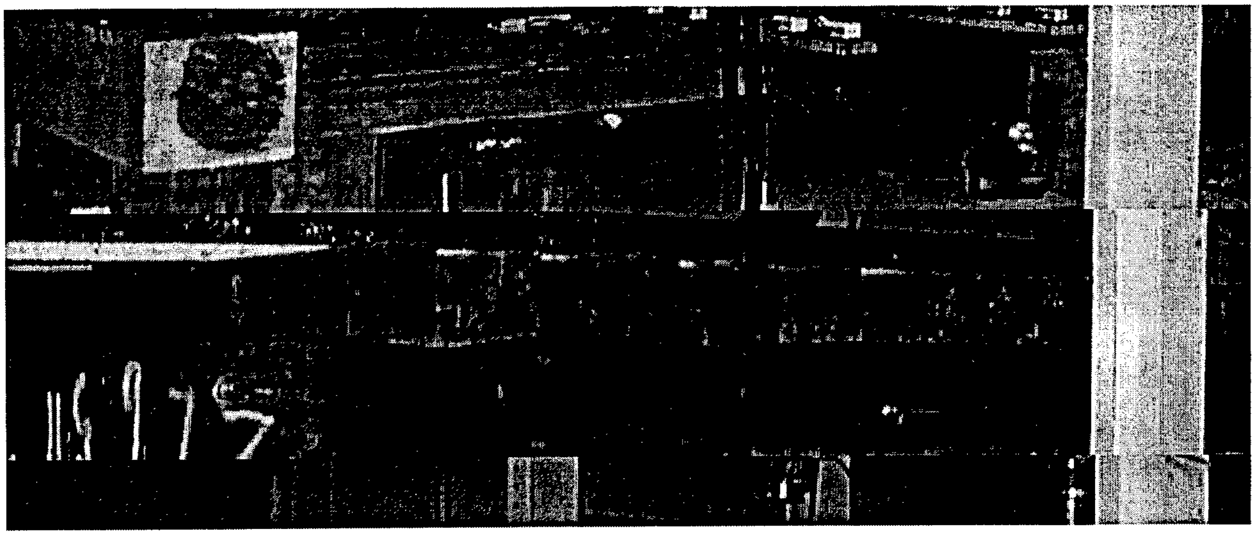
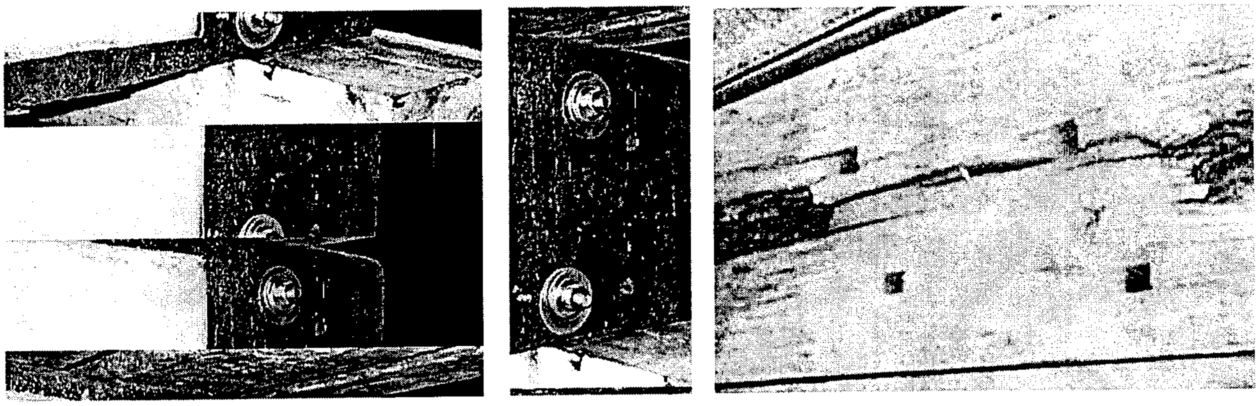


Figure 2.1 Laboratory Single Span Rail Bridge

Replacing timbers means that the structure must be unloaded during the required period of time. In applications such as timber railroad bridges, the large, high quality timber required is becoming more difficult to obtain. Thus, repairs have become much more desirable. Present repair procedures have the disadvantage that the plates or wrap can only be applied, in-situ, to exposed timber. Further, many of the composite material "patches" have been applied without consideration for the type of performance degradation that has occurred. Degradation can show up in a variety of modes, however, the most common mode for the span timbers seems to be a loss of flexural stiffness. This reduction in flexural stiffness often is related to substantial cracking in the middle of the beam as can be seen in Figure 2.2. Since the aspect ratio (length to height) of the span timbers is relatively low, flexural stiffness is strongly related to shear performance of the beam.

Also, the application to the exterior of the wood timber is complicated by preservatives used on the wood, which degrade the bonding performance and by large splits in the timber which are not filled, or healed, during the process.[10] Thus the effectiveness and efficiency of these patches has not been high. Finally, many of the patches are found to not be aesthetically appealing as they are exposed and in open sight.



(a) (b) (c)

Figure 2.2 (a) Composite Shear Plate Concept on Span Timber, (b) Surface Treatment Reduces Bonding Effectiveness, (c) Severe Cracking may Benefit from Epoxy Filling.

Structural performance of the wraps and plates also has been shown to degrade with time. This can be from direct exposure to the sun, in addition to problems related to the ingress of moisture and subsequent problems during freeze-thaw cycles.

3 PROPOSED REPAIR CONCEPT

Based on the evaluation of present approaches to repair of timber structures, and in particular to the repair of railroad span timbers, it was determined that an improved approach to the use of composites was necessary. The primary drawbacks of the existing approaches to use of composites for repair revolve around field installation and on-site cure of composite overwraps. While at least one composite repair concept applies preformed shells, it still requires removal of the timbers from the structure to reinforce any but the outermost span timbers. Thus, the criteria applied in the development of this new approach to utilizing composites involved the application of preshaped and precured, commercially available, composite shapes to keep cost and quality in check, and the ability to install the repair without disassembly of the wood structure. The remaining criteria related to necessary improvement of the stiffness and strength of the degraded existing structural timbers.

The resulting repair strategy resulted from evaluation of the above criteria and the determination that, for the most part, the loss of stiffness in the span timbers is related to a loss of shear performance as the timber degrades. This is a weakness which also is noted in many aerospace-style laminated composites, and thus, the repair approach evolved from a technique used to enhance the interlaminar shear performance in the aerospace composites industry known as "Z"-spiking. The process of "Z"-spiking involves the insertion of small steel rods through the laminate thickness, thus reinforcing the laminate in shear. Some research has been performed investigating optimal frequency and position of these through thickness spikes.[11,12] To extend the concept of "Z"-spiking to timber beams it seemed important to scale up the size of the shear spike, look for a material with good compatibility, and use a commercially available product.

The mode of operation of this repair strategy addresses the types of failure and structural loadings seen in wood timbers used in railroad bridges. In particular, the beams just below the ties are subject to decay and are difficult to repair and replace. These beams are relatively short with respect to their cross-section and therefore, are subject to failure through the loss of shear carrying capability at the neutral axis (mid-way between the top and bottom) of the beam.

Rather than apply an exposed "patch" to the vertical sides of the beam, which can be quite an inefficient approach to shear stiffening, this report suggests an approach that inserts composite rods from the bottom of the beam to the top. These rods or "shear spikes" are inserted in locations of damage at spacings dictated by the degree of decay, or by the amount of performance enhancement desired. The shear spikes act much as nails attaching multiple 2x4's into a single stiff beam. The shear spikes are completely within the timber, exposed only at the bottom insertion points. This means that aesthetics are unchanged; the bond is improved, since the preservatives applied to the timber only penetrate a short distance from the surface; and the exposure to the elements is minimized.

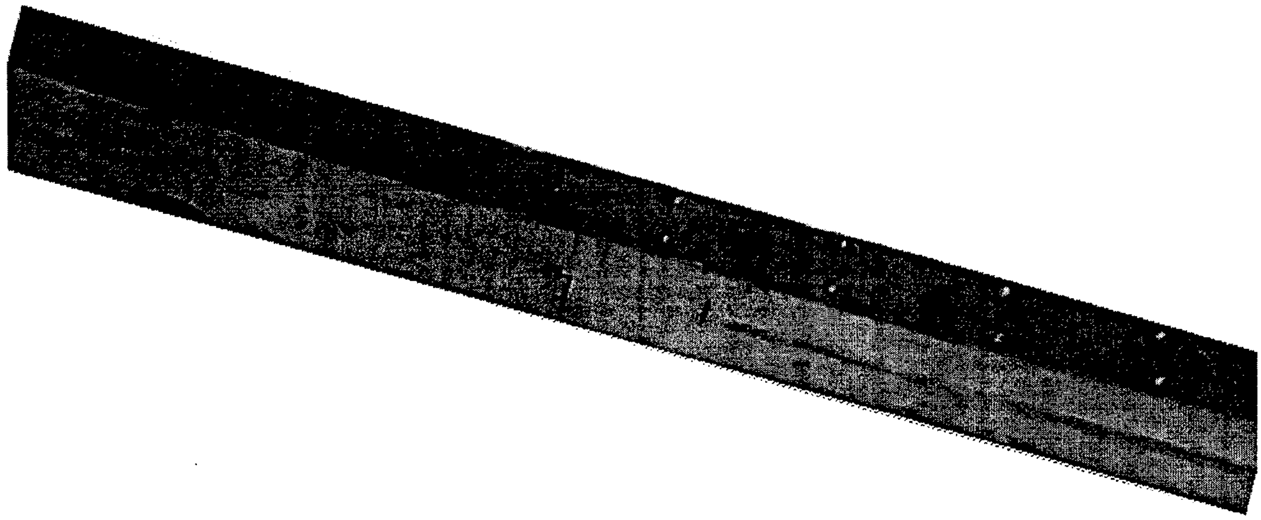


Figure 3.1 Proposed "Shear Spike" Repair Concept

(Inverted view, showing shear spikes on right half repairing split.)

The "shear spikes" are to be fiber reinforced plastic composites produced by pultrusion, with the principal fiber content being in the axial direction. This material has high performance and low cost, but is chosen primarily for its compatibility with wood and its ability to be readily bonded to wood.

The spikes are to be inserted from the bottom of the beams, which enables in-situ application. Holes will be drilled prior to insertion and an adhesive will be injected at the time of insertion. This adhesive is used to improve the load transfer from the wood to the shear spike, to exclude water from the interface, and to spread into gaps in the wood beam. This process constitutes a substantial wood beam rejuvenation.

The research project and flexural test results reported in the following sections involve scale tests of the effect of changing numbers of composite shear spikes and the relative location of these spikes on the beam stiffness. The length to height ratio (11:1) was maintained at a value similar to that of the span timbers of a typical multi-span railroad bridge to match the stress states. Flexural tests were performed using 2x2s and 2x4s of roughly 4' in length.

4 EXPERIMENTAL

To evaluate the shear spike repair concept, flexural testing was initiated using 2x2 and 2x4 dimension lumber and 0.125” diameter pultruded fiberglass rod. These dimensions were chosen to replicate the use of 1” diameter pultruded rods for shear spiking the full-size span timbers. This 1” diameter pultruded rod was chosen as a baseline as it is readily available and has sufficient rigidity and overall structural performance to enable field insertion. Stiffness and failure strength were investigated.

4.1 Test Geometry/Apparatus

Three-point bend testing was used to evaluate scale span timbers. The test specimens were produced from both 2x4 and 2x2 dimension lumber. This means that the nominal width of each of the 48” long beam test specimens was 1.5”. The depth of the beam depended on whether it was a split 2x4, which was nominally 3.5” deep, versus a pair of stacked 2x2s that resulted in a depth of nominally 3.0”. The 3-point bend test dimensions placed the loading points at a 34” span and 1”, or smaller, diameter loading points were used in conjunction with stress distribution plates to reduce the likelihood of loading point damage to the test beams. The resulting length-to-depth ratio is 11.3:1 for the stacked 2x2s and 9.7:1 for the 2x4 cases. All testing was conducted using a 20,000lb Instron leadscrew mechanical tester.

The actual test setup is shown in Figure 4.1. The test placed a stiff I-beam base frame in the Instron to support the test beams and attached the central load point directly to the load cell. The loading points also are shown in the figure, along with the stress distribution points to stop local crushing. At the mid span of each test beam a small nail was inserted to allow the connection of the strain gage string device. The strain gage device is the light-colored box shown on top of the base frame. Data was collected using a PC data acquisition system connected to the load frame and to the strain-measuring device.

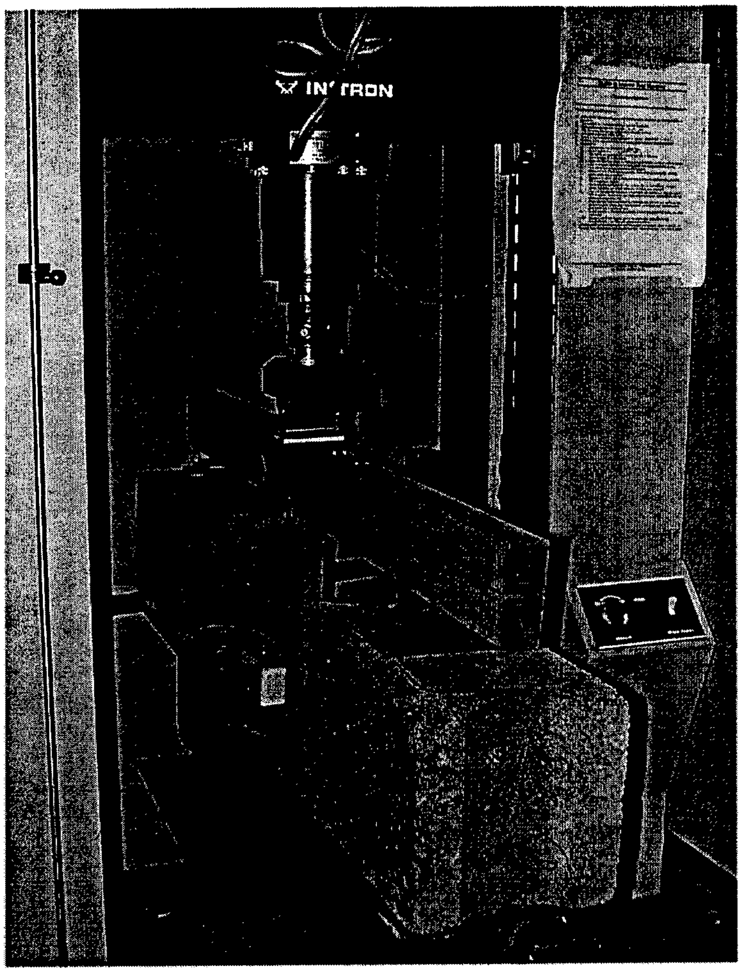


Figure 4.1 3-Point Flexure Test Setup with String Gage.

Load and deflection were measured. Mid-span deflection was measured using a string potentiometer strain gage. Beam performance was computed based on ASTM D790, which uses measured mid-span load and deflection to determine the modulus, strength and strain. ASTM 790 equations follow:

Strength:

$$S = \frac{3PL}{2bd^2} \tag{1}$$

where:

S = stress in outer fibers at midspan, (psi)
P = load at a given point on the load-deflection curve, (lb)
L = support span, (in)
b = width of beam tested, (in)
d = depth of beam tested, (in)

Strain:

$$r = \frac{6Dd}{L^2} \tag{2}$$

where:

r	=	maximum strain in the outer fiber, (in/in)
D	=	maximum deflection of the center of the beam, (in)
L	=	support span, (in)
d	=	depth of beam, (in)

Modulus:

$$E_B = \frac{L^3 m}{4bd^3} \tag{3}$$

where:

E_B	=	modulus of elasticity in bending, (psi)
L	=	support span, (in)
b	=	width of beam tested, (in)
d	=	depth of beam, (in)
m	=	slope of the tangent to the initial straight-line portion of the load-deflection curve, (lb/in of deflection)

Note: *Shear deflection for wood specimens of this low an aspect ratio can reduce the apparent modulus, and thus the modulus numbers presented in this report are meant only for case-to-case comparison.*

4.2 Materials Used

In addition to the dimension lumber used as the test beams, 0.125” diameter pultruded fiberglass rod was used as the shear spikes. Epoxy resin (Shell EPON 813 and V3140) was used as the adhesive in all situations. This epoxy resin was chosen for its reasonably low viscosity, ease of use and room temperature cure.

5 REPAIR CASES EVALUATED AND EXPERIMENTAL RESULTS

The experimental plan to evaluate the shear spike concept, as it applies to short aspect ratio timber structural members, was evaluated through laboratory flexural testing of 2x4s, purposely split along the mid-plane over a portion of the beam length, and paired, stacked 2x2s. 2x4 specimens were evaluated for stiffness and strength in the undamaged configuration as a baseline, unsplit with spikes to evaluate any negative effect of the spiking process, and split with varying numbers of shear spikes. 2x2 specimens were evaluated in an uncoupled, stacked configuration as a baseline; in an epoxy-bonded configuration; a configuration with fiberglass sideplates; and in cases with shear spikes of varying number and position. Due to the inherent variability of wood a large number of specimens of a specified condition generally are tested and properties determined on a statistical basis. For the preliminary study described in this report few repetitions were performed, as absolute performance values were of less interest than were relative performance gains. Thus, the trends described by the data are the focus of this report.

5.1 Stiffness Evaluation - Stacked 2x2 Beams

To evaluate the effect that shear reinforcement would have on timber structural members, specimens were generated by stacking two 2x2s to create a test beam nominally 3" high and 1.5" wide. When stacked without any form of physical attachment between the two 2x2s the stiffness is expected to be low due to the inability of the beam to carry significant shear stress at the neutral axis. As the shear transfer between the two stacked 2x2's is increased, the stiffness is predicted to increase.

The reinforced baseline case of the paired, stacked 2x2's was first tested, followed by cases involving various techniques for increasing the shear carrying capability of the neutral axis. A first set of specimens were tested and evaluated, followed by a second, more refined set of tests.

5.1.1 Preliminary Stacked 2x2 Flexural Testing – Nails as Shear Spikes

Each of the first preliminary group of test beams was tested sequentially for stiffness, starting with no reinforcing shear spikes and then after inserting an additional pair of nails as shear spikes at equal distances from center. Figure 5.1 indicates the relative position of each pair of shear spikes, starting with the first group of pairs, R1, followed by R2, through R6. Holes smaller than the nails were predrilled and the nails were then driven into place. Six flexure specimens were evaluated in this way, with each specimen showing a substantial increase in stiffness as predicted. On average the modulus, as calculated by equation (3) of the previous section, increased from 378ksi to 683ksi as nails were added to the beams.

The greatest single increase was from 360ksi with no nails to 936ksi with a full set of nails as shear spikes.

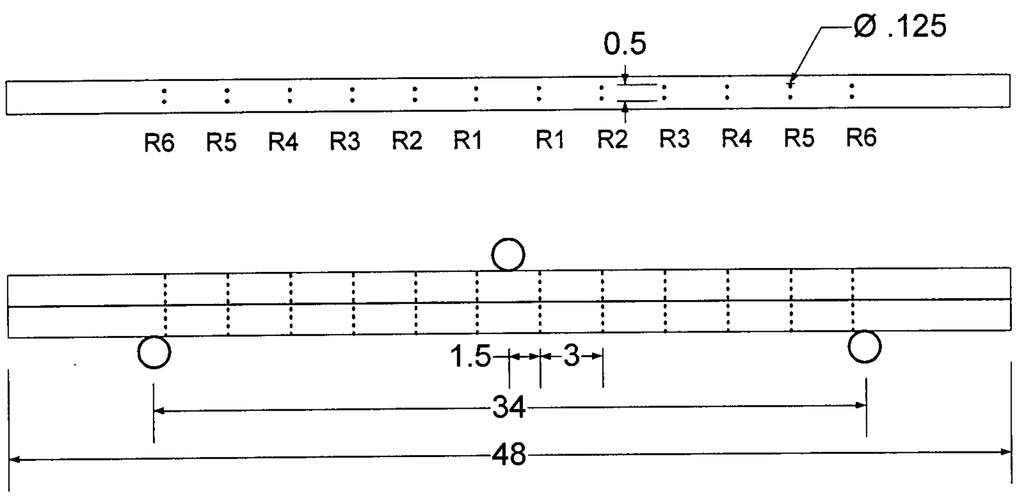


Figure 5.1 Specimen Geometry for Preliminary Stacked/Shear Spiked 2x2 Testing

As suggested earlier, some of the most important results, given the small number of specimens in the test group, are related to the trends apparent in the stiffness, as pairs of shear spikes are added. Figure 5.2 compares the results of the six beam specimens reinforced with nails as shear spikes. In all cases it is apparent that the increased number of pairs of nails leads to increasing modulus.

Since in all the preliminary group of tests reinforcing rods were inserted first nearest the central loading point and then stepwise to the outer ends of the beams, the stiffening effect is somewhat exaggerated near the end of the process. This is because reinforcement near the beam ends is significantly more effective in improving the shear carrying capability. The second set of tests described later in this report reverses the order of insertion, for this reason. Specimen 6, shown in Figure 5.2, was further modified through the addition of five more sets of spikes at positions intermediate to the original six sets. This resulted in a total of 11 sets of spikes, and a further increase in the beam stiffness. This result indicates that the density of shear spikes must be evaluated to determine the appropriate number and position, within a damage area, to achieve the desired beam performance.

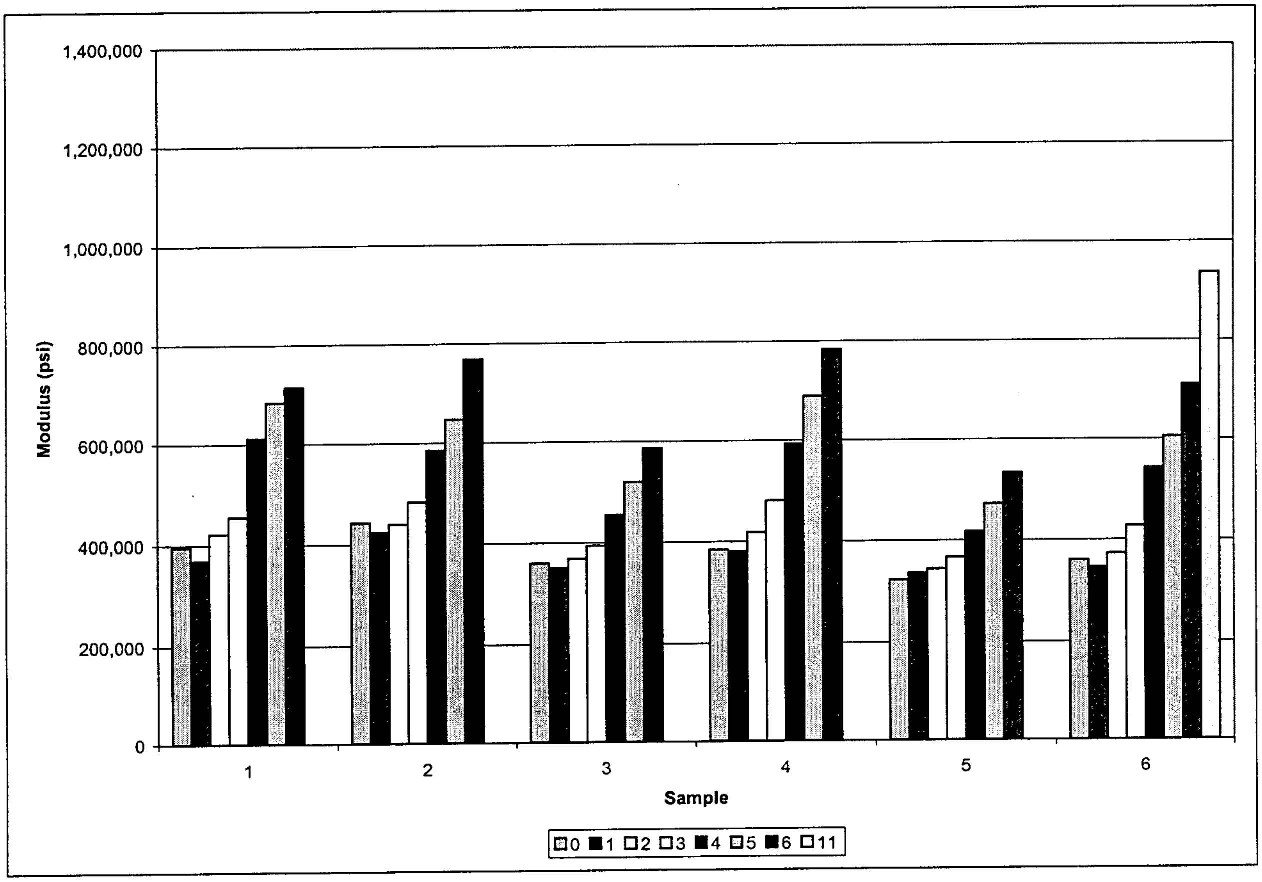


Figure 5.2 Summary of Modulus Related to Number of Pairs of Nails

5.1.2 Preliminary Stacked 2x2 Flexural Testing – Pultruded Fiberglass Rods

Each of this preliminary group of test beams was tested sequentially, starting with no reinforcing shear spikes and then after inserting an additional pair of fiberglass pultruded shear spikes an equal distance from center. Figure 5.1 indicates the relative position of each pair of shear spikes, starting with the first group of pairs, R1, followed by R2, through R6. Unlike the previous case using nails, epoxy was applied to the pultruded fiberglass rods prior to insertion to ensure a strong interface between the shear spikes and the wooden beams. Figure 5.3 summarizes the modulus change with increasing numbers of fiberglass shear spikes. The trends are as seen for the nails as shear spikes, with the modulus increasing consistently as the number of pairs of shear spikes in increased.

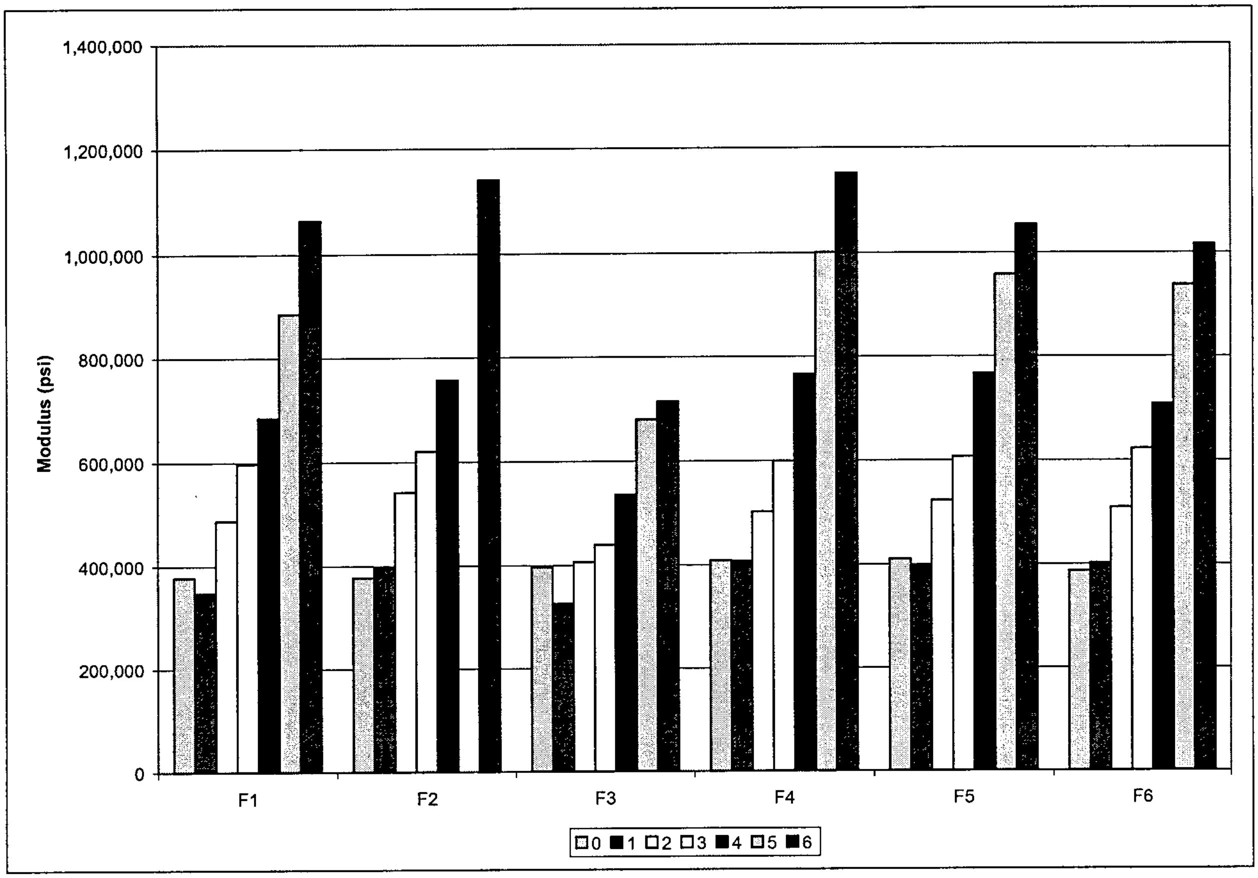


Figure 5.3 Summary of Modulus Related to Number of Pairs of Pultruded Fiberglass Rods

It is interesting to note that in half of the test cases the stiffness shows a slight decrease upon the addition of the first pair of reinforcing spikes. This first set of spikes was inserted nearest the center loading point, as shown in Figure 5.1, which could explain this initial minimal reinforcement since this location of shear reinforcement is of least effect. Thus, the results are showing a combination of the number and position of the shear reinforcement.

Comparing the modulus changes noted for the beams reinforced with epoxy-bonded pultruded fiberglass rods to those reinforced with nails, it is obvious that the percentage increase in modulus is greater with the fiberglass rods. The average increase for the fiberglass spiked beams was from 393ksi for the unreinforced cases to 1,023ksi for full reinforcement. This is more than double the increase noted for the nailed beams, and even considering the small sample size, it seems that this difference is appreciable.

The higher gain in modulus seemed greater than might be expected from simply a better bond of the shear rod to the beam, leading to further evaluation of the specimens. This evaluation was inconclusive; however, it did suggest a concern that a portion of the epoxy applied to the rods was spreading out onto the interface between the two 2x2s. While this is exactly the effect planned in the overall repair strategy it does potentially bias the effect of the shear rods alone. Thus, a second set of tests was planned with techniques proposed to overcome any adhesive at the neutral axis and to evaluate the effect of spike position by inserting rods from the ends of the beams toward the center.

5.1.3 Preliminary Stacked 2x2 Flexural Testing – Epoxy-Bonded

During the preliminary testing, five beam specimens were prepared by bonding the two 2x2s together with the same epoxy used for bonding the pultruded fiberglass shear spikes in place. A large stiffness increase was predicted and the resulting values were consistent with that expectation. The average modulus value of the unbonded, stacked 2x2 beams increased from 295ksi to 1,013ksi when epoxy-bonded at the neutral axis. This was a similar stiffness increase to that measured for the beams reinforced with fiberglass pultruded rods. The stiffness values for the unbonded and bonded case for each of the five specimens is shown in Figure 5.4.

It is instructive to note that variation in moduli measured for the unbonded stacked 2x2s is replicated in specimens after epoxy bonding – the natural variation in stiffness due to the variability of the wood is unchanged after bonding. Thus, the stiffest of the unbonded specimens was the stiffest specimen after bonding as well.

For a complete application of the proposed repair approach for timber rejuvenation, the addition of shear spikes and epoxy is planned. The approach would allow injection of the adhesive, such as the epoxy of these tests, during the insertion of the pultruded fiberglass shear spikes. The adhesive would generate a strong, durable shear bond between the wood and the spikes, and would flow into adjacent cracks, rejuvenating the whole area of damage.

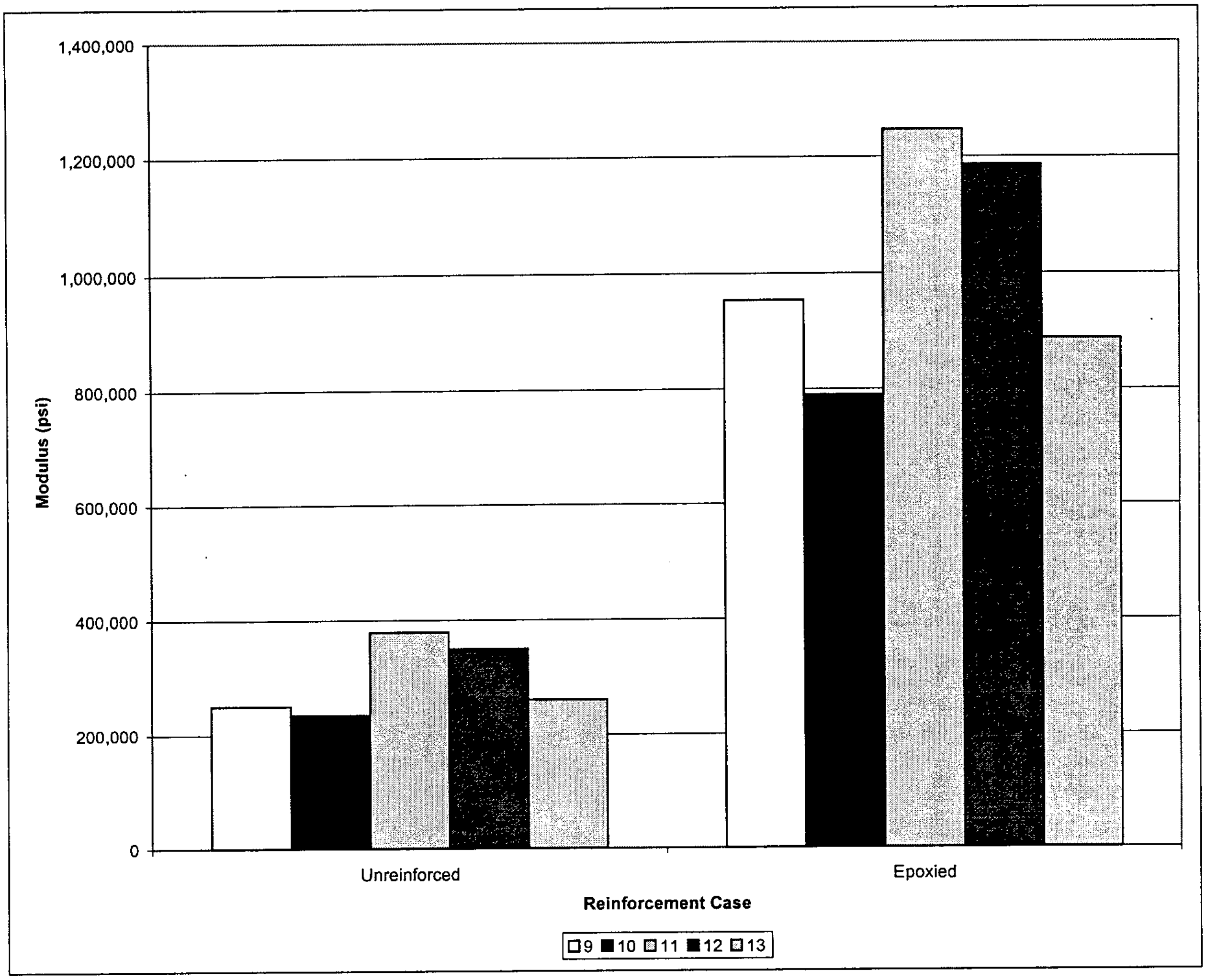


Figure 5.4 Modulus Increase for Epoxy-Bonded 2x2's

5.1.4 Preliminary Stacked 2x2 Flexural Testing – Additional Fiberglass Shear Plates

As a final test case, in the preliminary group of stacked 2x2 tests, five beams were fabricated with fiberglass shear plates bonded to each side. These plates were molded, in a prior operation, on a flat surface and then, once cured, were bonded to each face of the stacked beam specimens. During the bonding process epoxy was allowed to flow into the joint between the top and bottom 2x2. Figure 5.5 shows three of the specimens with the nominally 0.125" thick fiberglass plates in-place. The areas of each fiberglass plate that are ground off near the central loading point were removed to ensure that the plates did not interfere with the load application. The wood beam section was not reduced.

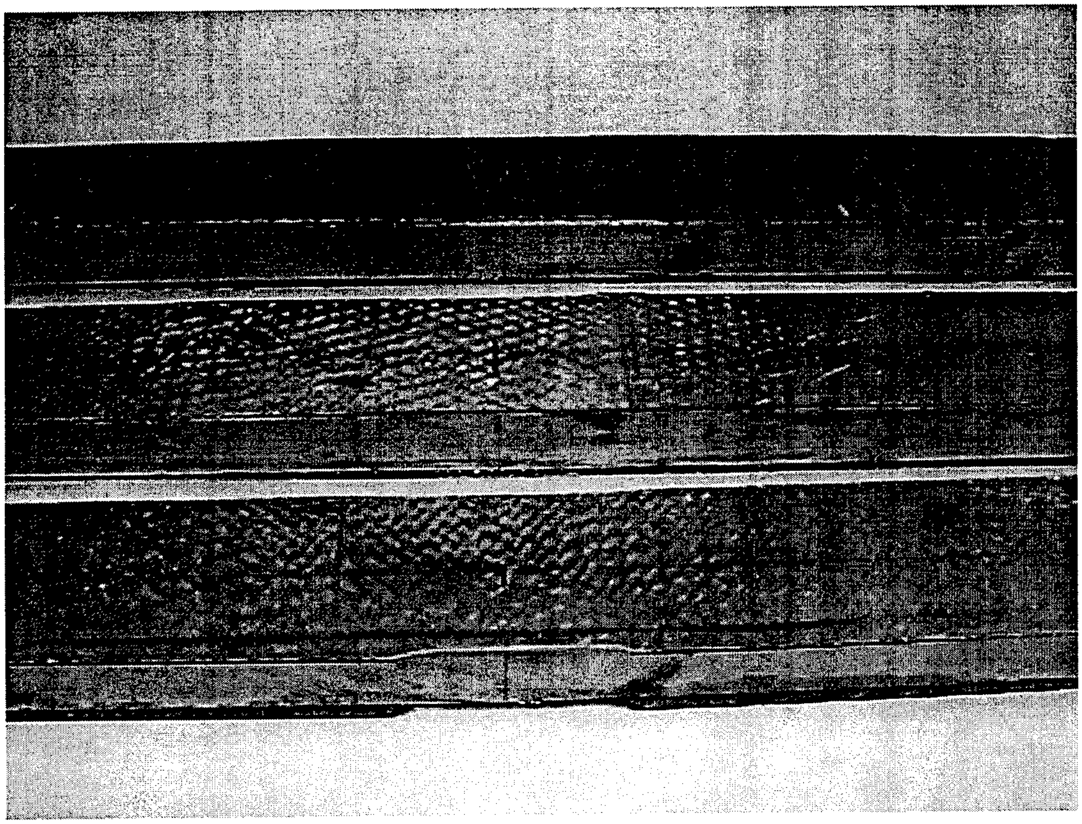


Figure 5.5 Stacked 2x2's Reinforced with Fiberglass Shear Plates

Due to the addition of a substantial amount of fiberglass reinforcement, the stiffness values measured for these specimens were higher than for any of the previous cases. The average stiffness increase was from 379ksi to 1,423ksi. Data for each of the five specimens is shown in Figure 5.6. Again, it is interesting to note that the stiffest unreinforced set of beams remained the stiffest of the group after the fiberglass plates were added. In fact, the complete test group retained the initial stiffness trend after shear plate addition.

The fiberglass shear plates were produced to maximize the shear stiffness, and thus, as is apparent in Figure 5.5, the fiberglass cloth was oriented at 45 degrees to the neutral axis of the beams. This ±45° orientation is critical to high shear carrying capability, while at the same time, minimizing the effect of the fiberglass in other directions. The fiberglass plates are relatively transparent in these specimens, indicating good wet out of the fabric and thus high quality of the shear plates. Note that the details of the wood, the knots, texture and line between the two 2x2s, are clearly visible through the cured shear plates.

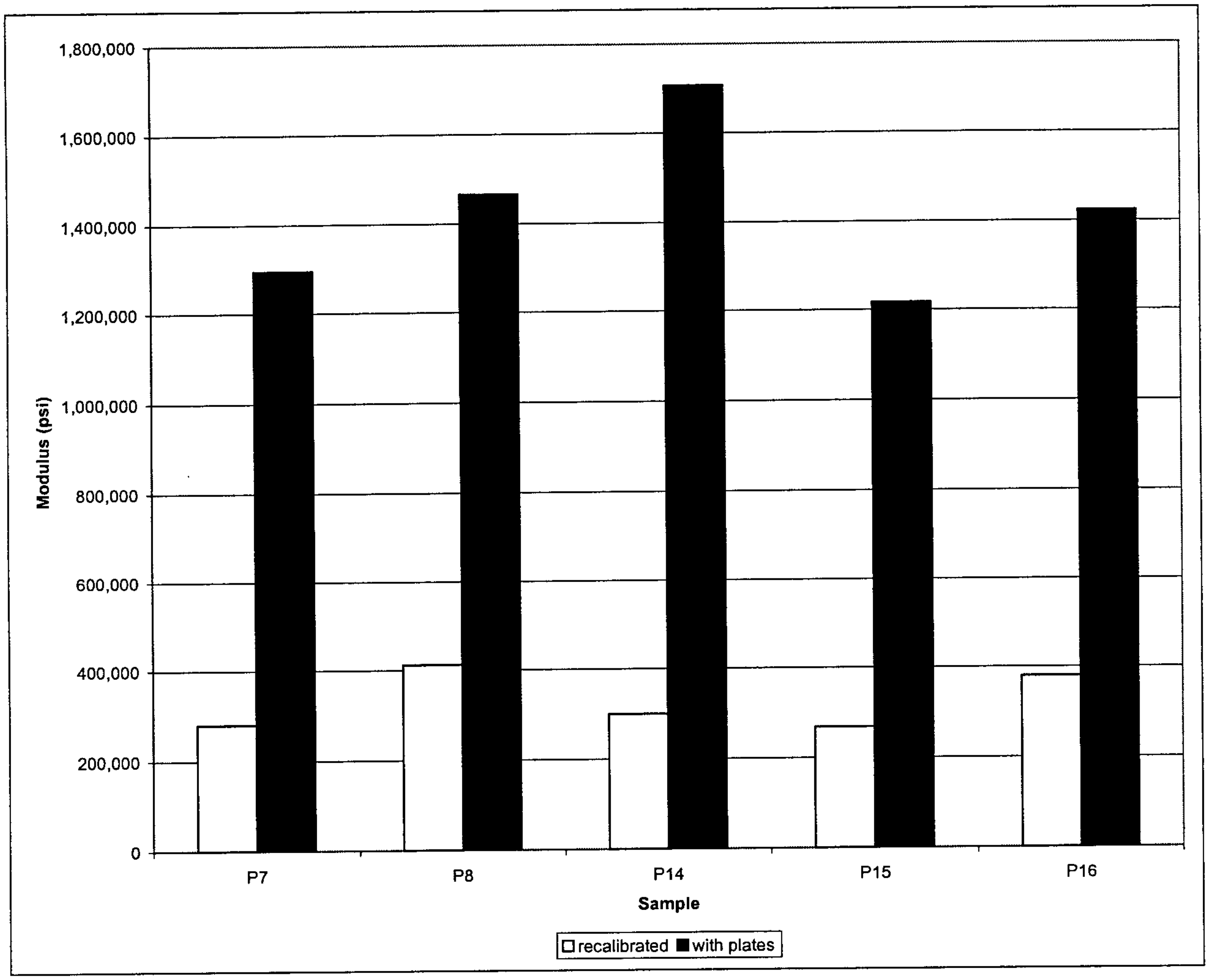


Figure 5.6 Modulus Increase Due to Additional Fiberglass Shear Plates

5.1.5 Stacked 2x2 Flexural Testing – Optimized Specimen Preparation

Based on information gained during the preliminary set of stacked 2x2 flexure tests, a second set of eight stacked 2x2 specimens was planned. This set of tests implemented two principal changes to the specimen preparation procedure which were: (a) wax paper inserted between the two 2x2s to ensure minimal epoxy bonding at the interface, and (b) a change in the order of insertion of the shear spikes to start near the outer support points rather than near the central loading point. Figure 5.7 shows the modified insertion order. All specimens used pultruded fiberglass rods as the shear spike material, and in each case the rods were again bonded into the wood with epoxy. (Insertion force is only moderate and, unlike the case of the nails as shear spikes, the epoxy is critical in carrying the stresses into the rods.)

Rather than test the stiffness of all of these specimens for all numbers of fiberglass pultruded rods, the specimens were produced in a staged fashion. All 2x2s were individually measured for stiffness, as were all stacked pairs of 2x2s and then a first set of pultruded rods was inserted into each stacked pair. All specimens were again tested for stiffness and then a single specimen, chosen at random, was set aside for follow-up strength testing. Each of the remaining seven specimens then had a second set of shear spikes inserted, and once cured, were stiffness tested. Upon completion of stiffness testing, a representative specimen was again set aside to allow strength measurement for the case of two reinforcing pairs. The remaining six specimens were then once again modified through the addition of another set of shear spikes. This procedure continued until the final three specimens had all six pairs of shear spikes inserted.

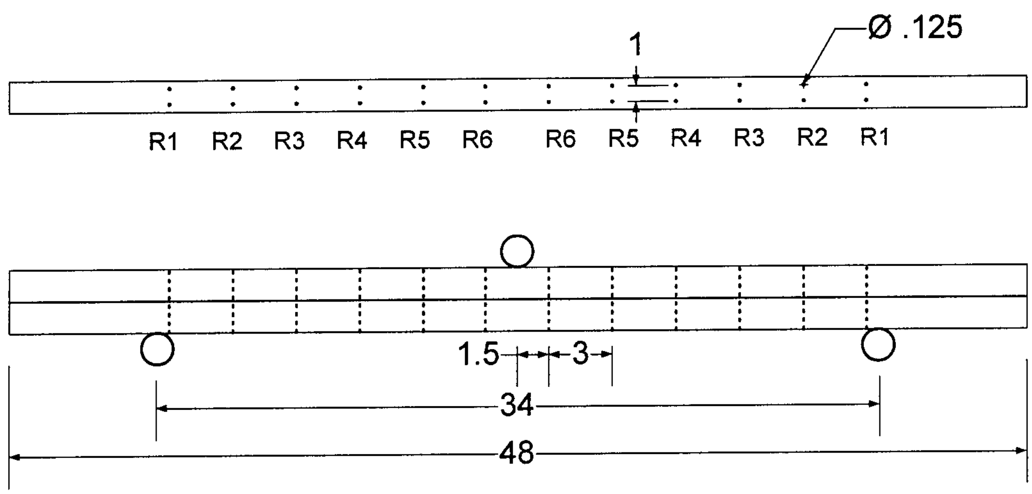


Figure 5.7 Stacked 2x2 Flexure Test Geometry showing Modified Spike Order

Of the resulting three specimens, two were chosen for further modifications, which involved the addition of further fiberglass rods to generate a higher shear spike density. For the first of these increased density specimens a third shear spike was added between each pair of existing rods, and then five additional rows of three rods were inserted, sequentially, in positions intermediate to the initial three-inch spacing. The second specimen was modified in reverse order, by first sequentially inserting paired rods at intermediate locations along the length of the beam, followed by the addition of a third shear spike to each existing row.

The stiffening effect that is observed, related to the increasing number of pairs of shear spikes, is summarized in Figure 5.8. Again, due to the lack of repetitions the trends are much more important than the absolute numerical results. In all cases noticeable stiffness increases were achieved, with the incremental gain in stiffness slowing as more pairs of spikes were inserted. Figure 5.8 also indicates a series of stiffness "rechecks," which were performed at approximately three-day intervals. These rechecks show that a small increase in stiffness may exist over the first six days, related to continued cure of the epoxy. However, this additional stiffness related to cure completion seems to be a relatively small contribution in most cases.

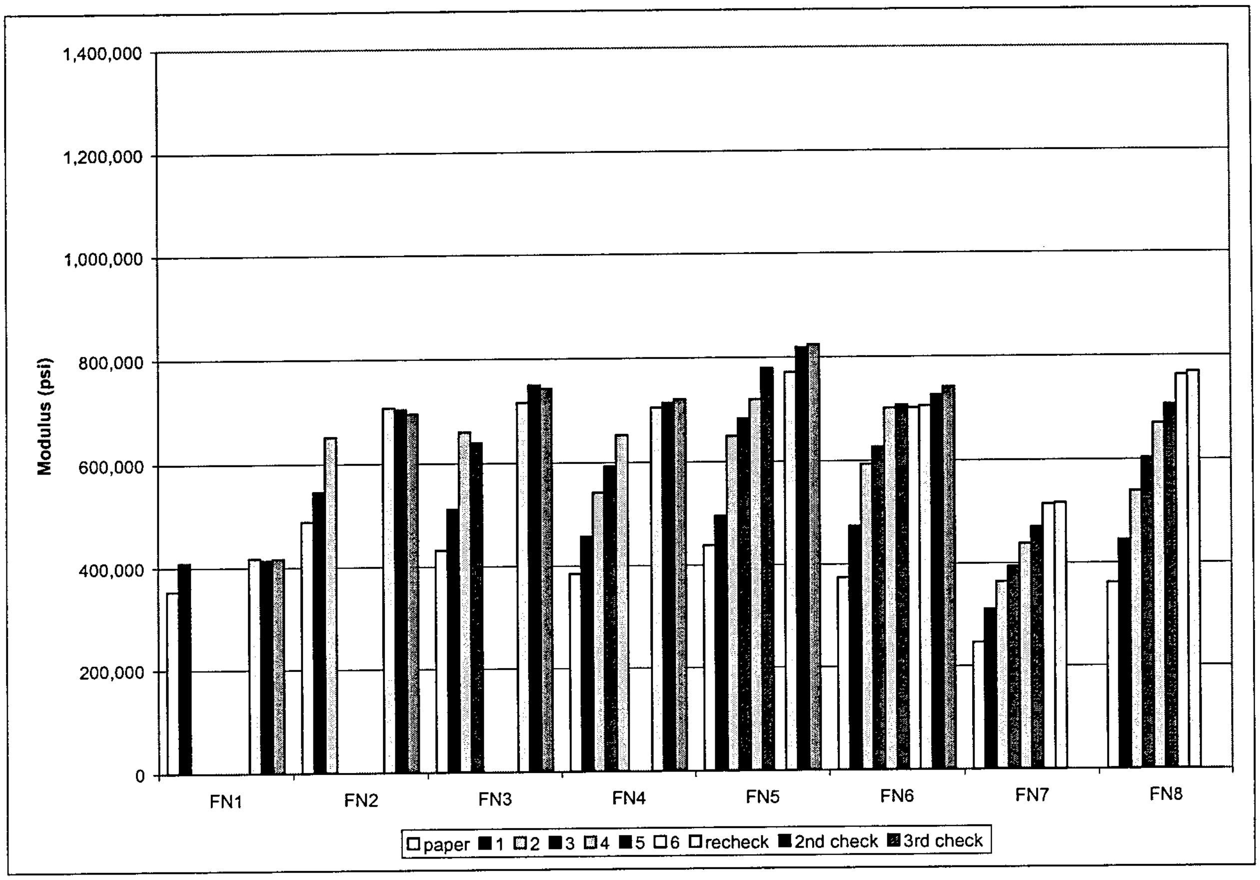


Figure 5.8 Effect of Amount of Shear Reinforcement on Modulus

These stiffness results compare well with the preliminary set of tests performed with pultruded rods as shear spikes; however, the maximum modulus values measured are somewhat lower. This is as expected based on the assumption that during the preliminary test specimen preparation epoxy was allowed to bond the 2x2s together. Further, the pattern of rod insertion was reversed for this second set of tests, which was predicted to show greater shear stiffening for the first sets of rods. It also should be noted that while the final pairs of shear spikes seem to have little effect on the stiffness in this second set of tests, these spikes are placed near the central loading point where shear reinforcement is not efficient. Thus, to evaluate the effectiveness of additional pultruded rods it was necessary to evaluate modified insertion patterns, which would concentrate greater numbers of shear spikes nearer the outer load support points.

Figure 5.9 shows the stiffness results for the specimen in which a third spike was added in the middle of each pair, followed by intermediate rows of three rods. The complete reinforcement cycle is included in this data, showing the stiffness change starting from the unreinforced case. The trend clearly indicates that the additional intermediate rows of three rods, especially those furthest from the center of the beam (mi1, mi2), do have an appreciable effect on the stiffness.

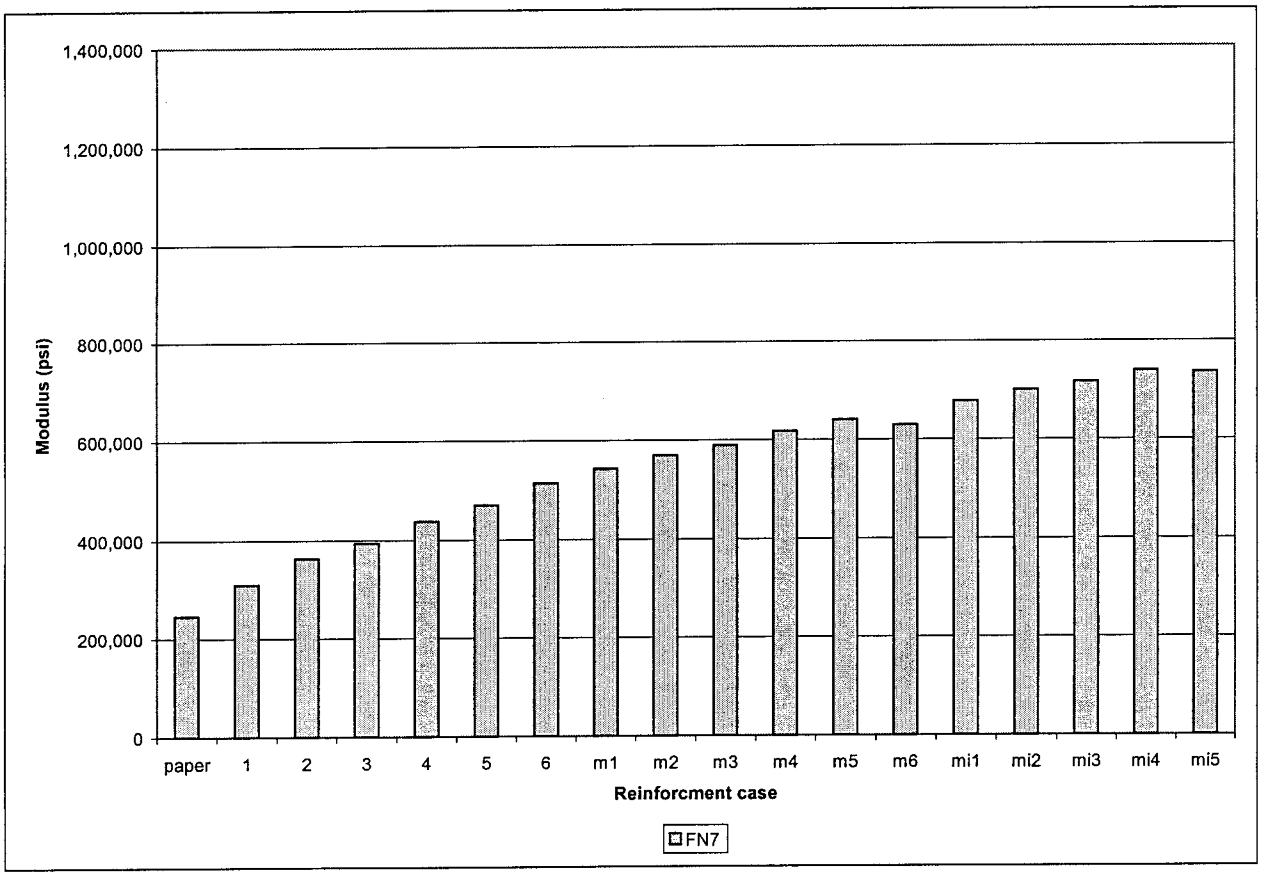


Figure 5.9 Modulus Change with Mid, then Intermediate Shear Spike Additions

The opposite insertion case is shown in Figure 5.10. In this case the ultimate modulus values are higher; however, the initial unreinforced stiffness also is significantly higher. Again, focusing on the trends, it is obvious that the additional shear spikes are effective in increasing the modulus. The ultimate value achieved is approximately 50 percent greater than the stiffness with the original six pairs of rods. In this case the additional intermediate pairs of rods seem to give the greatest increase, while the additional third spike in each row is only slightly less effective. Again, it should be noted that the addition of shear spikes nearest the mid-span of the beam is less effective than the insertion of spikes further from center. More stiffening may be possible by concentrating more of the spikes further from the central loading point.

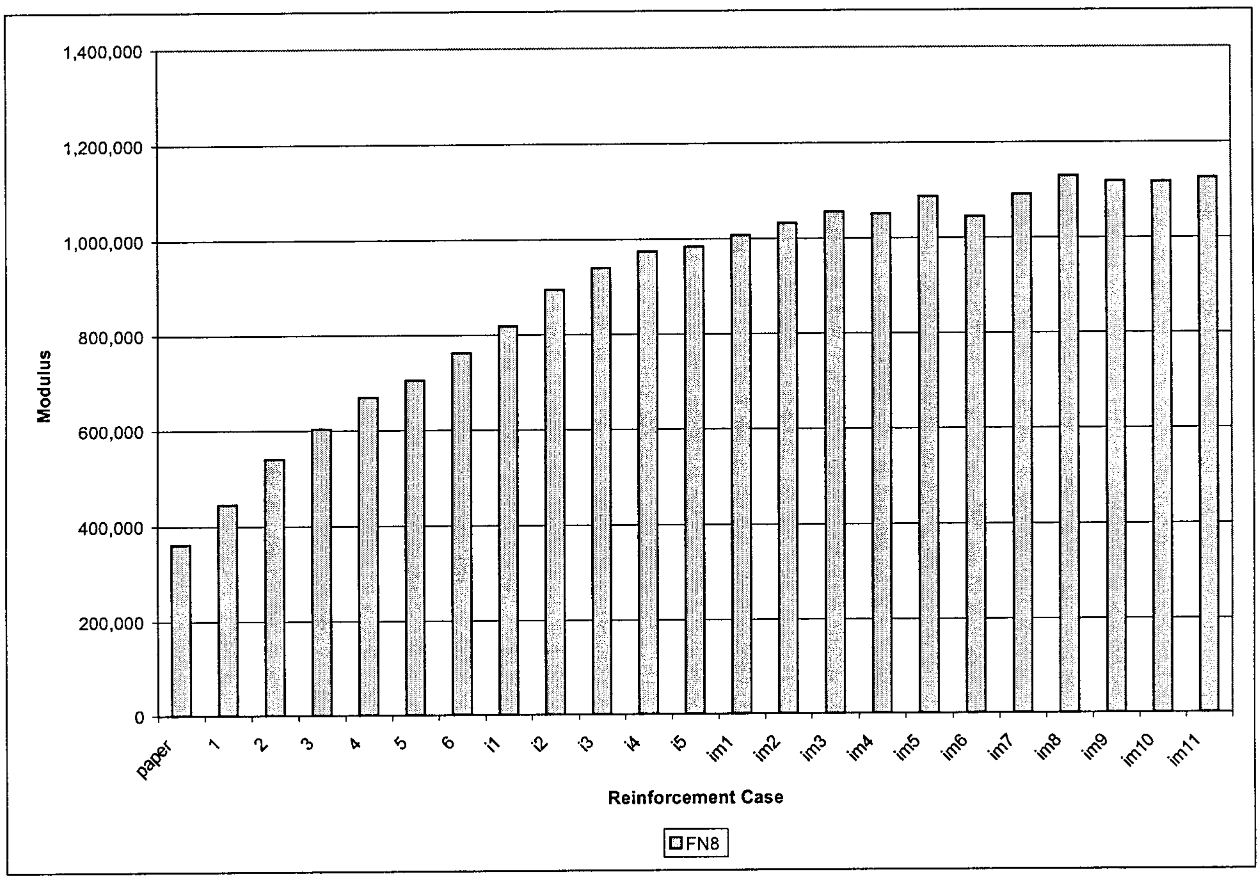


Figure 5.10 Modulus Change with Intermediate, then Mid Shear Spike Additions

It seems that pultruded fiberglass rods can be used as effective shear spikes to increase the stiffness of beams with poor shear carrying capability. It also is apparent that an optimal number and position of these spikes must be determined to most efficiently effect repairs on damaged timber structures.

5.2 Stiffness Evaluation – 2x4 Beams

To evaluate a case that more directly resembles the repair of a damaged timber, 2x4s were cut along the neutral axis, approximately half the length of the beam. These beams were tested prior to the introduction of "damage," immediately after the introduction of damage, and then at various levels of reinforcement of the split end of the beam. The 2x4 beams are of different dimensions from the stacked 2x2 beams, (3.5" high versus 3" high) leading to a slightly lower aspect ratio of approximately 9.7. This difference in aspect ratio from the stacked 2x2 specimens and the actual bridge span timbers was not considered an issue as the 2x4 beams were inserted into the test program as a demonstration of repair of a partially cracked beam. Again, as in the stacked 2x2 testing, the number of repetitions of each case is insufficient for full statistics and thus the resulting trends are much more useful than the absolute modulus values.

In preparation for these "split" 2x4 tests three undamaged test beams were evaluated for change in stiffness with addition of reinforcing spikes. Pairs of spikes were sequentially inserted, as they were inserted in the stacked 2x2 specimens of the previous section, and stiffness measurements were performed at each step. The net result was that no distinct trends were observed. Thus, it is assumed that the addition of shear spikes does neither noticeably degrade nor enhance the stiffness of undamaged 2x4 beams.

The 2x4 beam tests were performed in two sets similar in nature to the two sets described for the stacked 2x2 beams. Thus, the descriptions of the following split 2x4 flexure tests follow the same procedures as the corresponding stacked 2x2 tests described in the previous section.

5.2.1 Preliminary Split 2x4 Flexural Testing – Pultruded Fiberglass Rods

To prepare the 2x4 specimens, each beam was first tested for stiffness in the as-received condition, with no purposely imposed damage. When this test was complete, the beam was split, along the mid-plane, from one end to a distance approximately 2" from the central loading point, as shown in Figure 5.11. Initial attempts were made to "wedge-split" the beams; however, due to the variability of the grain in the specimens it was deemed impractical to generate well-controlled split specimens, as the position of the damage was highly variable. Thus, the procedure was modified through the use of a bandsaw to produce the majority of the mid-plane damage. A saw cut (approximately 0.0625" kirf) was made from one end to roughly 2.5" from the central loading point, along the beam mid-plane. The final 0.5" of damage was then produced using a wedge, resulting in a "sharp" mid-plane flaw.

Once damage was imposed, each of this preliminary group of 2x4 beams was tested, sequentially, starting with no reinforcing shear spikes and then after inserting each additional pair of fiberglass pultruded shear spikes over the area of the damage. Pultruded rods were inserted using the same practice as was used for the stacked 2x2 tests. Rods were coated with epoxy and inserted into pre-drilled 0.125" holes. The epoxy was allowed to cure prior to testing. Figure 5.11 indicates the relative position of each pair of shear spikes, starting with the first group of pairs, R1, followed by R2, through R6. It should be noted that the first set of spikes, R1, is only 1.5" from the beam center, which places them in the undamaged region. This positioning was chosen to match the cases of the stacked 2x2 beam specimens.

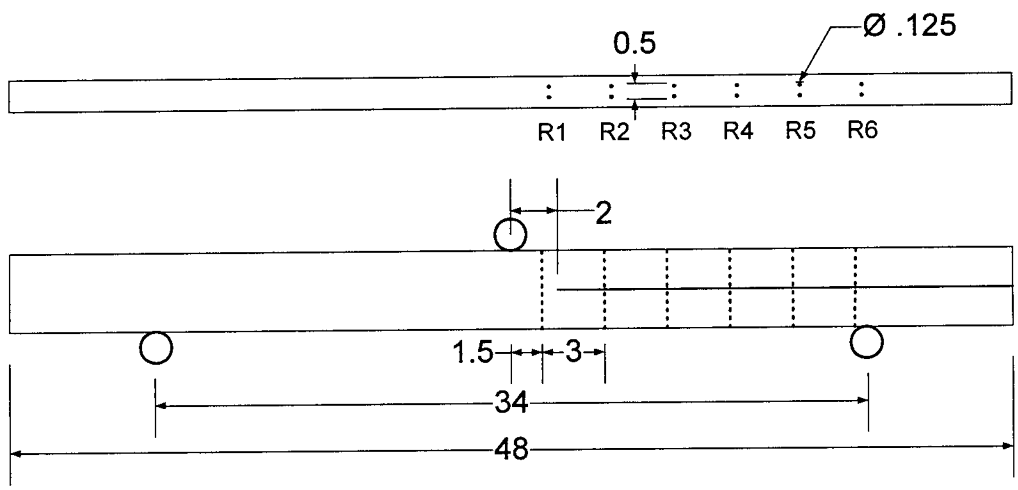


Figure 5.11 Geometry of 2x4s first round (nails and fiberglass rods) saw split

Figure 5.12 summarizes the modulus change with increasing numbers of pairs of fiberglass shear spikes. In general, the "as-split" modulus was reduced to approximately one-half the value for the "undamaged" specimen. In the majority of specimens a significant gain in stiffness was noted after the addition of the fourth pair of shear spikes. The stiffness continued to improve as more pairs of spikes were added. With a full set of pultruded fiberglass reinforcing rods inserted the measured modulus of the repaired beam matched or exceeded the original modulus, prior to "damage," for four of the six beams tested. This ability to directly compare the effect of the shear spike repair to the undamaged modulus of a specific specimen was not possible in the stacked 2x2 tests of the previous section. Thus, these tests indicate that the level of repair, based on stiffness, generated by the pultruded fiberglass shear spikes is effectively 100 percent.

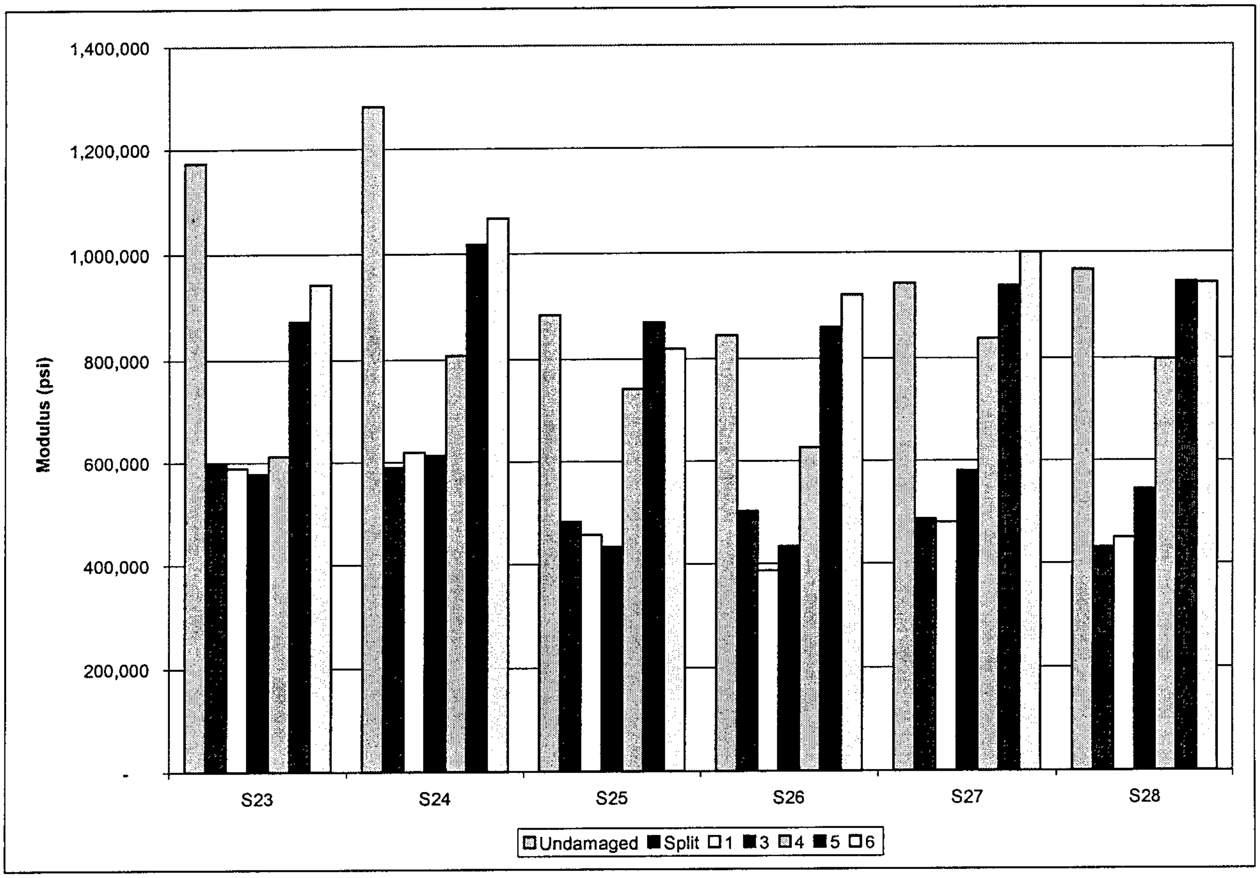


Figure 5.12 Summary of Effect of Reinforcing Spikes on Saw-Split 2x4 Beam Modulus

As with the stacked 2x2 tests, concern arose regarding the possibility of the repair quality being enhanced by excess epoxy bonding the damaged area. The poor effect noted for the first few pairs of reinforcing spikes seemed most likely due to starting near the central loading point, where shear reinforcement is the least effective. Therefore, a second set of split 2x4 tests was developed, which directly parallels the second set of stacked 2x2 tests.

5.2.2 Split 2x4 Flexural Testing – Optimized Specimen Preparation

As in the second set of stacked 2x2 tests, eight additional 2x4 beams were prepared in a modified fashion. This set of tests implemented two principal changes to the specimen preparation procedure which were: (a) wax paper inserted in the saw cut of the "damaged" end of the 2x4 specimens to ensure minimal epoxy bonding at the interface, and (b) a change in the order of insertion of the shear spikes to start near the outer support points rather than near the central loading point. Figure 5.13 shows the revised geometry. All specimens used pultruded fiberglass rods as the shear spike material, and in each case the rods were bonded into the wood with epoxy.

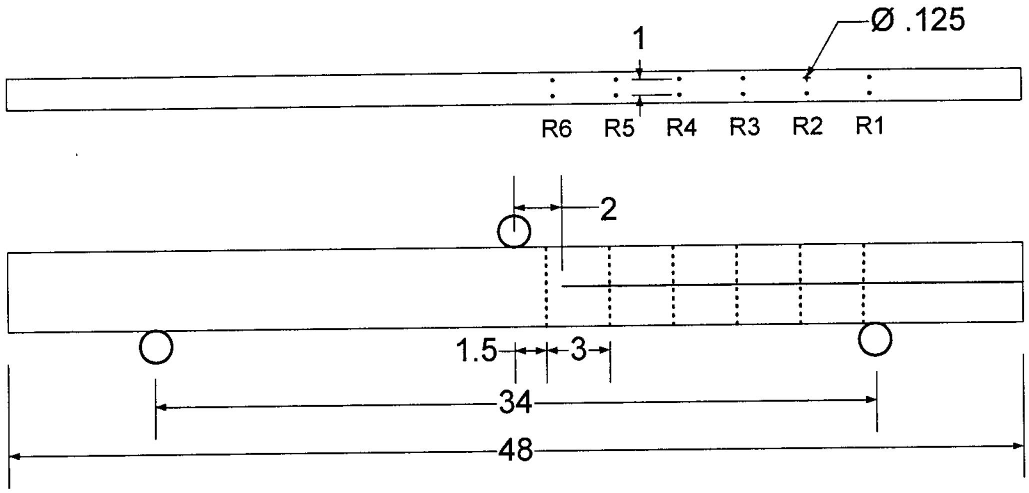


Figure 5.13 Specimen Geometry for 2x4s Showing Modified Spiking Order

Rather than test the stiffness of all of the specimens for all numbers of fiberglass pultruded rods, these specimens were produced in a staged fashion. All specimens were again tested for stiffness and then a single specimen, chosen at random, was set aside for follow-up strength testing. Each of the remaining seven specimens then had a second set of shear spikes inserted, and once cured, was stiffness tested. Upon completion of stiffness testing a representative specimen was again set aside to allow strength measurement for the case of two reinforcing pairs. The remaining six specimens were then once again modified through the addition of another set of shear spikes. This procedure was continued until the final three specimens had all six pairs of shear spikes inserted.

The stiffening effect that is observed, related to the increasing number of pairs of shear spikes, is summarized in Figure 5.14. Again, due to the lack of repetitions the trends are much more important than the absolute numerical results. In all cases noticeable stiffness increases were achieved, with an immediate gain after insertion of the first pair of spikes, followed by an incremental gain in stiffness which slows as more pairs of spikes were inserted closer to the central loading point. Figure 5.14 also indicates a series of stiffness "rechecks" which were performed at approximately three-day intervals. These rechecks show that a small increase in stiffness may exist over the first six days, related to continued cure of the epoxy. However, this additional stiffness related to cure completion seems to be a relatively small contribution in most cases.

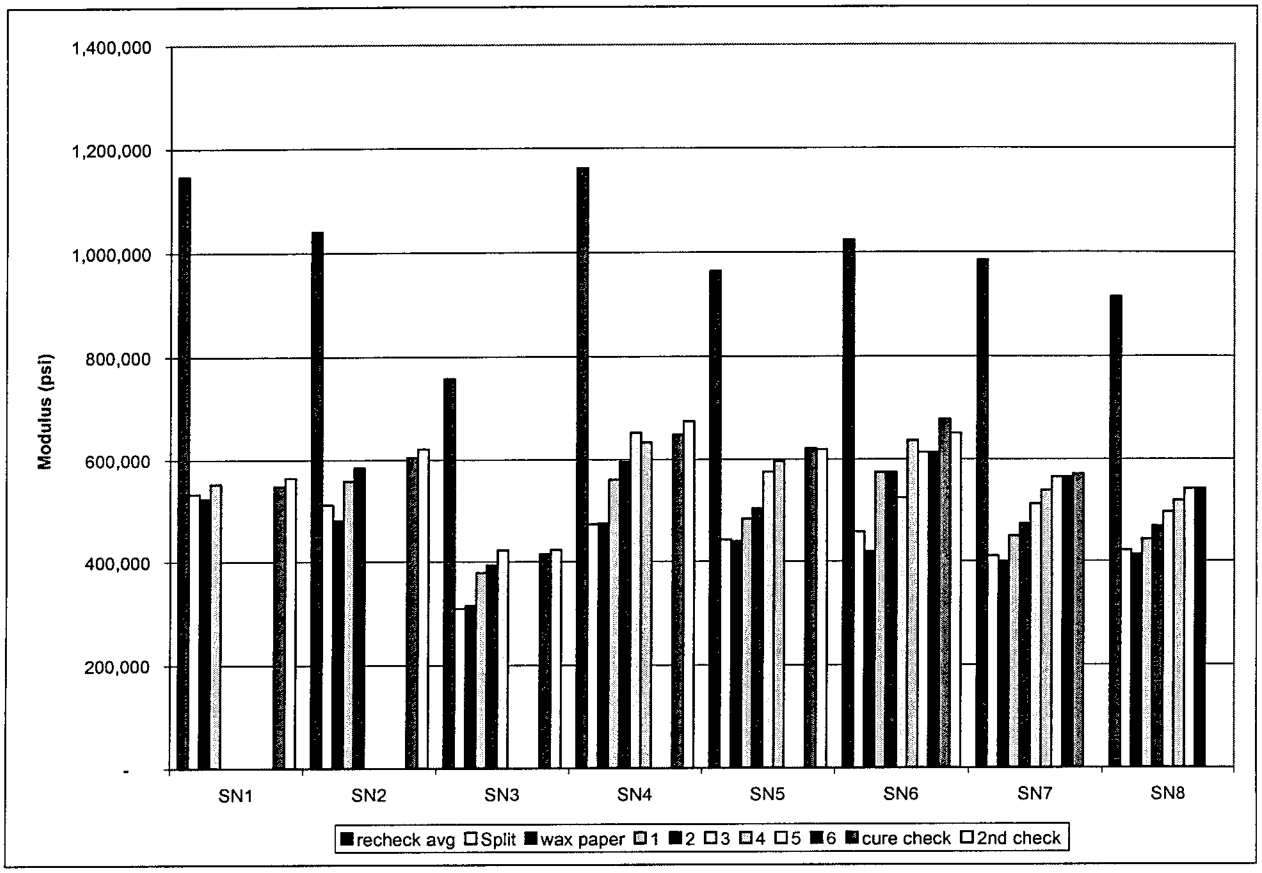


Figure 5.14 Effect of Amount of Shear Reinforcement on Modulus (Optimized 2x4 Specimens)

It is important to realize that the stiffening effect measured for this group of 2x4 specimens did not result in a level of "repaired" stiffness equal to that of the "undamaged" 2x4s. The amount of modulus loss from the undamaged case to the case immediately after imparting damage seems more severe than the group of specimens described in the previous section. At this time it is unclear why the degree of damage seems greater for this group of beams; however, the smaller resulting "repaired" modulus is believed to be related to the exclusion of epoxy bonding between the split sections. In the actual, proposed repair concept the adhesive used to bond in the shear spikes is planned to be injected and allowed to fill any cracks. Thus, the ultimate stiffening effect that should be able to be achieved is likely much closer to the first group of tests, in the previous section.

To evaluate the effect of additional shear spikes, two of the three beams with six pairs of shear spikes were further modified in a manner matching that described for the stacked 2x2 specimens in Section 5.1.5. The first of these beams (SN7) was modified through the addition of a third spike to each of the previous pairs, as shown in Figure 5.15, starting from the outside (M1) and working to the center (M6). Stiffness tests were performed after each addition. Once all six rows included three pultruded rods, a further five sets of three spikes were added, intermediate to the initial rows. Stiffness tests were performed after each intermediate row was added.

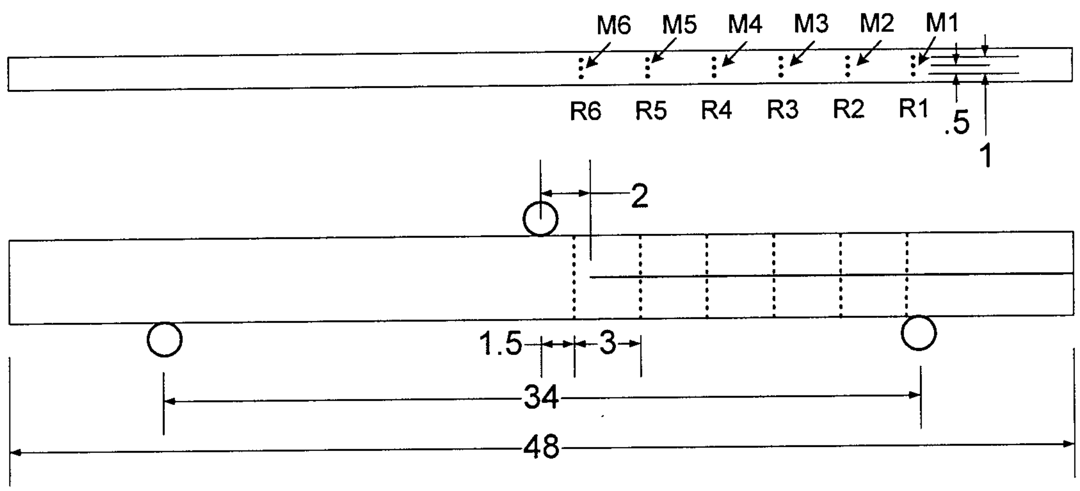


Figure 5.15 Geometry of Additional Reinforcement in 2x4 Specimens (Mid first, then Intermediate)

The results of stiffness testing of this modified beam, shown in Figure 5.16, indicate that the additional shear spikes do enhance the modulus of the specimen. The addition of the third pultruded rod in each row has only a limited effect, with the greatest gain (m5) being approximately 10 percent more than the final set of paired spikes. The additional intermediate rows of spikes showed a somewhat greater gain, with the final modulus showing an increase of approximately 25 percent more than the final paired shear spike case (6).

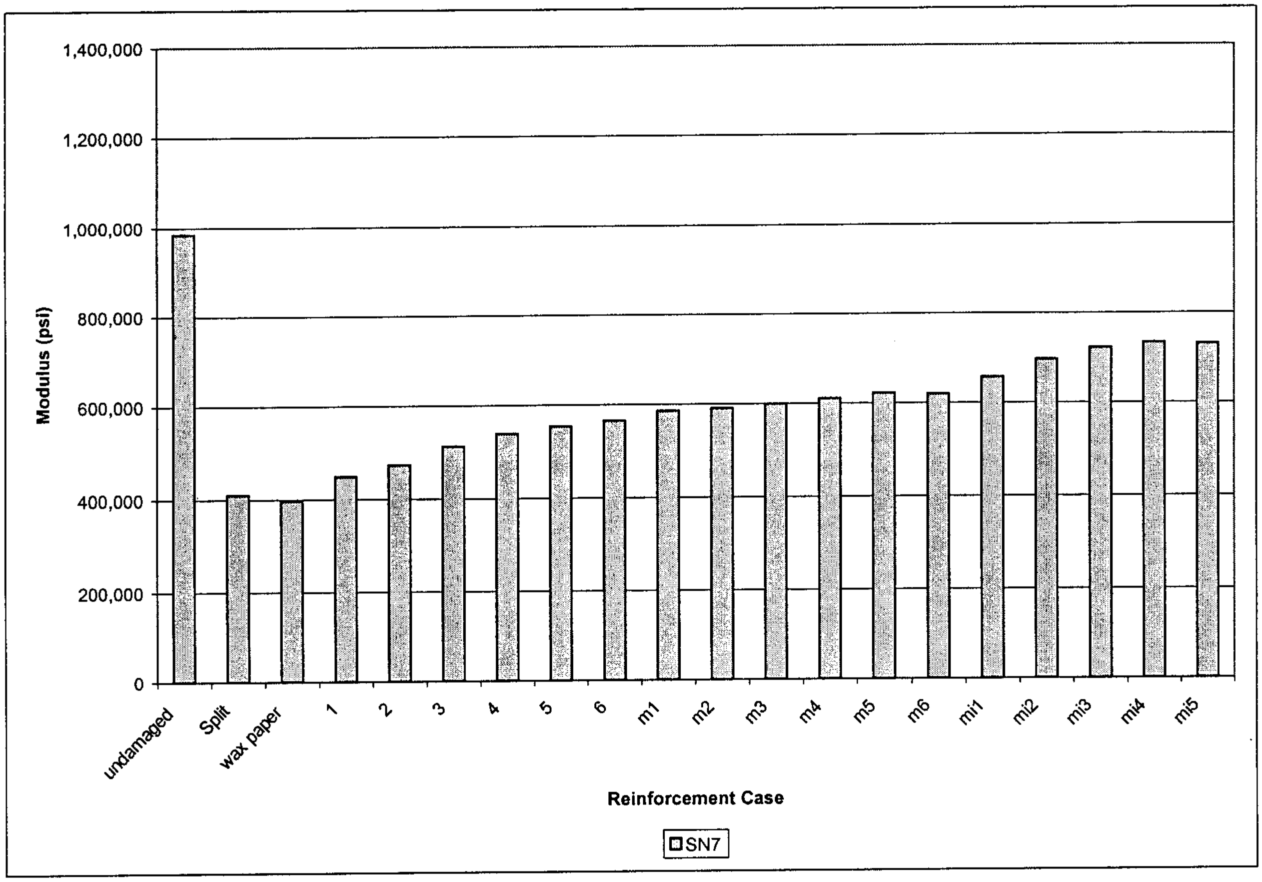


Figure 5.16 Stiffness Related to Additional Reinforcement in 2x4 Specimens (Mid, then Intermediate)

The second beam (SN8) to get additional reinforcement was modified through the insertion of intermediate rows of two spikes, as shown in Figure 5.17, starting from the outside (I1) and working to the center (I5). Stiffness tests were performed after each addition. Once all five intermediate rows were inserted, 11 more spikes were added in the middle of each pair of rods, resulting in 11 rows of three rods. Stiffness tests were performed after middle rod was added. The final level of reinforcement attained is the same as in the previous case. It should be noted that for each sequence of rod insertion a cure time in excess of 24 hours is required. This, in addition to the insertion time itself, means that each of these highly reinforced specimens requires roughly 30 days to prepare and test.

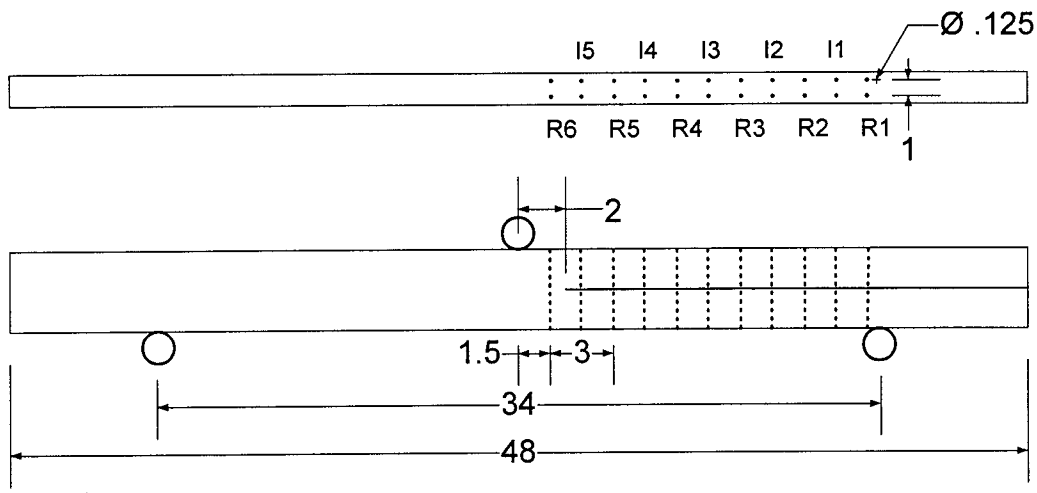


Figure 5.17 Geometry of Additional Reinforcement in 2x4 Specimens (Intermediate first, then Mid)

The results of stiffness testing of this modified beam (SN8), shown in Figure 5.18, indicate that the additional shear spikes do enhance modulus of the specimen. Again, the addition of the intermediate pairs of rods seems to have a somewhat greater overall effect. The additional intermediate rods (i5) yield an approximate gain in modulus of 20 percent more than the final paired shear spike case (6), while the added middle rods only generate an additional 10 percent increase. The final modulus increase is similar to the previous specimen (SN7), having an approximate gain of 30 percent over the final paired shear spike case (6).

The overall effect of these additional shear spikes is an appreciable gain in stiffness; however, neither specimen (SN7 or SN8) reaches the same level of modulus, upon completion of repair, as the undamaged 2x4 initially showed. The expected modulus gain for the actual repair case would be greater due to the additional epoxy rejuvenation, and should approach the level of improvement noted in the preliminary set of tests. In addition, it is possible that the density of shear spikes must be increased for optimal stiffening effect, plus, it may be necessary to optimize the distribution with respect to position along the beam. This could result in a non-uniform spacing of the reinforcing rods along a damaged region depending on the location in the beam.

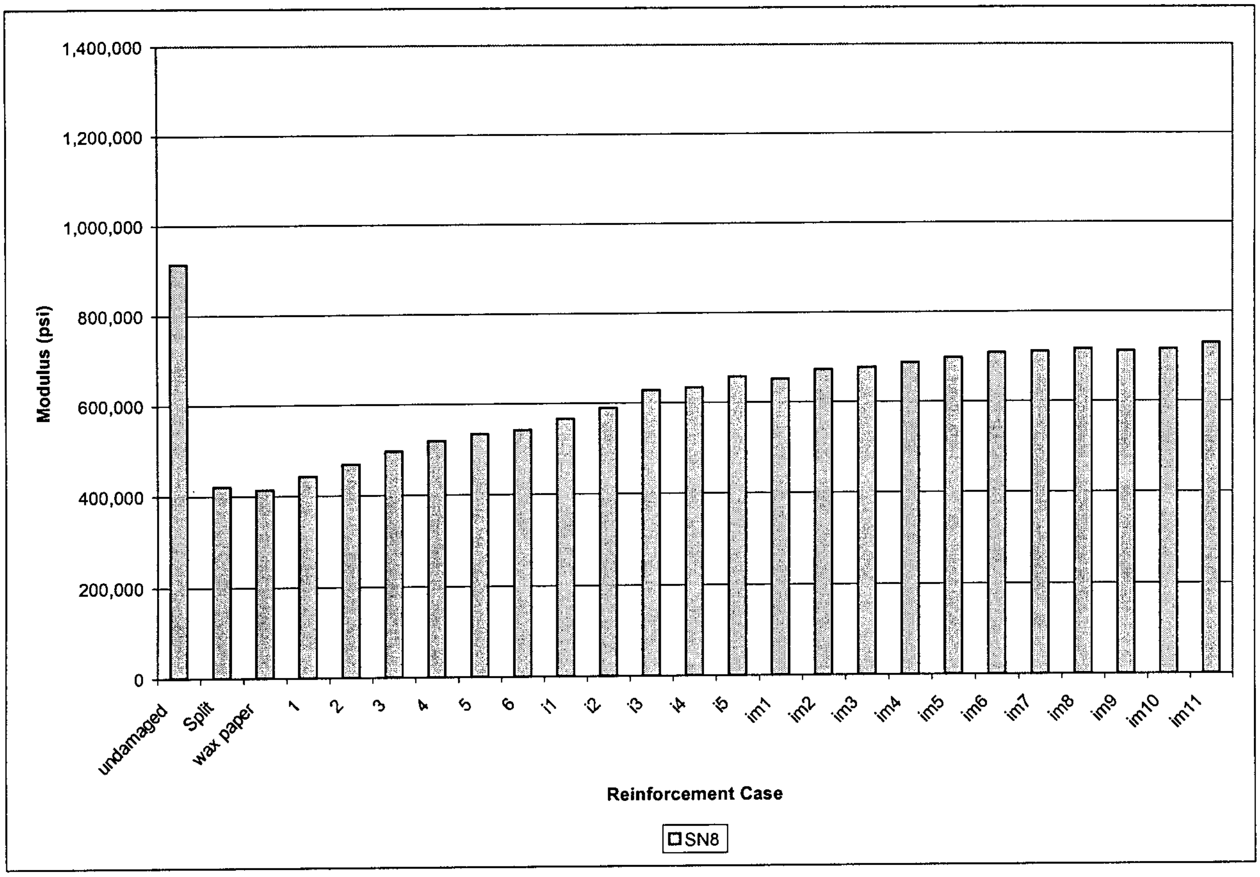


Figure 5.18 Stiffness Related to Additional Reinforcement in 2x4 Specimens (Intermediate, then Mid)

Another reason for the difficulty in generating full stiffness in the "repaired" 2x4 specimens seems related to the positioning of the shear spikes. In all test cases shear spikes only were added to the region of the split from the central loading point to the outer support point. However, the region beyond the outer support point is critical in the generation of stiffness through shear transfer improvement. Thus, future tests should evaluate the ability to further stiffen the beam through increased length of repair.

5.3 Failure Strength Evaluation

To measure the modulus, as described in the previous sections, the maximum beam loading purposely is maintained at a value substantially below that which would cause damage to the wood beam. In this way the elastic properties are evaluated and can be checked and measured numerous times. However, in addition to stiffness information, knowledge of the effect of shear reinforcement on the failure strength and form of failure is important. Thus, a large fraction of the beams produced for stiffness testing were scheduled for loading to failure. For the second group of 2x2s and 2x4s this failure testing was planned early and a specimen at each level of reinforcement was set aside so that failure strength could be evaluated in a sequential fashion. Specimens from the first set of preliminary tests also were loaded to failure. Test results are summarized in Table 5.1, which includes information on the strength of each beam, the modulus as measured during the failure test and the strain achieved.

Table 5.1 Overall Summary of Failure Performance of Flexure Tests

Category	Sample	Modulus	Strength	ASTM D790 Strain
2x4 Preliminary Unmodified	B5	1,482,434	7,373	0.57%
	B6	1,514,394	10,744	0.98%
	B7	1,293,282	9,217	0.97%
2x4 Preliminary Saw Split	19	775,744	3,127	0.56%
	23	933,568	7,518	1.40%
	24	1,117,147	8,156	1.56%
2x2 Preliminary Nails (only)	2	775,106	6,264	1.22%
	3	575,220	6,786	1.98%
	5	406,287	4,889	1.51%
2x2 Preliminary Fiberglass Rods	F6	953,508	4,825	1.13%
	F4	1,161,959	6,559	1.39%
	F1	397,511	4,918	1.12%
	F2	826,368	6,264	1.22%
2x2 Preliminary Fiberglass Shear Plates	P7	1,261,707	8,044	0.71%
	P8	1,550,309	9,239	0.65%
	P16	1,479,226	9,860	0.82%
2x2 Preliminary Epoxy-Bonded	9	1,177,028	6,449	0.59%
	10	1,036,716	7,544	0.74%
2x2 Optimized Tests with Fiberglass Rods	1	462,375	2,975	0.80%
	2	705,230	6,499	1.21%
	3	742,567	5,766	1.37%
	4	646,131	7,617	1.87%
	5	789,611	7,466	1.52%
	6	684,511	4,278	1.31%
2x4 Optimized Tests with Fiberglass Rods	1	555,896	5,152	1.36%
	2	607,045	5,272	1.47%
	3	414,598	4,236	1.60%
	4	632,419	7,356	1.72%
	5	537,966	4,276	1.04%
	6	557,103	4,112	1.03%

It is useful to note the wide variability in the values of modulus, strength and strain from specimen to specimen in a single group. As discussed earlier, it is for this reason that data, based on this limited number of specimens, is most appropriate for a determination of trends rather than for evaluating a specific numeric value. Further, batch-to-batch comparison by numeric value also is problematic, as the average value for the modulus of the six 2x4's in the second group of tests, prior to the addition of damage was 912,000psi.

5.3.1 General Strength Results

As noted, it is difficult to use this data for direct numerical comparison; however, Table 5.1 does show that the range of strengths is relatively narrow for most of the cases. Notably two of the three as-received 2x4 specimens showed strength values near 10,000psi, and the 2x2 specimens reinforced with the fiberglass shear plates also show strength values higher than typical. For the as-received 2x4s this difference is considered to be related to the variability of wood. For the 2x2 specimens reinforced with fiberglass shear plates, the shear plates add a significant amount of new material, which adds strength. For the remainder of the specimens tested for strength, once a small number of shear spikes were in place, the values seem to range from roughly 4,000psi to 7,500psi. In fact, visual examination of the specimens indicate that in this range of failure strengths the vast majority of the beams fail in either "Cross-Grain Tension" or "Simple Tension" as defined by ASTM D 143. Thus, for the majority of the beams the failures are not concentrated in the region near the repair.

It may be instructive to review the strain results as shown in Table 5.1. Since the strain is related to modulus and strength, it is interesting to note that in general the beams that showed the highest strengths also showed low strains to failure. This is especially true for the as-received 2x4s and the stacked 2x2s with bonded fiberglass shear plates.

In the preliminary set of beams it is useful to compare the performance of the epoxy-bonded 2x2s to the 2x2s with fiberglass shear spikes. For the best cases of each, the modulus and strength are roughly equal, with the epoxy-bonded beams showing slightly better performance. This seems to indicate enhanced shear transfer between the two beams; however, this should not be used as justification for an "epoxy only" rejuvenation approach. When the failure strains for these same specimens are compared, it is obvious, from the higher failure strains, that reinforcing rods are critical of the repair strategy.

5.3.2 Stacked 2x2 Flexural Testing – Optimized Specimen Preparation

During the second set of tests a 2x2 beam specimen of each reinforcement case, from one set of pultruded rods to six sets of rods, was retained for strength evaluation. Unfortunately this means that no repetitions of these cases are available. The resulting values of ultimate strength, modulus and failure strain for the six different reinforcement cases are shown in Figure 5.19. The ultimate strength values seem to reach a maximum at four sets of shear spikes and drops off at the maximum reinforcement. However, the failures which occurred, as seen in Figure 5.20, indicate failure away from the repair for all cases except that of the single set of shear spikes.

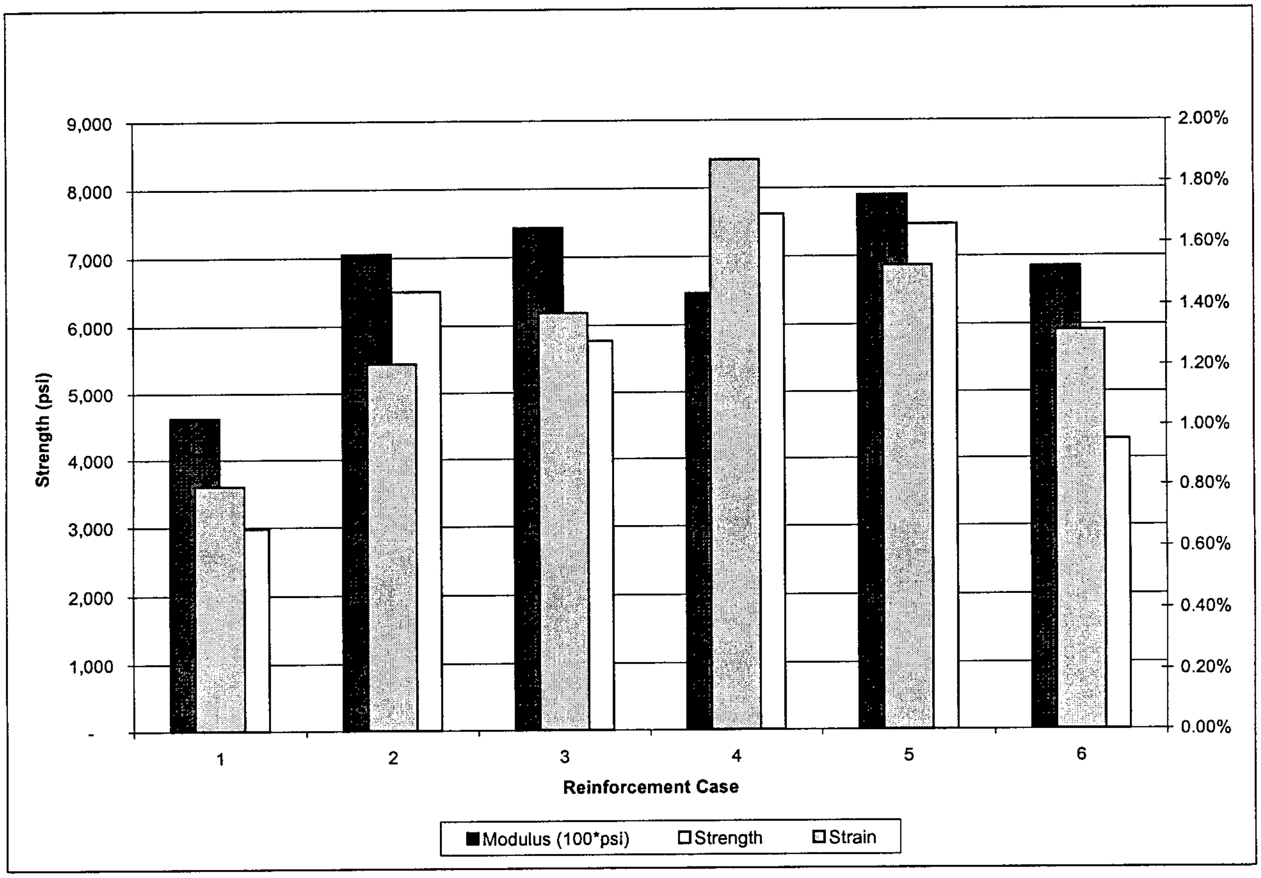


Figure 5.19 Summary of Modulus, Strength, and Strain with Reinforcement for Optimized 2x2s

Figure 5.19 also shows the effect of the number of reinforcing spikes on the failure strain. It is noted that the strain to failure begins to fall after reaching a maximum at four sets of shear spikes. Since the highest ultimate strengths were realized in conjunction with relatively low failure strains, the results shown in Figure 5.19 seem to indicate that a greater number of shear spikes are necessary to attain high strength and modulus, and to generate failures that most closely replicate the undamaged beams.

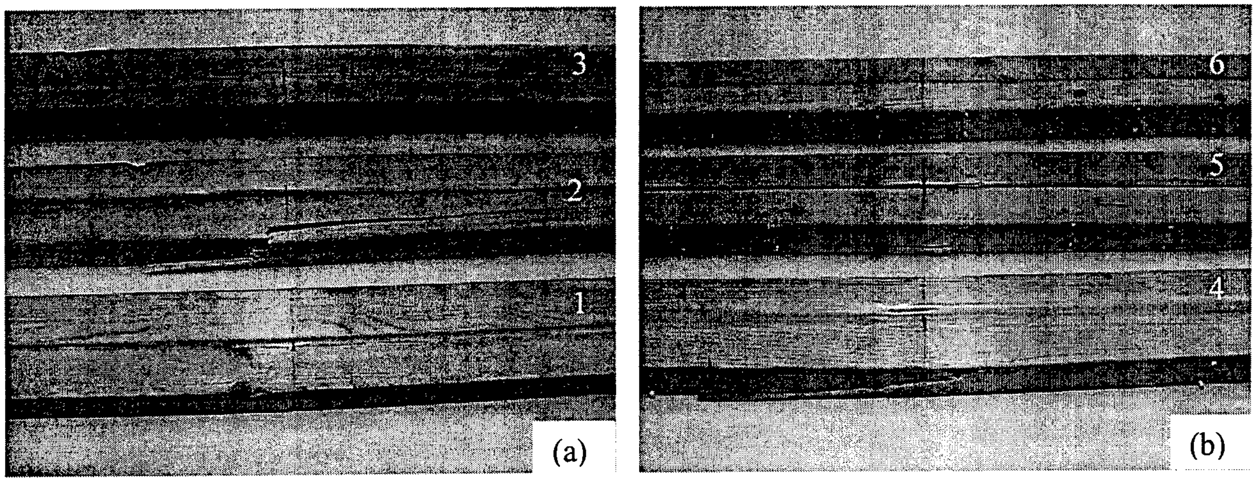


Figure 5.20 Failure Modes of Stacked 2x2 Optimized Beam Reinforcement Cases

Possibly more instructive are the stress-strain charts for the various reinforcement cases. Figure 5.21 is a compilation of the stress-strain charts for the six 2x2 reinforcement cases under discussion. The additional straight line is included to show the modulus of each specimen more clearly. These charts correspond to the six specimens shown in Figure 5.20. Upon review of these stress-strain charts it is immediately apparent that a significant failure event occurs at a stress well below the ultimate strength, where the curve changes from the initial linear slope. If we consider this "yield" strength, it seems that increasing numbers of reinforcing rods more directly corresponds to improved performance. Since the majority of the actual failures occur in the lower 2x2, as a tensile failure, the region of the stress-strain curve to the right of this "yield" strength is likely dominated by the performance of the wood, and not by the repair. Thus, for the stacked 2x2s tested is seems apparent that the pultruded fiberglass shear spike repair approach results in improvements in the modulus and the strength. The effect of increased numbers of reinforcing rods on the strength should be evaluated in any future research.

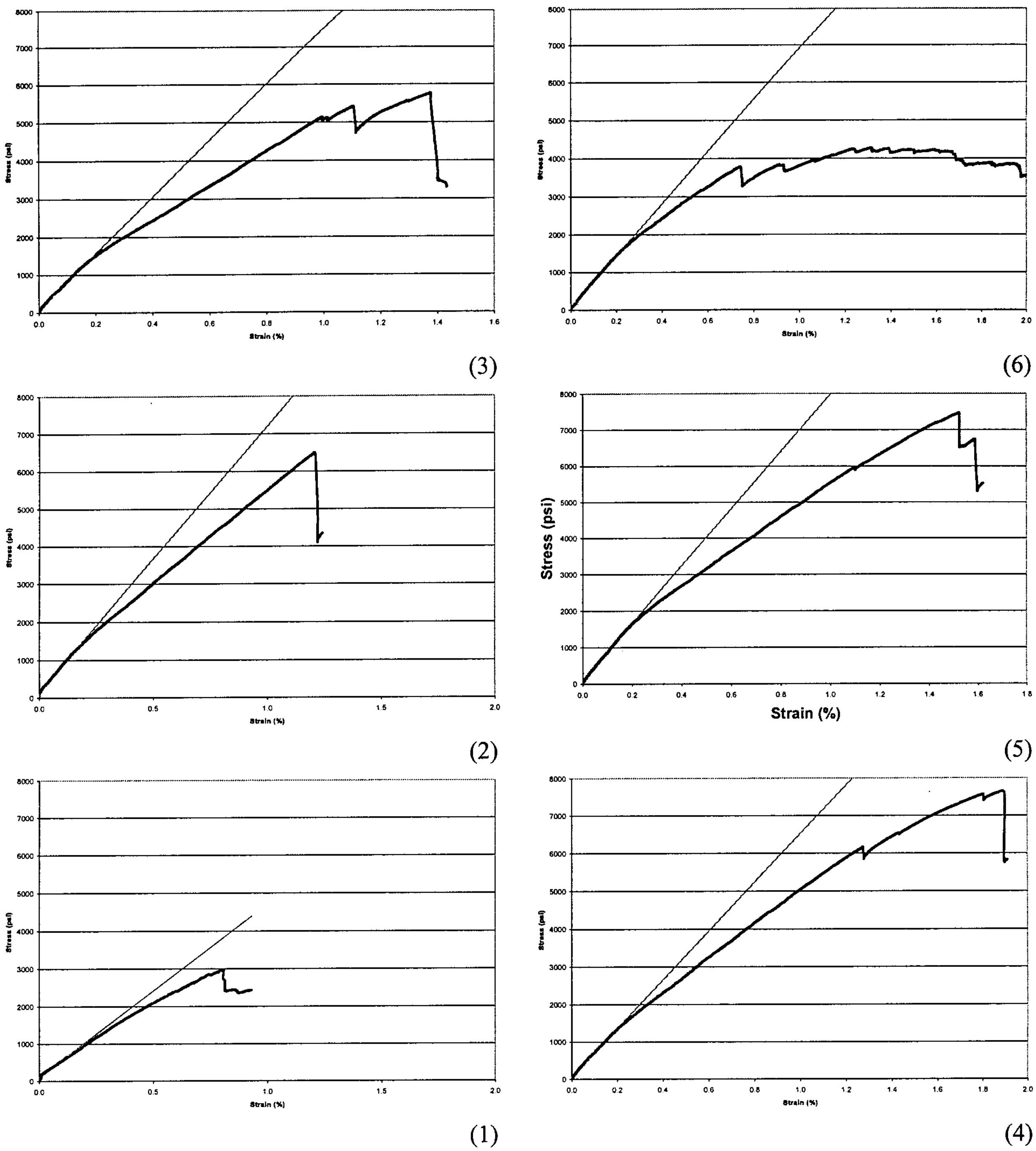


Figure 5.21 Stress-Strain Plots for Stacked 2x2 Optimized Beam Reinforcement Cases

5.3.3 Split 2x4 Flexural Testing – Optimized Specimen Preparation

As for the stacked 2x2s, during the second set of tests, a 2x4 beam specimen of each reinforcement case, from one set of pultruded rods to six sets of rods, was retained for strength evaluation. The resulting values of ultimate strength, modulus, and failure strain for the six different reinforcement cases are shown in Figure 5.22. The ultimate strength values seem to reach a maximum at four sets of shear spikes and drops off at the maximum reinforcement. However, the failures which occurred, as seen in Figure 5.23, indicate failure away from the repair for all cases except that of the single set of shear spikes.

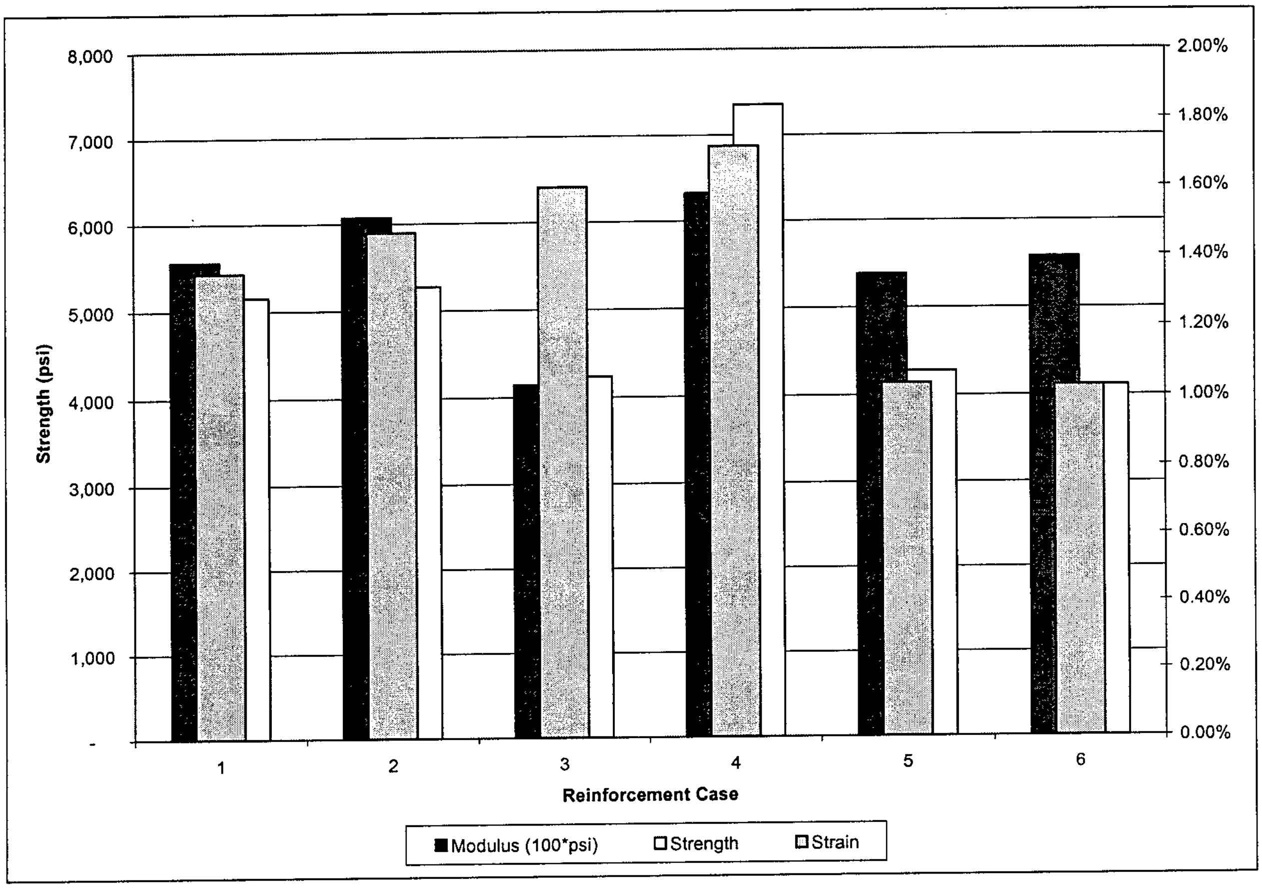


Figure 5.22 Summary of Modulus, Strength and Strain with Reinforcement for Optimized 2x4s

In general, the comments made regarding the stacked 2x2 specimens apply to the split 2x4s. However, differences are not as readily apparent as the undamaged side of the 2x4 specimens keeps the performance somewhat higher in all cases. Figure 5.23 shows that, as for the stacked 2x2s, all of the split 2x4 cases after the single reinforcement case show some form of tensile failure at the lower surface of the beam. The discussion relating to strains also is consistent with the discussion of the previous section. Thus, once again it would seem that the most interesting information will come directly from the stress-strain curves rather than from an evaluation of the ultimate strength and the failure strain.

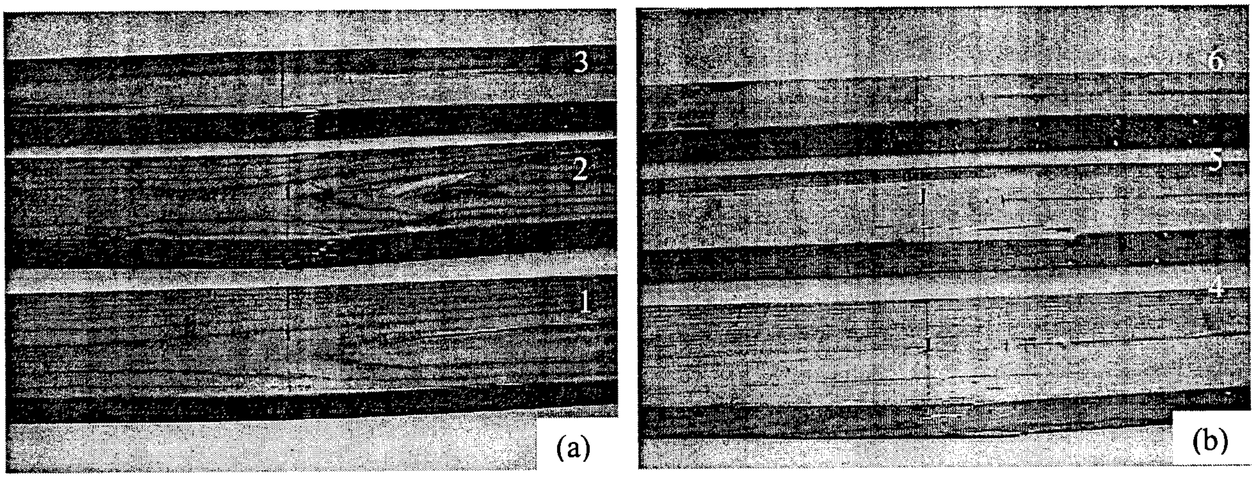


Figure 5.23 Failure Modes of Split 2x4 Optimized Beam Reinforcement Cases

Figure 5.24 shows the six stress-strain charts corresponding to the tested 2x4 beams shown in Figure 5.23. A knee once again exists in each stress-strain curve, at a value significantly below failure. Unfortunately the trend relating the knee of the curve to the amount of shear reinforcement is not as clearly apparent as it was for the stacked 2x2 specimens. It is suggested that this is due to the effect of the undamaged portion of the split 2x4 beams. Modulus changes with addition of reinforcement are obvious; however, while the "yield" strength increases consistently from the case of one set of reinforcing rods up through four sets of rods, the test specimens with five and six sets of shear spikes definitely show reduced performance. Referring back to Figure 5.14 and Figure 5.22 it is apparent that the moduli of these two specimens consistently were lower than the value measured for the case, which terminated with four sets of reinforcing rods. Thus, it seems that the limited effect of higher numbers of reinforcing rods may be being obscured by the variability of the wood specimens. This would again indicate that if numerical values are required for tasks such as design, testing of many more repetitions of each case should be necessary. As a final comparison, Figure 5.25 shows the stress-strain curves for three undamaged 2x4 specimens. Each of these curves shows a small knee at roughly 3,000psi, and then significant deviation from linearity by 7,000psi. Thus, the performance of the "repaired" split 2x4 beams seems to be progressing toward the undamaged case.

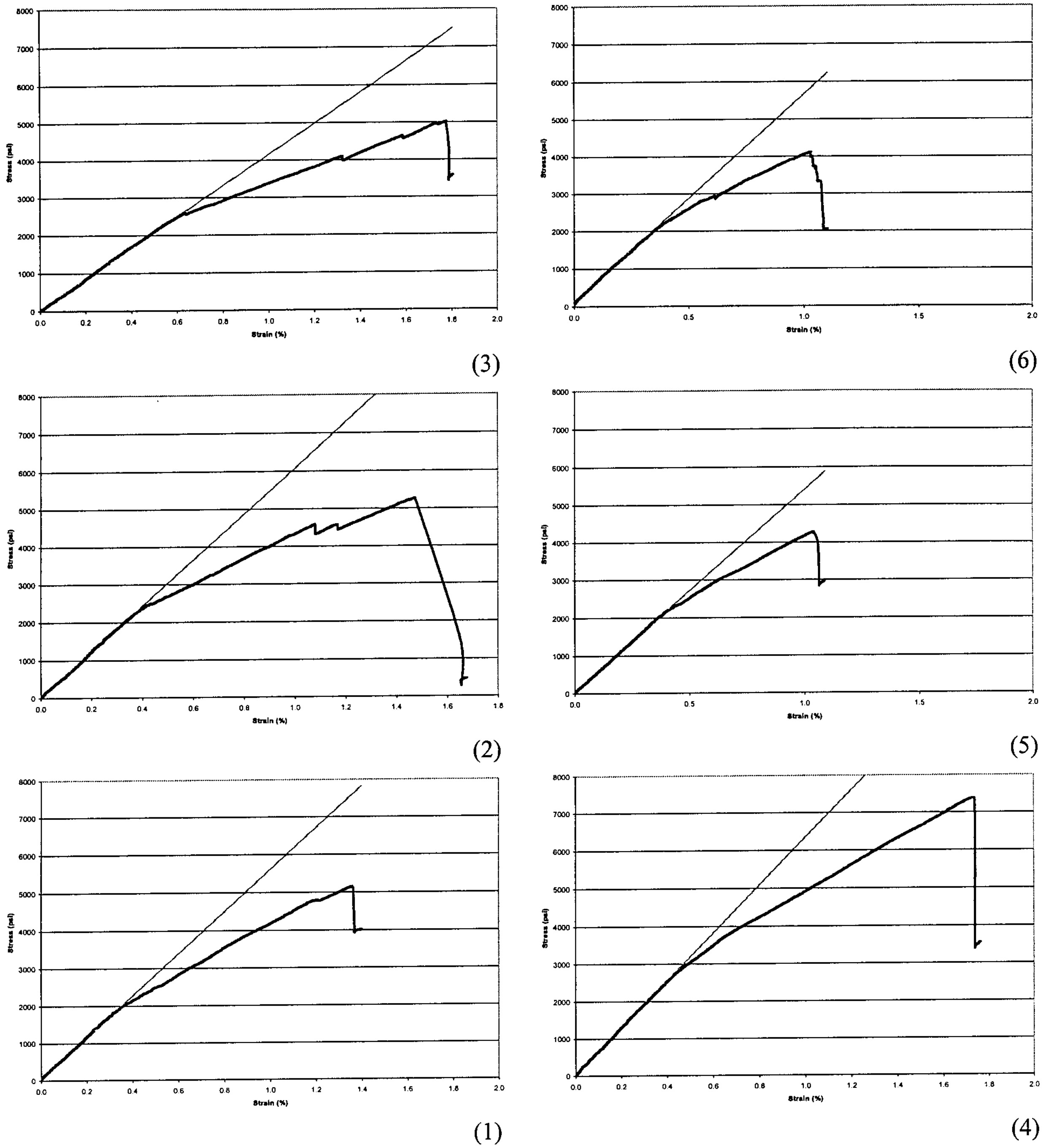


Figure 5.24 Stress-Strain Plots for Split 2x4 Optimized Beam Reinforcement Cases

Given the variability of the wood and the relatively high modulus measured for the undamaged specimens, they seem to match the case in Figure 5.24 of four sets of shear spikes most closely. This is quite a promising result, as the ultimate strength of this reinforced beam is as high as one of the undamaged beams of Figure 5.25.

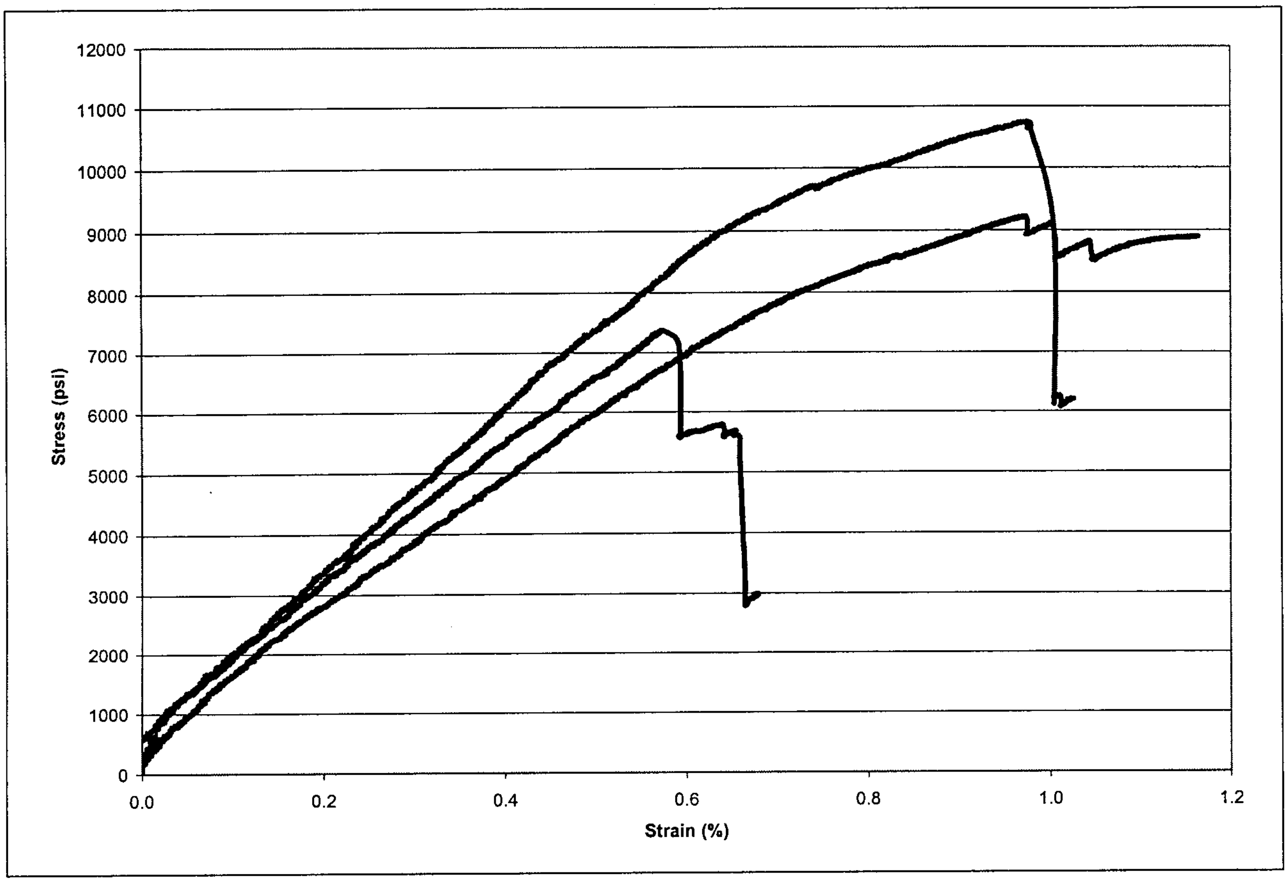


Figure 5.25 Representative Stress-Strain Curves for Undamaged 2x4 Specimens (B5, B6, B7)

6 CONCLUSIONS

Based on the extensive amount of preliminary data collected, it is clear that further investigation of the use of fiberglass pultruded rods as shear spikes in wood timber repair should be undertaken. The preliminary results indicate that the proposed approach recaptures stiffness and strength of the undamaged material. Enhanced repair performance is directly related to the number and position of the shear reinforcing rods. It also is obvious that the addition of the epoxy adhesive has a strong effect on the success of the proposed repair approach. Beams tested with shear spikes inserted without adhesive showed lower stiffness improvements than those tested with epoxy adhesive. An even greater improvement was noted when the epoxy was allowed to bond the top and bottom halves of the test beams together. This epoxy in the "damage" area significantly improved the stiffness and strength of the beams. However, comparing the twice as large failure strain values of the preliminary pultruded rod reinforced beams, where epoxy was allowed to flow to the interface, to the beams bonded only with epoxy, it is obvious that the combination of shear spikes and epoxy is much more effective than epoxy alone. Thus, this research has shown that the application of bonded-in, fiberglass pultruded rods as shear spikes in wooden beams can overcome severe damage, and that the approach should be further developed on a larger physical scale to determine the ultimate potential as an in-service repair approach for timber railroad infrastructure.

7 LITERATURE CITED

[1] U.S.Department of Transportation, FHWA, 1986, "Seventh Annual Report to the Congress on Highway Bridge Replacement and Rehabilitation Program."

[2] Peterson, M. L., J. Downs III and R.M. Gutkowski, 1996, "Non-Destructive Inspection of Timber Bridge Structures," in Proceedings 3rd Conference Non-Destructive Evaluation of Civil Structures and Materials, Sept 1996 (Atkinson Noland & Associated Boulder CO, 1996).

[3] Scott, W.R., "Assessment of a Timber Bridge-Aultnaslanach Viaduct," Proceedings of the Institute of Civil Engineering Transportation, Vol.111, 1995, p.132-134.

[4] Oomen, G. and R.A.P. Sweeney, "Application of Modern Technologies in Railway Bridge Infrastructure Management and Decision Making," Proceedings of Transportation Infrastructure – Environmental Challenges in Poland and Neighboring Countries, NATO ASI Series, Sub-Series 2, Vol.5, Springer Verlag 1996.

[5] Avent, R.R., "Design Criteria for Epoxy Repair of Timber Structures," Journal of Structural Engineering, Vol.112, No.2, Feb, 1986, p.222.

[6] Avent, R.R., "Decay, Weathering and Epoxy Repair of Timber," Journal of Structural Engineering, Vol.111, No.2, Feb, 1985, p.328.

[7] Triantafillou, T.C., "Shear Reinforcement of Wood Using FRP Materials," Journal of Materials in Civil Engineering, May 1997, p.65.

[8] Hallstrom, S. and J.L. Grenestedt, "Failure Analysis of Laminated Timber Beams Reinforced with Glass Fibre Composites," Wood Science and Technology, 31 (1997) p.17.

[9] Sonti, S.S. and H.V.S. GangaRao, "Strength and Stiffness Evaluations of Wood Laminates with Composite Wraps," 50th Annual Conference, Composites Institute, The Society of the Plastics Industry, Inc., Jan 30, 1995.

[10] Barbero, E., J. Davalos and U. Munipalle, "Bond Strength of FRP-Wood Interface," Journal of Reinforced Plastics and Composites, Vol.13, Sept, 1994, p.835.

[11] Steves, C.A. and N.A. Fleck, "In-Plance Properties of CFRP Laminates Containing Through-Thickness Reinforcing Rods (Z-Pins)," Proceeding ICCM-12, Paris, July 1999.

[12] Cox, B.N., "Mechanisms and Models for Delamination in the Presence of Through-Thickness Reinforcement," Proceeding ICCM-12, Paris, July 1999.