Ultimate Strength of FRP-Reinforced Glulam Beams Made with Douglas-Fir and Eastern Hemlock

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ABSTRACT

With recent changes in availability of forest resources, high quality tension laminations necessary for glulam construction have become more expensive and increasingly difficult to procure. Fiber reinforced plastics (FRP) offer good promise to serve both as a substitute for the high quality wood laminations and as reinforcement for glulam beams. Glulam, like reinforced concrete, can be reinforced in tension to more efficiently utilize the wood's compressive strength.

In this paper, the ultimate strength behavior of FRP-reinforced glulam beams made with Douglas Fir and Western Hemlock is quantified experimentally. Like many other wood species, lower grades of Western Hemlock and Douglas Fir have higher compression strength than tensile strength. It is primarily this differential in tension and compression values that justifies the use of tension reinforcement. Sixty, 21 ft span glulam beams, reinforced with fiber-reinforced plastics (FRP) on the tension side and thirty unreinforced controls were instrumented and tested to failure in four-point bending. FRP reinforcement ratios were 1 and 3%.

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TESTING PROGRAM

Research on reinforced wood technology has been on-going at the University of Maine and the Composite Materials Technology Center (COMTEC) in Minnesota since 1989. The objective of this paper is to briefly summarize four-point bending test results of 21 ft span CR-glulam beams made of douglas-fir and western hemlock.

The beam testing program followed eight months of extensive development of wood-to-FRP bond strength and durability. A total of 102 beams were manufactured, 90 of which were used for flexural strength testing and the remaining 12 were used for flexural creep testing. The beams were all 22 feet long, with a cross-section of 5-1/8 inches x 12 inches (See Figure 1). The 102 beams were distributed as follows:

Western hemlock:

- 0% reinforcement ratio: 15 beams for bending strength + 2 beams for creep
- 1% reinforcement ratio: 15 beams for bending strength + 2 beams for creep
- 3% reinforcement ratio: 15 beams for bending strength + 2 beams for creep

Douglas fir

- 0% reinforcement ratio: 15 beams for bending strength + 2 beams for creep
- 1% reinforcement ratio: 15 beams for bending strength + 2 beams for creep
- 3% reinforcement ratio: 15 beams for bending strength + 2 beams for creep

The reinforcement ratio is the total cross-sectional area of the FRP reinforcement divided by the total cross-sectional area of the beam.. For consistency, all beams (including the reinforced beams) have the same cross-sectional area. This was accomplished by reducing the thickness of the wood 'bumper strip' (lowest lamination of a beam, below the FRP layer) by the total thickness of the FRP layer.

The reinforcement consisted of one or more 1/8" thick GFRP pultruded sheets placed above the lowest wood lamination (referred to as the "bumper strip"). Thus a single 1/8 inch thick GFRP produced a beam with 1% reinforcement. Three GFRP sheets produced a beam with 3% reinforcement.

Special "low grade" douglas-fir and "low-grade" western hemlock laminations were developed for this study, consistent with resource-optimization needs. Tension and compression strength and stiffness properties of these laminations were obtained by laboratory testing of a representative sample according to ASTM D198. The tension strength test used a gage length of 7 feet, and sixty samples each of douglas fir and western hemlock.. The compression test used 12 inch long 2x6 members, sixty each for douglas fir and western hemlock.. The tension and compression specimens were also matched, that is they were obtained from the same board of lumber.

Tension strength and stiffness properties of the GFRP sheets were obtained using five specimens from the production line.

Fourteen beams of each type were tested in four-point bending at APA in December 1996. The APA testing measured loads and deflections through failure. Three beams of each type were tested at UM; one of each type was instrumented with strain gages and tested in four-point bending. The remaining two beams of each type were used in a creep study. The results of the creep study are provided in a separate paper in these proceedings.

TEST RESULTS

Flexural test results indicated that the addition of reinforcement substantially increases the MOR. Reinforcement also increases the MOE, though to a much lesser degree. Table 1 summarizes the performance gains.

TABLE 1 - SUMMARY OF PERFORMANCE GAINS OVER CONTROLS
(each figure is average of 14 test specimens)

Structural Property	Hem-fir with 1%	Hem-fir with 3%	Doug-fir with 1%	Doug-fir with 3%
Modulus of Elasticity (MOE)	7%	14%	8%	16%
Modulus of Rupture (MOR) of gross section at bumper lamination failure	13%	25%	21%	29%
Modulus of Rupture (MOR) of reduced section at ultimate failure	42%	60%	56%	63%

The allowable stress for each group of beams was calculated according to ASTM D3737. A table summarizing the allowable levels and the Modulus of Elasticity for each beam type is shown below.

TABLE 2 - SUMMARY OF ALLOWABLE STRESSES AND MOE

Test Specimen	Average MOE (10^6 psi)	Allowable F_b, based on gross section at bumper lam failure (psi)	Allowable F_b, based on reduced section at ultimate load (psi)
Hem-fir Control	1.31	1398	N/A
1% Hem-fir	1.40	1676	1916
3% Hem-fir	1.50	2067	2848
Doug-fir Control	1.49	1557	N/A
1% Doug-fir	1.61	1413	2288
3% Doug-fir	1.74	2059	3452

Clearly, the use of reinforcement adds value to low grade lumber. The tension reinforcement of glulam with composite sheets significantly increases the bending strength but increases the bending stiffness to a lesser degree. Continued refinement of the material, fabrication and design of reinforced glulam beams is warranted.

ACKNOWLEDGEMENTS

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CENTER FOR ADVANCED ENGINEERED WOOD COMPOSITES IN CONSTRUCTION: *RESEARCH AND DEMONSTRATION PROJECTS*

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ABSTRACT

In the mid-19th Century, reinforcing concrete with steel significantly changed building and bridge construction throughout the world. As the 20th Century ends, many of the factors that contributed to the success of reinforced concrete are found in combining wood with Fiber Reinforced Polymers (FRP). The NSF-funded Advanced Engineered Wood Composites Center at the University of Maine is developing the underlying science and engineering principles that will allow FRP-Wood hybrid materials to change the face of the wood construction industry. This paper briefly outlines the goals and objectives of the Center, the various research projects now on-going within the Center, and features two FRP-glulam demonstration projects constructed in the US.

INTRODUCTION

Recent research has shown that Advanced Engineered Wood Composites (AEWC), i.e. hybrid wood-FRP structural elements, offer superior properties at reduced costs. University of Maine and other studies demonstrated, for example, that 2% FRP reinforcement can increase the strength of laminated wood beams by over 70%.

As in the development of reinforced and prestressed concrete, basic research is being conducted to unlock the full potential of a wide variety of AEWC structural members, e.g., beams, columns, panels, and connections. The objectives of the AEWC Center are to develop FRP materials that are compatible with wood, develop an interface/interphase system between the two materials that will ensure full composite action, and to develop the basic understanding of the short-term and long-term behavior of AEWC structural elements including performance over the full range of loading, ultimate strength, ductility, creep, fatigue, and moisture/temperature/UV cycling.

The Center consists of a multi-disciplinary team of engineers and scientists from two US universities supported by national industry associations in the wood and Composites areas, individual companies, and a national laboratory. The Center will be housed in a new research laboratory currently under construction at the University of Maine. The $5 million laboratory/pilot plant is designed to speed the development and application of AEWC technology.

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objective is to develop the science and processing parameters which will allow the full benefits of AEWC materials to be realized.

From a research and development viewpoint, AEWC elements present unique challenges because two different classes of materials, FRP and wood, are used together. Consequently the principles governing the short- and long-term structural behavior differ in many ways from those involving each of the materials by itself. Wood exhibits large changes, relative to FRPs, in strength, stiffness, dimensions, and creep properties with changes in ambient relative humidity. Accordingly, the FRP in the hybrid system will experience long-term stresses and strains that would not be present in the FRP alone. The short-term behavior of AEWC hybrids is also different from that of either the FRP or wood. While the bending failure of a wood beam is typically brittle, the corresponding failure of a wood beam properly reinforced on the tension side with FRP is ductile.

Although there has been recent interest in the reinforcement of wood with FRP systems, there has not been a systematic investigation of the controlling mechanisms and interactions between materials necessary to develop an optimized, durable hybrid composite. The goals of the research team at the AEWC Center are to:

- Develop a new class of FRP reinforcing materials that are compatible with the hygro-expansion and visco-elastic characteristics of wood.
- Develop and maintain over time the interface between the two materials needed to ensure full composite action.
- Develop a basic understanding of the short- and long-term behavior of AEWC structural elements including performance over the full range of loading, ultimate strength, ductility, creep, fatigue, and moisture/temperature/UV cycling.
- Establish microstructure property-performance relationships.
- Develop durability and long-term performance models.
- Develop models of structural element behavior under service and ultimate loads to aid structural engineers in the analysis of AEWC hybrids.
- Optimize structural element shapes and material composition to maximize the efficiency of the AEWC hybrid systems.
- Develop Reliability Based Design (RBD) criteria for the AEWC hybrid systems.

ON-GOING RESEARCH

A number of research projects are currently being conducted at the Center. These projects range from laboratory work on the microstructural level of fibers, matrix and the hybrid interphase to the component level of beams, and to the structural level of building and bridge systems. Research on the micro level consists of identification, selection and characterization of FRP constituent materials to continue the development of reinforcement systems that are compatible with various wood species. The characterization program consists of determination of physical, mechanical, environmental and durability properties of FRP systems. Other research efforts are focused on the effect of surface conditions on the bond quality and durability of the bond between wood and

FRP reinforcement using micro-tomography for interface/interphase analysis, hygrothermal stresses and mechano sorptive creep.

Research on the component level consists of investigation of damage-performance relationships, nondestructive evaluation, and fracture toughness testing of FRP reinforced glulam beams. Research on the structural level includes creep properties of AEWC systems; construction, testing and monitoring of piers and bridges; and development of nonlinear probabilistic models for the design of FRP reinforced wood structures.

In addition to these research projects, the AEWC Center is actively working with existing industries to develop new products, test existing products, and provide potentials for new and expanded markets. Member companies from the composites industry, the engineered wood industry and the construction industry are engaged in a wide variety of research and development projects ranging from new FRP-wood product development to design and construction of bridges using the AEWC technology.

EXAMPLE DEMONSTRATION PROJECTS

Two of the fourteen demonstration projects constructed, tested and monitored by the Center are described below.

The Bar Harbor Pier. The Bar Harbor (ME) Yacht Club Pier, the first of its kind, is a 124 foot long, 5 foot wide FRP-reinforced glulam pier which replaces a structurally deficient 40-year-old steel girder pier. The reinforced glulam girder pier allows pedestrian travel from the shorefront to an offshore platform used for docking sailing vessels.

The 40-year-old steel girder pier had a significant corrosion problem and presented a safety hazard to club members. It consisted of 3 spans, one span was 26'6" and the other two spans were approximately 49 feet each. The pier's 5 foot wide wooden deck was supported by two steel girders along the entire length of the pier. On the shore side the steel girders rested on a concrete abutment while over the ocean the girders rested on granite block piers topped with concrete caps. The bottom of the steel girders rested 6'5" above the high water line. On the ocean end of the pier was a gangway which allowed passage to the offshore platform.

The FRP-reinforced glulam pier was modeled after the steel girder pier. The existing concrete abutment and concrete-capped granite block piers were re-used. Consequently, the new pier has the same deck width and approximately the same spans as the original pier. The new pier offers a chance to demonstrate the use of FRP technology, to identify real-life fabrication problems for FRP glulams, and the opportunity to evaluate long-term durability of FRP glulam in an aggressive environment.

The pier is designed for a live load of 85 psf. The structure is designed for a dead load of 39 psf. The lateral bracing system, consisting of a combination of lumber and galvanized steel angle sections, is designed for a 90 mph wind speed (approximately 32 psf using UBC standards). The post and rail system along the walkway was designed to BOCA 1990 standards. The girder seats were designed for an uplift force on the structure of 25 psf and buoyant forces considering the girders fully submerged in water. The FRP-glulam girders and other structural wood components of the pier were designed for wet service conditions. Construction of the FRP glulam girders and the bracing system is shown in Figure 1, and the completed pier is shown in Figure 2.

The beams for the Bar Harbor pier were fabricated by Unadilla Laminated Products in Sidney, NY. Constructed in the summer of 1995 by Harbor Place, the pier will be monitored through 1999. It will be load tested and inspected every year to see if there is any reduction in properties over time. The FRP-wood bond will be examined periodically using acoustic methods to monitor the quality of the bond between the FRP and the wood.

The MEDWAY Bridge: The Medway bridge is a 54 ft long two lane vehicular bridge constructed in Medway, Maine. The bridge consists of 15 FRP reinforced glulam beams. The beams were designed to meet the Maine DOT specifications of:

$F_b = 2{,}400$ psi   MOE = $1.8\text{x}10^6$ psi

Beam Depth = 51 inches;   Beam width 5-1/8 inches;   Beam Length = 54 ft.

Each beam consists of a total of 33 red maple wood laminations which are 1-1/2 inch thick, 1 percent FRP reinforcement on the tension side, and one wood lamination acting as a "bumper strip" below the FRP. The total depth of the beams is 51-1/4 inch. The total thickness of the FRP reinforcement is ½ inch and the total length is 54 ft. The properties of the FRP laminas in the longitudinal direction consist of a minimum modulus of elasticity of 6 million psi, and a minimum strength of 104,000 psi. The lay-up of the beams is shown in Figure 3 and a close-up of a beam with the FRP reinforcement is shown in Figure 4.

The FRP reinforcement was produced by Strongwell (formerly Aligned Fiber Composites) in Chatfield, Minnesota. The FRP reinforced glulam beams were fabricated by Unadilla Laminated Products in Sidney, NY under the supervision of a research team from the University of Maine. Construction of the Medway Bridge was completed in October 1997.

SUMMARY AND CONCLUSIONS

Advanced engineered wood composite hybrids represent a new class of construction materials that offer superior properties at reduced cost. These materials feature low-cost wood combined with fiber reinforced plastics to produce a variety of structural members that can readily be applied in today's buildings and bridge construction. A number of studies and demonstration projects conducted by many researchers show the potential of these new materials. The Center for Advanced Engineered Wood Composites at the University of Maine is established to develop the underlying science and engineering principles that will allow the full benefits of these materials and to facilitate their applications in the construction industry.

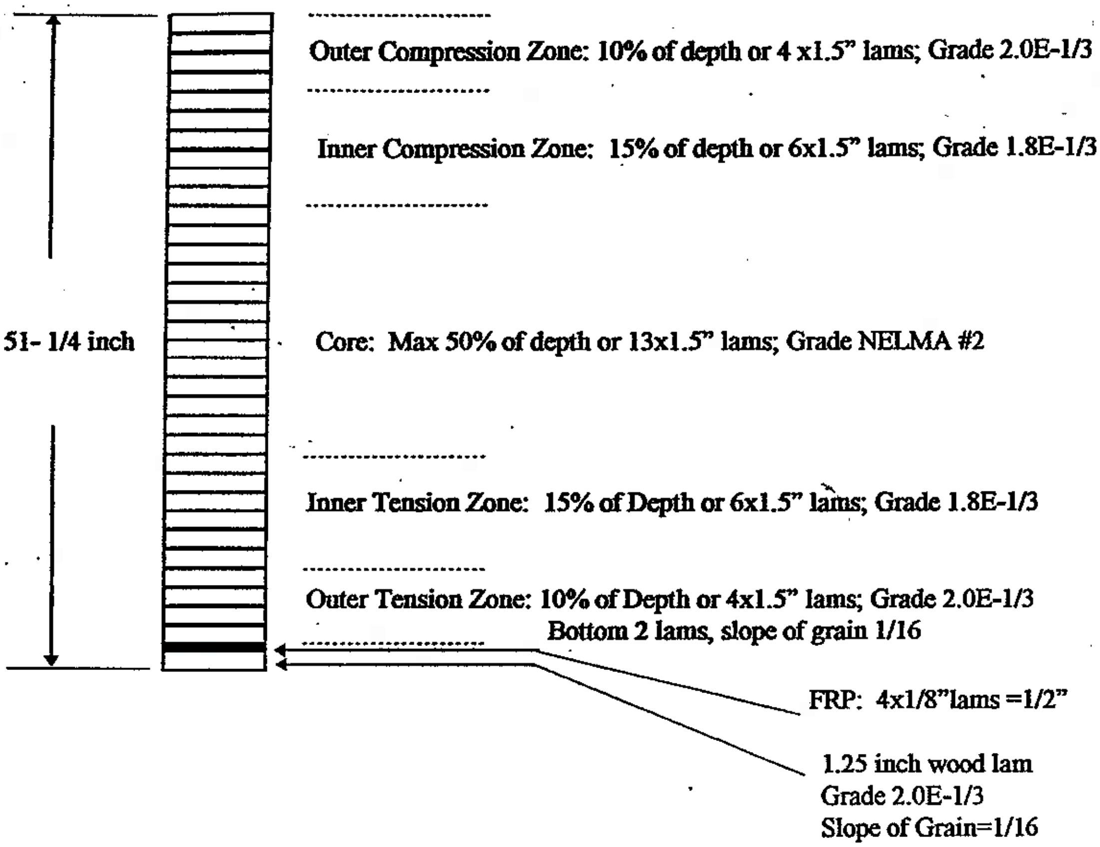

Figure 3. Red Maple and FRP Lay-up of the Medway Bridge Glulam Beams

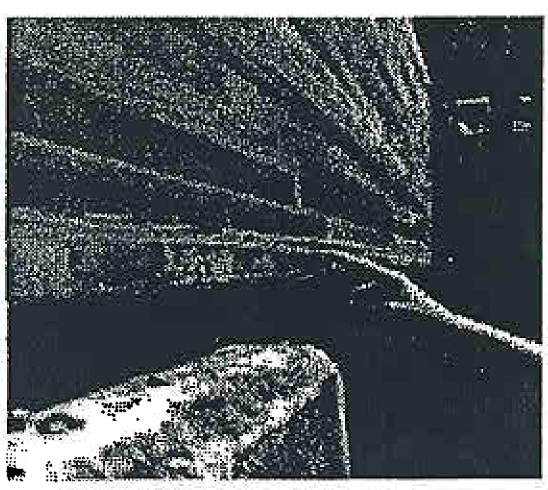

Figure 4. Close-up of the FRP Reinforced Glulam Beam.

Creep Behavior of FRP-Reinforced Glulam Beams

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ABSTRACT: Twelve 22-ft. long glulam beams are tested for creep in 4-pt bending in a controlled environment and a stress of 25% above the allowable design stress. This is an ongoing test that was started in May, 1997, and is scheduled to continue for one year. The beams are Douglas-fir and Western hemlock and are reinforced with 0%, 1%, or 3% E-glass FRP on the tension side. A wood bumper strip was added to the tension face to protect the FRP layers. Mid-span deflections and wood moisture content are reported for the first five months of testing. Reinforced beams can support up to twice as much load as unreinforced beams. However, the preliminary results show that despite the increased loading for reinforced beams, there is no increase in relative creep.

Introduction

Glued-laminated beams (glulam), like reinforced concrete, can be reinforced in tension to more efficiently utilize the wood's compressive strength. With recent changes in availability of forest resources, high quality tension laminations necessary for glulam construction have become more expensive and increasingly difficult to procure. Fiber reinforced plastics (FRP) offer good promise to serve both as a substitute for the high quality wood laminations and as reinforcement for glulam beams.

While there has been significant interest in FRP-glulam technology, there is no published information on creep behavior of FRP-glulam beams. What is of most concern from a long-term performance standpoint is that the sustained wood stresses in FRP-glulam beams are substantially higher than they are in unreinforced beams. Creep is an important consideration in the design of wood structures and is even a more important consideration when the sustained stress levels are higher than has been typically used in the past. The objective of this paper is to compare experimentally the creep behavior of glass-FRP reinforced and unreinforced glulam beams under relatively constant temperature and humidity conditions. Two different wood species, Western hemlock and Douglas-fir, and three different reinforcement ratios, 0%, 1%, and 3%, are used in this study.

Background

Durability of a new material is a key issue that must be addressed before the material can be utilized. Creep and creep rupture are important aspects of durability, since high sustained loads could cause higher time-dependent deflections than are allowed and possibly creep-rupture failures. Previous studies on FRP-reinforced glulam have shown that FRP considerably increases the strength, stiffness, and ductility characteristics of wood members, especially those made from low-grade timber (Dagher et. al. 1996, Abdel-Magid et. al. 1994, Plevris and Triantafillou 1995).

However, the only known published work on creep in FRP-reinforced wood members is a study in 1995 by Plevris and Triantafillou. They tested 5.5-ft. long solid-wood beams with carbon FRP bonded to the tension face. They also conducted several parametric studies to determine the effect of different types and amounts of reinforcement, and changes in temperature and humidity. They drew two important conclusions from their results. First, increasing the FRP area fraction decreases the amount of immediate and creep deflections for the same applied load. Second, creep behavior of FRP-reinforced beams is controlled by the creep in the wood.

Since wood behavior controls creep deflection in reinforced beams, it is relevant to look at previous research on creep in wood alone. The effects of stress level, temperature, and humidity on creep in wood have all been studied in great detail. Researchers have shown that as any one of these parameters increases, so does creep (Dinwoodie et. al. 1992, Fridley et. al. 1992, Shen and Gupta 1997, Hoyle et. al. 1995).

An interesting effect of moisture changes on creep in wood has been discussed by Ranta-Maunus (1975) and supported by Hunt (1989) and Martensson (1994), and has been termed the mechano-sorptive effect. The first change in moisture content of a loaded member results in increasing creep strain for both wetting and drying. After several cycles, drying increases deflection and wetting decreases deflection, but the net effect is an increase in creep (Ranta-Maunus 1975). This mechano-sorptive creep effect is much greater than normal time-dependent creep (Hunt 1989). Both Hunt (1989) and Martensson (1994) were able to show by modeling that it is possible to consider mechano-sorptive creep as a separate effect, and it can be included or subtracted directly from the total creep.

Experimental Procedure

The experiment was performed in a structural testing facility at the University of Maine. The 1800 ft^2 facility contains a concrete strongfloor and a steel reaction frame used for testing full-size bridges. The test area was insulated to maximize environmental control. Three heaters, four humidifiers, and three dehumidifiers were added to keep the conditions in the range of 65°F to 75°F and 50% to 70% relative humidity. Temperature, relative humidity, and moisture content of the beams were recorded 2 to 4 times daily.

There are six sets of test specimens with 2 beams in each set. The control beams contain no reinforcement while the reinforced beams contain either 1 layer or 3 layers of 1/8 in thick E-glass FRP. These amounts correspond to 0%, 1%, and 3%, respectively, of the beam cross-sectional area. The beam designations are given in Table 1. All of the beams have 8 wood laminae. On the reinforced beams, the 8th lamina is termed a "bumper strip" and bonded to the outer face of the FRP, providing protection for the reinforcement.

Table 1: Beam Designations in Test Categories

FRP Reinforcement Ratio	Western Hemlock	Douglas Fir
0%	WH-0%-1 WH-0%-2	DF-0%-1 DF-0%-2
1%	WH-1%-1 WH-1%-2	DF-1%-1 DF-1%-2
3%	WH-3%-1 WH-3%-2	DF-3%-1 DF-3%-2

The beams are loaded in 4-point bending at the one-third points using the setup depicted in Figure 1. The beams are 5 1/8 in. (130mm) by 12 in. (305mm) in cross-section and 22 ft. long

(6.7m), with an effective span of 21 ft. (6.4m). The loads consist of two concrete blocks per beam. The beams are simply supported on steel pins that allow rotation but not translation. The pin supports are positioned on top of 4-ft. high concrete block supports. Dial gages with 0.001-in. increments are mounted at the mid-span and at the supports. Each reinforced beam is laterally braced near the supports.

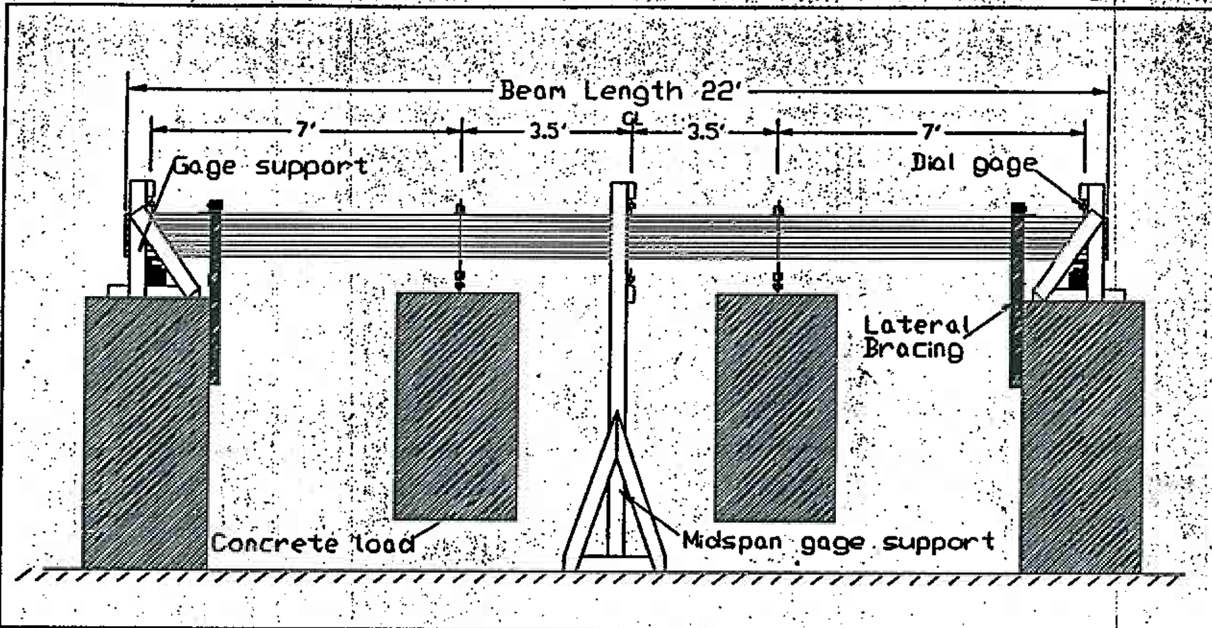

Figure 1: Test setup for accelerated creep study.

The beams are loaded at 25% above their allowable design stress. The allowable design stress values were determined from data of previous ultimate strength tests on 90 beams (15 in each of the 6 categories). The allowable design stress was calculated by taking the 5% Lower Tolerance Limit of the ultimate strength and dividing by 2.1, in accordance to the glulam beam design procedure in ASTM D3737 (1996) and as described by Moody (1974). The allowable design stress values and corresponding loads for each beam category are given in Table 2.

Table 2: Design Stresses and Loads

Beam	FRP (%)	Allowable Design Stress	Applied Load (1.25% of design)
Western Hemlock	0%	1400 psi (9.7 MPa)	5020 lbs. (22.3 kN)
	1%	2000 psi (13.8 MPa)	7060 lbs. (31.4 kN)
	3%	2300 psi (15.9 MPa)	8480 lbs. (37.7 kN)
Douglas Fir	0%	1550 psi (10.7 MPa)	5620 lbs. (25.0 kN)
	1%	1850 psi (12.8 MPa)	6700 lbs. (29.8 kN)
	3%	2700 psi (18.6 MPa)	10100 lbs. (44.9 kN)

Both concrete blocks were lowered simultaneously to apply the loads on the beams. A 30-second load-stabilization period was maintained before the first deflection reading was recorded. The time at which initial readings were recorded was taken as time T=0. Readings were taken every minute for the first five minutes, then at increasing intervals up to the first 2 days. Readings were then taken twice daily up to 16 days, then once daily up to 100 days, and finally once every two days for the remainder of the continuing study. Instrumentation and loading of the twelve beams took place in three different weeks with a group of four beams loaded every week. The group number, starting date, and beam designations are given in Table 3.

Table 3: Starting Dates for Loading

Group Number	Loading Date	Beam ID
1	5/7/97	WH-0%-1 WH-1%-2 DF-0%-1 DF-0%-2
2	5/14/97	WH-3%-1 DF-1%-1 DF-1%-2 DF-3%-1
3	6/11/97	WH-0%-2 WH-1%-1 WH-3%-2 DF-3%-2

Results and Discussion

The results of this continuing creep study as of 10/20/97 are given in Table 4 and Figures 2 and 3. The data is presented in terms of relative creep. Creep deflection is defined as the time-dependent deflection after the initial elastic deflection, i.e. $\delta_{creep} = \delta_{total} - \delta_{initial}$. Since the initial deflection can change greatly depending on species, loading, and environmental conditions, it is difficult to compare creep deflection data curves. Relative creep (Dinwoodie et. al. 1992) is used to normalize the data sets and is given by the following equation:

$$RC(\%) = \delta_{creep}/\delta_{initial} \qquad (1)$$

According to Equation 1, each beam starts at 0% relative creep at time T=0. The average relative creep of the two beams in each set is given in Table 4, for the specified time durations.

Table 4: Initial Deflection and Relative Creep(1)

Beam	FRP	Applied Load	Initial Deflection	Relative Creep (%)			
	(%)	1.25% of design		1 day	7 day	28 day	120 day
W. Hemlock	0%	5020 lbs. (22.3 kN)	1.3 in. (33 mm.)	10.7%	20.6%	34.9%	48.1%
	1%	7060 lbs. (31.4 kN)	1.9 in. (48 mm.)	11.8%	21.1%	34.7%	46.7%
	3%	8480 lbs. (37.7 kN)	1.9 in. (48 mm.)	10.6%	18.1%	31.3%	42.5%
Douglas Fir	0%	5620 lbs. (25.0 kN)	1.3 (33 mm.)	8.8%	16.0%	22.6%	28.9%
	1%	6700 lbs. (29.8 kN)	1.5 (38 mm.)	8.9%	14.5%	22.3%	29.3%
	3%	10100 lbs. (44.9 kN)	2.2 in. (56 mm.)	8.0%	11.8%	22.6%	24.3%

(1) each result is the average of data from two beams

The 120-day data indicates that the relative creep of the glass FRP reinforced beams is similar to that of the unreinforced beams. For the Western hemlock beams, the 4-month relative creep values range from 48.1% for no reinforcement to 42.5% for 3% reinforcement. For the Douglas-fir beams, the values range from 29.3% for 1% reinforcement to 24.3% for 3% reinforcement. While the data set is limited to two repetitions for each category, the beams with

3% reinforcement appear to have slightly lower relative creep ratios than the control beams. It should be noted that the results are important considering that the reinforced beams are subjected to significantly higher loads for the same size beam. For example, the 3% reinforced Douglas-fir beam has 80% more load than the unreinforced beam of the same size.

It is interesting to look at the relative creep curves of each beam individually. Figures 2 and 3 are plots of relative creep for the 6 Douglas-fir beams and the 6 Western hemlock beams, respectively. The time axis gives the date rather than elapsed time in days. The average estimated moisture content in the wood is also shown on these graphs. These plots illustrate the effect of the different starting dates and corresponding moisture content changes.

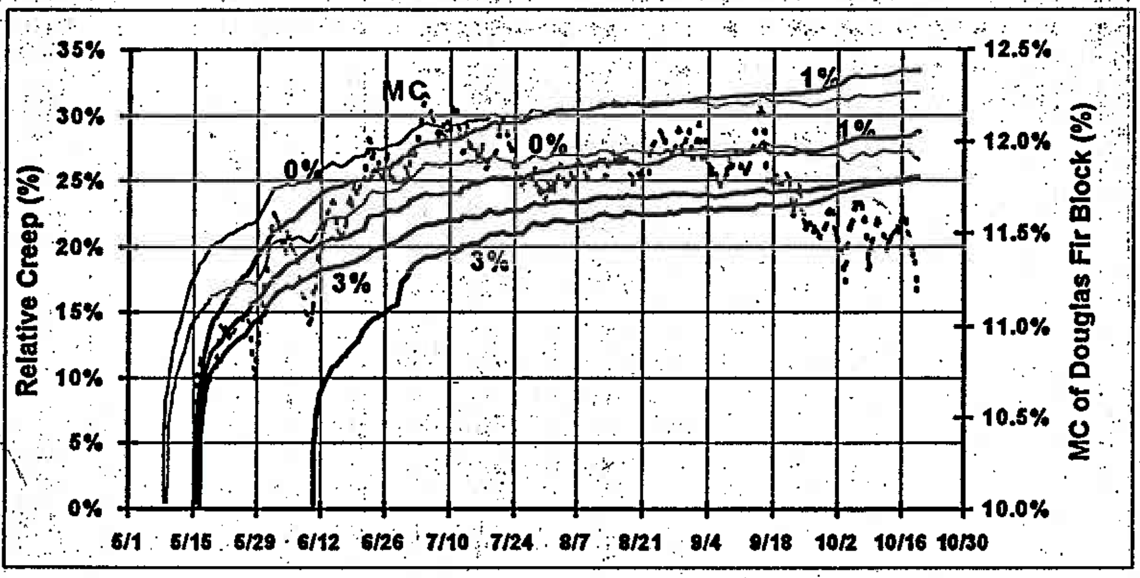

Figure 2: Relative creep and moisture content for six Douglas-fir beams.

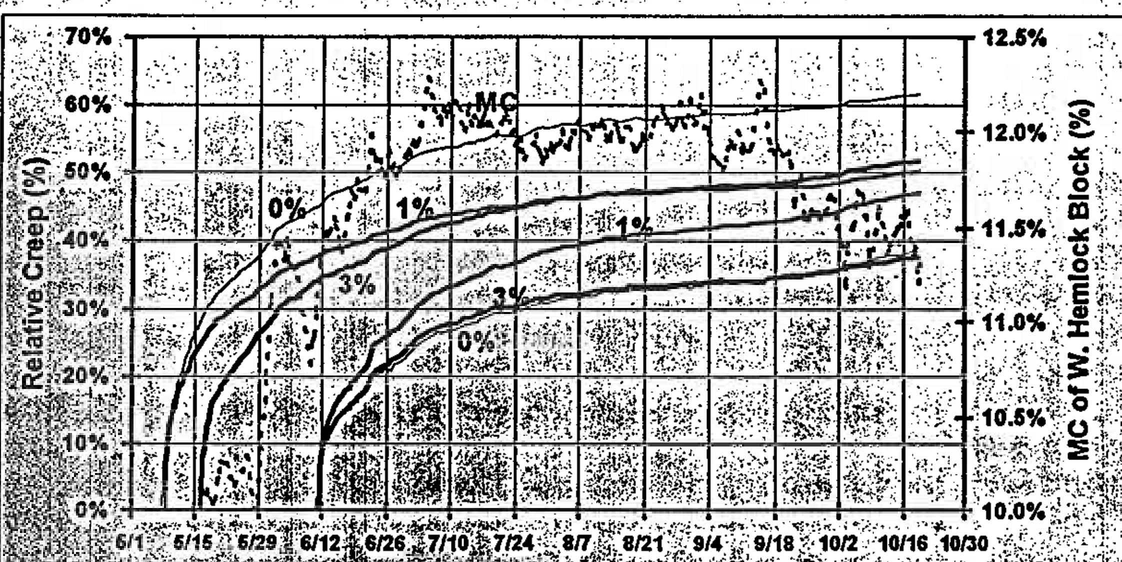

Figure 3: Relative creep and moisture content for six Western hemlock beams.

Creep curves may be characterized by three stages: primary, secondary, and tertiary creep. In primary creep, the creep rate is initially very fast, then steadily slows down. Once the creep rate reaches a constant value, the element is in secondary creep. If the rate further increases, the element has reached the tertiary stage and creep can increase rapidly and lead to creep rupture (Bodig and Jayne 1982). The twelve test beams were all in the primary creep stage for the first two months, then entered secondary creep where they have remained until this writing. The increased creep rate for the last two weeks of testing does not appear to be tertiary creep, since all the beams experienced increasing creep at the same time. This increase in creep rate may be attributed to the change in the moisture content of the beams, brought on by the onset of the heating season.

Figure 2 for Douglas-fir shows that the addition of 3% reinforcement decreases the relative creep. However, the curves for the controls and 1% reinforced beams are more difficult to interpret, since the curves for the two beams in the same FRP category do not match each other. Figure 3 for Western hemlock clearly shows the difference between the beams in Groups 1 and 2, which were started in May, and Group 3, which were started in June. All three beams in Group 3 have experienced less relative creep than the beams in Groups 1 and 2. However, within the two sets, both of which include one beam from each value of reinforcement, the beams with 3% reinforcement have creeped less than, or the same amount as, the two beams with less reinforcement.

The most probable reason for the difference in creep between Groups 1-2 and Group 3 may be explained by observing the change in moisture content over the course of the test. In May, the moisture content was only 10%, and it increased suddenly (+1.5%) at the end of the month. The moisture content then remained relatively constant for the remainder of the summer. The first two groups of beams were loaded before the moisture change occurred, while Group 3 did not experience this moisture change while loaded. As is characterized by the mechano-sorptive effect described previously, the change in moisture content of the wood (either adsorption or desorption) has the effect of increasing the creep rates in loaded members.

The moisture content also dropped dramatically in the last few weeks of the test, due to the start of the heating season. Since the creep rate had been relatively constant when the moisture content was steady, then changed dramatically as the moisture content changed, it can be postulated that the increased creep rates in the last two weeks of testing is due to mechano-sorptive creep.

Conclusions

Based on four months of accelerated creep testing at small ranges of temperature (65-75°F) and moisture content (10-12.5%), there appears to be little difference in the creep behavior of glass FRP reinforced glulam and unreinforced glulam. This is despite the fact that the applied loads on the reinforced beams are significantly higher than on the unreinforced beams.

This study was designed to test the effect of adding reinforcement to a glulam beam on its creep performance under relatively uniform temperature and humidity conditions. Although temperature was controlled, the effect of the seasonal changes in ambient humidity was a factor in the test. Changes in moisture content cause an increase in relative creep. As the test continues, it is apparent that time is becoming less of a factor on creep behavior and moisture content changes are becoming more important. Current plans are to continue this test for one year, then unload the beams and obtain the creep recovery curves.

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FRP POST-TENSIONING OF LAMINATED TIMBER BRIDGES

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ABSTRACT

In this paper, Glass Reinforced Plastic (GRP) tendons, rather than the commonly employed steel threaded bars, are used to post-tension a laminated wood deck. Monitoring of prestress forces after 260 days shows that properly designed GRP tendons can significantly reduce prestress losses. While initial results are encouraging, long term concerns with GRP tendons include creep-rupture in the E-glass reinforcement and environmental attack on the glass fibers. The paper also reviews some of the literature on creep-rupture of GRP.

KEY WORDS: Composites, prestress, wood

1. INTRODUCTION

In stress-laminated bridges, longitudinal wood laminations, consisting of either solid sawn lumber, glulam girders, LVL girders, or a combination of these are post-tensioned transverse to traffic (1). The prestress force causes friction to develop between the wood laminations, enhancing the load sharing capacity of the system and causing the behavior of the individual laminations to approach that of a continuous orthotropic plate.

One of the biggest draw-backs of stress-laminated bridges is the need to periodically retention them in service. Creep in the wood laminations over time can cause significant losses of prestress. According to the AASHTO Guide Specification for Stress-Laminated Decks(2), the initial prestress p_i applied to the deck should be 2.5 times the minimum required value p to compensate for losses due to creep and relaxation. Also, the AASHTO Guide Specification calls for re-stressing the deck to the same initial level p_i during the second and again between the fifth and eighth weeks after the first laminating.

Even though the re-stressing operation is relatively easy to perform and requires less than one day on most bridges, DOT engineers and maintenance personnel are often not at ease with a bridge design that needs periodic re-stressing. Stress-laminated systems would therefore gain more acceptance if re-stressing in service can be avoided. The objective of this paper is to evaluate the effectiveness of Glass Reinforced Plastic (GRP) tendons in reducing prestress losses in stress-laminated wood decks thereby avoiding re-stressing in service.

2. BACKGROUND

2.1 Prestress Losses in Stress-Laminated Wood Decks An early study on prestress losses in stress-laminated wood systems was conducted at Queen's University using small-scale laboratory models post-tensioned using 19 mm Grade 5 steel threadbars (3). The test results showed that long-term prestress losses may be as high as 65 % of the initial prestress. Restressing could however reduce the prestress losses to 45% of the initial prestress. Subsequent restressing did not show any further reduction of prestress loss. About 50% prestress loss was observed in the Herbert Creek Bridge, the first stress-laminated wood bridge deck, constructed in Ontario, Canada. Another laboratory study of prestress loss conducted on a 14 m x 3 m deck at the University of Wisconsin showed that the long-term prestress losses exceed 50% (4).

In order to reduce the magnitude of the long-term prestress losses, it is possible to install the bridge at a moisture content (MC) below the expected Equilibrium Moisture Content (EMC) for the site. The wood expansion in service will compensate in part for the loss of prestress in the deck. Another way to reduce the prestress losses may be to reduce the stiffness of the steel stressing system by using curved-washer type springs (Belleville springs) in series with the steel prestressing threadbars. Dagher et. al. (5) monitored a stress-laminated timber bridge in which one-half of the post-tensioning threadbars used Belleville springs. Results showed little difference in prestress between the half of the bridge with the Belleville springs and the other half of the bridge. The lack of effectiveness of the Belleville springs in this application was attributed at least in part to the corrosion of the spring stacks which caused them to partially "lock" in place.

2.2 Advantages of GRP Stressing System GRP tendons have a relatively low modulus, about one-fourth that of steel; they also have a relatively high strength, as much as twice that of Grade 50 steel. It is however the low stiffness of GRP tendons that makes them attractive in stresslam deck applications. The low stiffness of GRP tendons is expected to reduce prestress losses in stress-laminated decks. GRP tendons are also desirable because of their low cost compared to carbon or Kevlar composites.

2.3 Disadvantages of GRP System: Creep-Rupture While GRP tendons have the advantages stated above, there are some important concerns with regard to their use as prestressing elements in bridges. When the sustained load is above a minimum threshold, GRP exhibit creep-rupture failures. The creep-rupture problem in glass reinforced plastics has been known for a long time (6,7). Creep-rupture of GRP may be accelerated in aqueous or seawater environments (8,9). As stress levels increase, cracks in the matrix accelerate water penetration into the GRP which may cause surface pitting and strength reduction of the glass fibers.

that GRP tendons that are prestressed to an initial level of 50% of the ultimate short term strength are safe from creep rupture (10,11,12). Several concrete bridges have been built in Europe using E-glass prestressing tendons with polyester resin. These bridges which were constructed in the 1980's and the early 1990's were prestressed to close to 50% of ultimate tensile strength. No creep-rupture problems have been reported as of 1993 (10,13).

However, other researchers have tested GRP strands and have observed creep-rupture failures at load levels of less than 50% of the ultimate strength (14,15). Tendon anchors are very important role in the behavior of creep-rupture(16,17). This is because of the shear lag effects in the tendon caused by higher than average stresses in the outer portions of the cross section where the anchors grip the tendons. A poorly designed anchor can lead to premature failure of the tendon.

Environment plays a key role in the creep-rupture behavior of GRP. This is particularly true in marine environments where GRP might be exposed to salt water or salt water spray, and GRP tendons may lose considerable strength under prolonged exposure (18). Because of environmental exposure in bridge applications, the prestress level in GRP tendons should be kept lower than 50%, possibly in the 20-30% range. Even lower stresses may be required if there is significant shear lag in the prestressing elements used.

3. EXPERIMENTAL WORK

3.1 Relaxation Test on GRP Tendon in Steel Frame To verify the manufacturer's claims with regard to the relaxation rates in the particular GRP tendons used in this study, a GRP tendon was placed in a laboratory steel reaction frame under an initial stress equal to approximately 50% of the ultimate strength of the tendon. The prestress force, temperature and humidity were monitored daily. As of this writing, the test has been on-going for 18 months and there is no evidence of a significant relaxation loss in the GRP tendon. Figure 1 shows the results of the relaxation test to date.

3.2 Laboratory Creep Test of Wood Deck Stressed with GRP Tendons A laboratory test was conducted to study prestress losses in a wood deck post-tensioned with GRP tendons. The stress-laminated wood deck was approximately 5 m x 3 m, in which rough-sawn 5 mm x 25.4 mm (2 inch x10 inch) eastern hemlock wood laminations ran in the 5-m direction and the GRP tendons ran in the 3 m direction. There were ten GRP tendons in the deck and six of them were instrumented using load cells. The load cells were calibrated using a hydraulic universal testing machine before they were mounted on the deck. The creep test was conducted in a relatively constant indoor environment with temperatures ranging from 20^0C to 30^0C (68^0F to 86 ^{0}F). Although there was a larger fluctuation of th relative humidity inside the lab (17%-54%), the moisture content of the wood remained below 6 percent throughout the creep test.

The initial prestress introduced between the wood laminations was 520 kPa (75 psi). The corresponding prestressing force in each GRP tendon was 62 kN (14 kips). The GRP tendons have an ultimate tensile strength of 116 kN (26 kips). The 63 kN (14 kips) initial tendon stressing force is 54% of the tendon's ultimate strength. This high value (54%) was used in the laboratory to simulate a worse-case scenario for creep-rupture of the GRP. As stated earlier field applications should use lower stresses in the GRP, in the order of 20%-30% of ultimate.

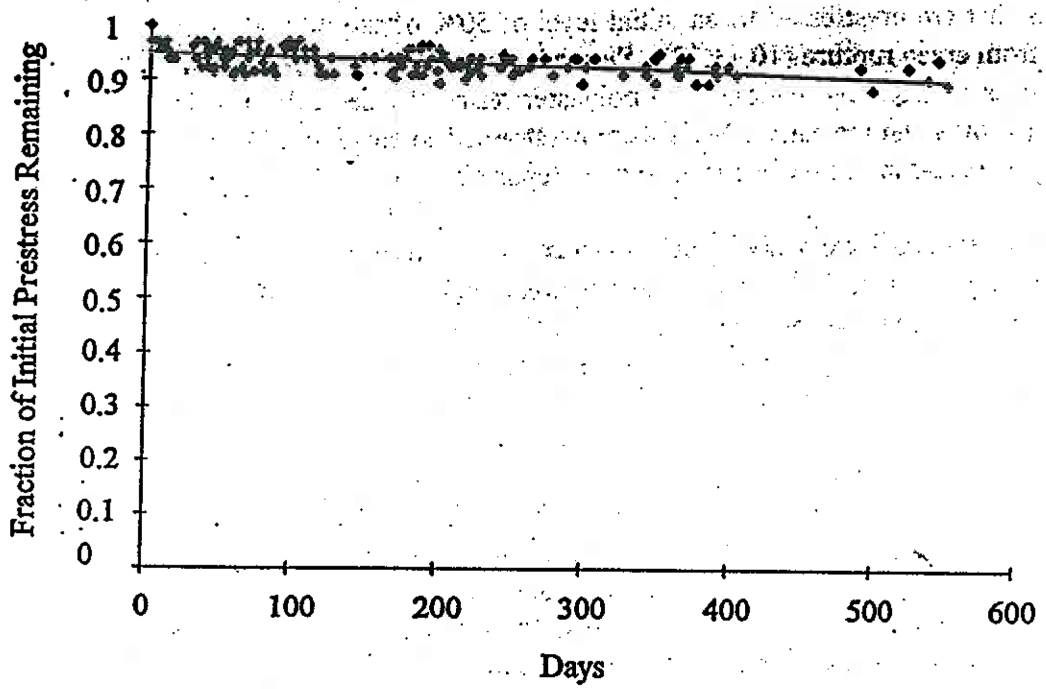

Figure 1. Relaxation Loss in GRP Tendon

The 520 kPa (75 psi) initial prestress introduced between the wood laminations is intentionally lower than the 860 kPa (125 psi) commonly used with steel stressing systems. The reason for the lower initial prestress is to take advantage of the lower anticipated prestress losses with the GRP system. The prestress was applied using an ENERPAC center-hole hydraulic jack. Tensile forces in each tendon were brought up to 63 kN (14 kips) sequentially from one end of the deck to the other. Only two passes were required before the desired prestress force in the first tendon (that was stressed in the second pass) was within 5% of the target value of 63kN (14 kips). This is a significant development because comparable steel stressing systems may require as many as five or more passes before the target forces in the prestressing bars are reached.

4. TEST RESULTS AND CONCLUSIONS

In Figure 2, the average prestress loss in the GRP tendons is compared with the results obtained in the Queen's University study discussed earlier (3). It is clear that the GRP tendons appear to significantly reduce the prestress losses over steel threadbars. The following is concluded:

1. After 260 days, the GRP prestress loss is less than 30% of the initial prestress. The corresponding value for the steel threaded bars used in the Queen's University study was about 60% of the initial prestress after 110 days.

2. Since prestress losses are reduced, the initial prestress in the GRP-wood deck does not need to be as high as is in the steel threadbar-wood deck, i.e. 860 kPa (125 psi). For the configuration tested, with an initial wood prestress of 520 kPa (75 psi), the residual prestress after 260 days is over 0.7x520 kPa = 364 kPa (52.5 psi). With a steel threadbar deck at an initial prestress of 860 kPa (125 psi), the remaining prestress after 110 days would be nearly 0.40 x 860 kPa = 344 kPa (50 psi).

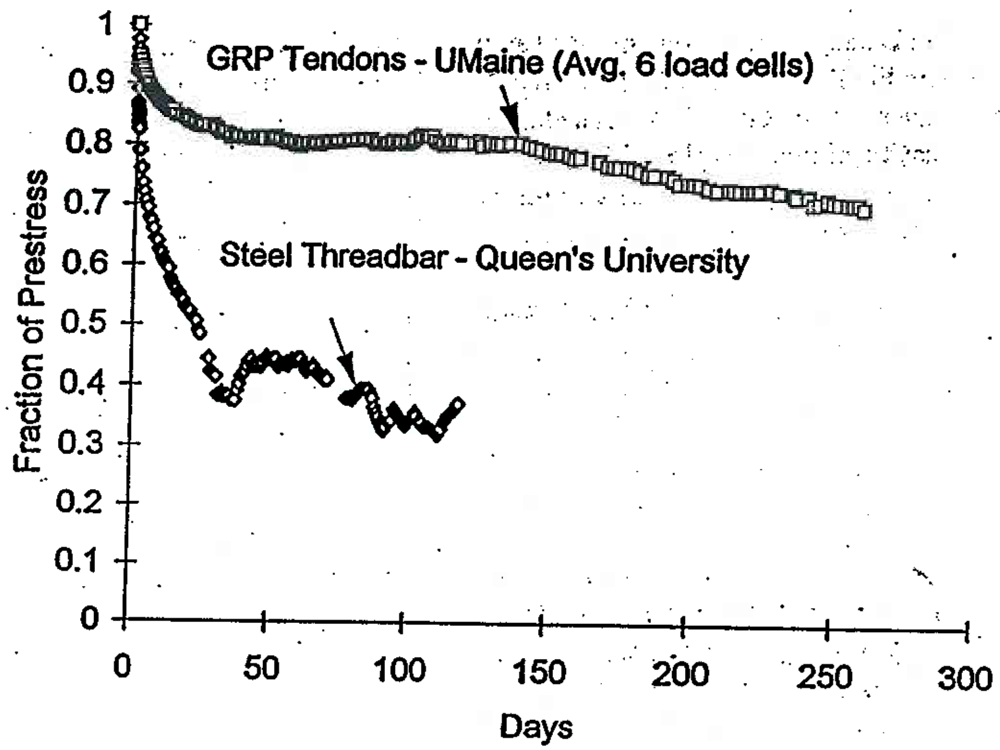

Figure 2. Prestress Loss in Wood Deck: GRP vs Steel

3. The GRP system significantly reduces the number of passes required to complete the initial prestress. For the tested configuration, only two passes were required.

4. The prestress in the GRP in this laboratory study was near 54% of the ultimate strength of the GRP. This high stress level in the GRP was used to simulate a worse-case scenario in the laboratory and is not recommended in field applications. To avoid creep-rupture failures which may be accelerated by environmental attack, it is recommended to keep the prestress level in the GRP at 20-30% of the ultimate strength.

Initial results are encouraging and it may be possible to significantly reduce or even avoid re-stressing in service. However, more laboratory and field testing are necessary to further verify the results described in this paper.

Acknowledgments

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Effect of FRP Reinforcement on Low Grade Eastern Hemlock Glulams

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Abstract

The benefits of reinforcing glulam beams made with eastern hemlock, an under-utilized wood species in the state of Maine, are discussed. Nine beams reinforced with fiber-reinforced plastics (FRP) on the tension side and three unreinforced controls were instrumented and tested to failure in four-point bending. Low, medium, and high quality wood were used in the experimental study. FRP reinforcement ratios ranged from 0.3% to 3.1%. A nonlinear numerical model that predicts the performance of the FRP-glulam beams through the entire load range was developed and its predictions are compared with the test results.

Keywords: glulam, FRP, reinforcement, nonlinear model, strength, stiffness, ductility.

Introduction

Glued laminated wood (glulam) has been in use since the late 1800's. Research on glulam at the USDA Forest Products Laboratory in Madison, Wisconsin began in the 1930's. Development of glulam in following years in the United States was encouraged [illegible] lack of adequate solid timbers and the high [illegible]and for large timbers created by the World War II effort (Freas and Selbo, 1954).

Glulam can be fabricated in many shapes and sizes, and has been used in numerous applications including keels for boats, arches for airplane hangers, churches, timbers for floor and roof systems, dome structures, transmission poles, along with girders and decks for timber bridges.

With recent changes in availability of forest resources, high quality laminations necessary for glulam design "have become increasingly difficult to procure, and more expensive as well." (Leichti, 1993, p. 3) Moreover, glulam, like reinforced concrete, can be reinforced in tension to more efficiently utilize the wood's compressive strength. Fiber reinforced plastics (FRP) offer good promise to serve both as a substitute for the high quality wood laminations and as reinforcement for glulam beams.

Over the past decades both FRP and non-FRP materials have been used to reinforce or prestress wood beams. With regard to non-FRP materials, Mark (1961) studied the effects of bonding aluminum to the compression and tension faces of wood core sections of eight different wood species. Sliker (1962) bonded aluminum sheets between various layers of laminated wood beams. Bohannan (1962) reinforced glulam beams of low-grade Douglas-fir using pretensioned steel wire strands in the tension zone.

Peterson (1965), in a study similar to Bohannan (1962), reinforced low-grade Douglas-fir glulam beams with a prestressed flat steel strip bonded in the tension zone. Lantos (1970) reinforced rectangular laminated wood beams with steel rods. Stern and Kumar (1973) studied the effect of steel plate reinforcement for vertically laminated timber beams. Coleman and Hurst (1974) reinforced No.2 southern pine beams with light gage steel reinforcement. Hoyle (1975) tested members composed of nominal dimension lumber with toothed steel plates between lumber pieces. Bulleit, Sandberg, Woods (1989) reported on Spruce-Pine-Fir glulam beams reinforced in the tension zone with special steel-reinforced tension laminations.

Prior to 1990, a number of studies on wood beams reinforced with fiber and FRP materials were also conducted. Wangaard and Biblis (Wangaard, 1964; Biblis, 1965) studied the effect of bonding unidirectional fiberglass/epoxy reinforced plastic to the compression and tension faces of wood cores of various species. Theakson (1965) studied the feasibility of strengthening both laminated and solid wood beams with fiberglass. Krueger and Sandberg (1974 b) studied laminated timber reinforced in the tension zone with a composite of high-strength bronze coated woven steel wire and epoxy. Krueger and Eddy (1974 a) carried out research similar to that of Kruegar and Sandberg (1974 b). Spaun (1981) studied finger-jointed western hemlock cores reinforced with wood veneers and fiberglass rovings.

In the nineties, research on wood beams reinforced with fiber and FRP materials has increased. Plevris and Triantafillou (1992) studied the effect of reinforcing fir wood with carbon/epoxy fiber-reinforced plastics. Plevris and Triantafillou (1995) also discussed the creep behavior of FRP-reinforced wood. Triantafillou and Deskovic (1992) studied the effect of prestressed carbon/epoxy FRP (CFRP) reinforcement bonded to European beech lumber. Davalos, Salim, Munipalle (1992) discussed the response of small yellow-poplar glulam beams reinforced on the tension side with glass/vinylester FRP. Tingley and Leichti (1993) discussed glulam made from lower grade ponderosa pine reinforced in the tension zone with pultruded kevlar and carbon FRP. Abdel-Magid, Dagher, and Kimball (1994) studied nominal 2x4 hemlock beams reinforced tension with carbon/epoxy and kevlar/epoxy FRP. Sonti, Davalos, Hernandez, Moody, and Kim (1995) discussed yellow-poplar glulam reinforced with pultruded glass/vinylester FRP in tension or both in tension and compression. Dailey, Allison, Minneci, and Bender (1995) studied glulam reinforced in the tension zone with pultruded glass/resorcinol-modified phenolic FRP sheets.

While timber has been successfully reinforced over the past few decades using various materials and reinforcing techniques, very few of these methods of reinforcing timber have reached the commercial market (Bulleit, 1984). As argued by Bulleit et. al. (1989, p. 433), there are several possible reasons for this lack of commercialization of reinforced timber: "(1) The material used to reinforce the wood was not commonly used in building applications; (2) the reinforcing material was too expensive; and (3) the fabrication required an additional and, thus, cost-increasing step in the laminating process." Another reason may be the incompatibility between the reinforcing material and the wood.

FRPs are a versatile class of materials that can be engineered to overcome the incompatibility problems with the wood. Because of FRPs falling cost and potentially simple incorporation into existing glulam manufacturing processes, this class of materials offers a good potential as a reinforcement for wood (Kimball, 1995). This paper describes an experimental and numerical study on reinforcing eastern hemlock glulams with FRP. Eastern hemlock is a relatively inexpensive, abundant and under-utilized Maine wood species with relatively low mechanical properties.

Experimental Work

A total of twelve glulam beams were fabricated and tested statically to failure (See Table 1). The beams had a clear span of 16 feet and a cross-section of 3 3/16 inches by 12 inches. The twelve beams consisted of three control (unreinforced) beams and nine beams reinforced with varying amounts and types of FRP in the tensile zone. Because of variations in lay up, three of the reinforced beams cannot be directly compared with the controls. These are described as Non-Comparison beams in Table 1.

All beams used No.2 and better 2x4 eastern hemlock. The No.2 visual grade material occupied over 75% of the sample. The lumber was condition to a 12 percent moisture content prior to laminating. Using MOE data, the wood was divided into three quality categories: Low, Medium and High. Both unreinforced controls and reinforced beams were constructed of each of the three wood categories.

Control Beams
Unreinforced control beams consisted of eight 1 3/8 inch laminations, with a 3/4 inch 'bumper-strip' lamination added to the outer wood tensile lamination.

FRP-Reinforced Glulam Beams
The reinforced beams were designed in the same way as the control beams with the exception of FRP placed between the 'bumper strip' and the outer tensile wood lamination. A transformed section of a typical reinforced beam is shown in Figure 1. Table 2 summarizes the properties of the two FRP types used in the beams.

Testing
The glulam beams were tested in four-point bending over a simple span as shown in Figure 2. The beams were braced to prevent lateral-torsional buckling and were tested according to the procedures outlined in ASTM D198-84.

Strain gages were applied throughout the depth of the beams within the constant moment region. A dial gage and LVDT were used to measure deflections. The beams were loaded at a rate of approximately 1000-pounds per minute. Readings included beam [illegible], load head displacement, strain gage, LVDT and dial gage over the duration of the test.

Table 1 - Beam Characteristics

Beam No.	FRP Type	Beam Depth (in)	Reinforcement Ratio (%)	Wood Quality
Comparison Beams				
10	-	11.75	0	L
9	-	11.75	0	M
11	-	11.75	0	H
2	1	12.12	3.1	L
1	1	12.12	3.1	M
3	1	12.12	3.1	H
6	2	11.88	1.1	H,M
5	2	11.88	1.1	M
7	2	11.88	1.1	H
Non-Comparison Beams				
4	1	12.12	3.1	N/A
8	2	12.00	2.1	L
[illegible]	2	11.79	0.3	L

L=Low, M=Medium, H=High, N/A=Not Applicable

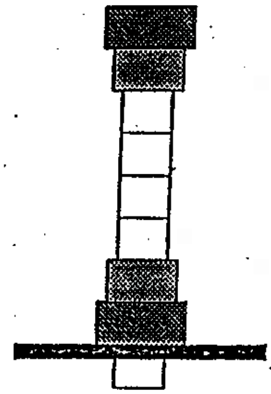

Figure 1 - Transformed Section

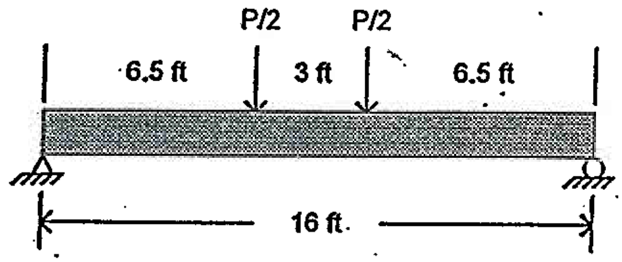

Figure 2 - Test Setup

Table 2 - FRP Properties

Property	FRP1	FRP2
Ultimate Tensile Strength (ksi)	114	137
Longitudinal Tensile MOE (Msi)	6.7	19.1

Load Deflection Data
For the low (L) quality comparison beams, the reinforced beam was 43% stronger and 31% stiffer than the control. In addition, the reinforced beam was more ductile than the control beam, deflecting 42% more at failure.

Figure 3 shows the experimental load-deflection curves for the medium (M) quality comparison beams including the control, FRP1 and FRP2 reinforced beams. The FRP1 reinforced beam was 51% stronger and 32% stiffer, while the FRP2 reinforced beam was 33% stronger and 37% stiffer than the control. The FRP1 reinforced beam showed improved ductility over the control beam.

Table 3 - Bending Test Data

Beam No.	FRP Type	Rein-force-ment Ratio (%)	Wood Quality	Max. Load (lb)	Max. Deflec-tion (in)	Beam MOE (Msi)	Str. Incr-ease (%)	MOE Incr-ease (%)
Comparison Beams								
10	-	-	L	8200	2.12	1.28	-	-
9	-	-	M	10780	3.08	1.26	-	-
11	-	-	H	12040	3.25	1.38	-	-
2	1	3.1	L	11750	3.02	1.68	43	31
1	1	3.1	M	16250	4.48	1.66	51	32
3	1	3.1	H	14900	3.57	1.78	24	29
6	2	1.1	H,M	15070	3.82	1.61	25	28
5	2	1.1	M	14310	3.05	1.72	33	37
7	2	1.1	H	15270	3.76	1.72	27	25
Non-Comparison Beams								
4	1	3.1	N/A	9040	1.94	1.74	-	-
8	1	2.1	L	12800	3.29	1.58	56	23
12	2	0.3	L	10930	3.54	1.26	33	-2

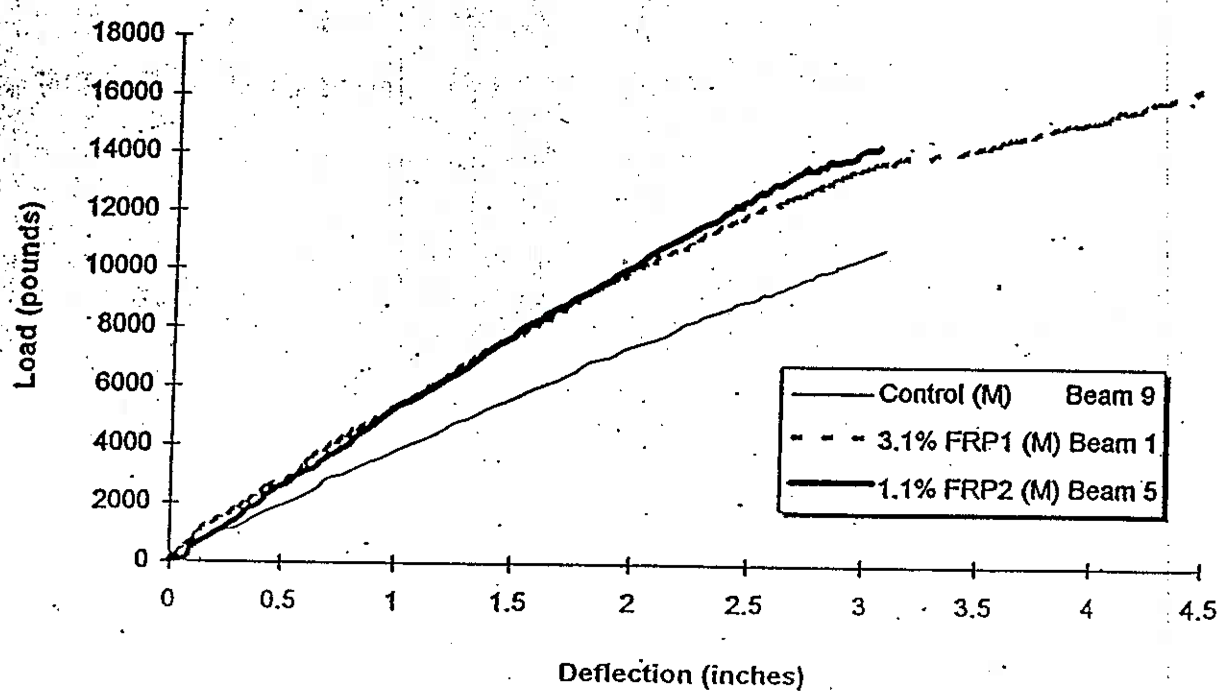

Figure 3 - Load-Deflection Curves for the Medium Quality Beams

The high (H) quality comparison beams include the control, FRP1 and FRP2 reinforced beams. The reinforced beams showed increases in strength of approximately 24 to 27% over the control. The reinforced beams showed increases in stiffness of 25 to 29% over the control.

With regard to the three non-comparison beams, the FRP1 reinforced beam with 3.1% reinforcement ratio failed prematurely in a wood tension lamination due to wood shake. Therefore, it is not compared to a control beam. The FRP1 reinforced beam with 2.1% reinforcement ratio showed an increase in strength and stiffness of 56% and 23%, respectively, over the low (L) quality control. Beam 12, which had only 0.3% FRP reinforcement, showed 33% gain in strength over the low (L) quality control beam but practically no change in stiffness.

Strain Gage Data

Stresses in the FRP at failure were calculated using strain data and beam MOE values. The tensile stresses in the FRP at onset of beam failure remained well below ultimate strengths in all cases. The stress in the FRP1 at beam failure was about 25% of its ultimate strength. The stress in the FRP2 at beam [fail]ure was less than 50% of its ultimate strength. However, the FRP2 reinforcement failed in interlaminar shear, thereby initiating overall beam failure.

Nonlinear FRP-Glulam Model

A nonlinear numerical model was developed to study the behavior of FRP-glulam beams for all stages of loading through failure. One objective of the model was to predict the full load-deflection curve of the laboratory beams. Using the nonlinear properties of the constituent materials, the moment-curvature relationship of a section is first determined. The moment-curvature relationship, together with the beam geometry and loading configuration, is then used to determine the load-deflection curve of the beam.

The model follows in some respects work by Plevris and Triantafillou (1992), Bazan (1980), and Buchanan (1990). The numerical model was implemented using two computer programs. The first computer program determines the moment-curvature relationship of a FRP-glulam section. The second computer program uses this moment-curvature relationship to determine the load-deflection curve for a beam loaded in four-point bending.

Comparison of Numerical and Experimental Results

To illustrate the application of the numerical model, the numerical and experimental load-deflection curves for reinforced beam 1 are compared in Figure 4. Also, the actual and predicted ultimate loads and deflections for the three medium (M) quality beams (beams 1, 5 and 9) are compared in Table 4.

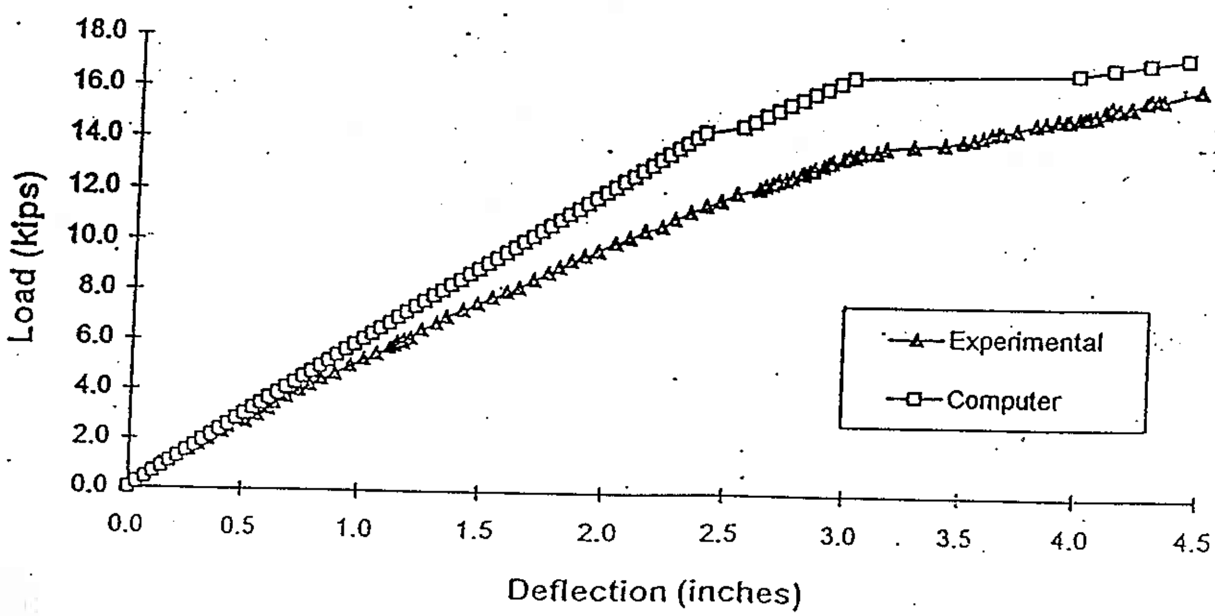

Figure 4 - Comparison of Experimental and Numerical Load-Deflection Curves for Beam 1

Table 4 - Comparison of Experimental and Numerical Results for Beams 1, 5 and 9

Beam Type	Analytical Load (lb)	Experimental Load (lb)	Error (%)	Analytical Deflection (in)	Experimental Deflection (in)	Error (%)
Beam 1	17,430	16,250	7.3	4.43	4.48	-1.1
Beam 5	15,990	14,310	11.7	2.96	3.05	-3.0
Beam 9	10,240	10,780	-5.0	2.52	3.08	-18.2

Discussion

Glulam made from one of Maine's relatively weak and under-utilized wood species, eastern hemlock, was reinforced with FRP in the tension zone. The reinforced beams performed very well and showed substantial gains in strength (up to 56%) and stiffness (up to 37%) by the addition of 1-3% FRP reinforcement.

The increased beam strength was due in part to the more efficient utilization of the compressive strength of the wood. Using familiar reinforced concrete terminology, the beams tested performed as over-reinforced beams. In an over-reinforced beam, failure occurs by compression of wood fibers near the top of the beam. The region of failed wood in compression propagates from the top of the beam down until the beam ultimately fails. The ductile failure of wood in the compression zone leads not only to increased strength but also to increased ductility of the reinforced beam. It should be noted that in contrast with reinforced concrete, an over-reinforced wood beam is ductile whereas an under-reinforced concrete beam is ductile.

In general, the largest increases in strength were obtained with the lower grades of wood. It appears therefore that the highest value-added benefits resulting from this technology may occur with the lower grades of wood. This is due to the lower grades of wood having a larger difference in relative tension/compression strength values, which can be remedied by adding FRP tension reinforcement.

A nonlinear numerical model was developed to study the ultimate strength behavior of FRP-glulam beams. The model was relatively successful in predicting the performance of the beams. It will be a useful tool in optimizing the lay-up of glulam beams.

Concluding Remarks

Fiber reinforced plastics appear to have good potential to serve as a substitute for the high quality wood laminations necessary in glulam. Placing the FRP in the beam tension zone uses the FRP's high tensile strength and stiffness to boost the strength and stiffness and ductility of the hybrid beam. Commercial success will ultimately depend upon the future savings of removing wood laminations being greater than the future expense of adding FRP reinforcement.

In addition to the cost/benefit issue, further research is necessary before FRP-reinforced wood beams are widely used in bridge applications. One major concern is the long-term durability of the FRP-wood interface in a bridge environment. The in-service hygro-thermal mechanical stresses that will develop at the wood-FRP interface need to be evaluated carefully. In addition, the interaction between moisture, temperature, fatigue, and their effect on bond strength and creep behavior of the system are not entirely understood. Fundamental research at the University of Maine is on-going to address these and other related issues.

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