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Structural evaluation and design procedure for wood beams repaired and retrofitted with FRP laminates and honeycomb sandwich panels

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ABSTRACT

As compared to other structural applications of polymeric composites, limited information is available on structural behavior of wood members strengthened with polymer composites. The focus of this paper is to evaluate the structural performance and practical use of wooden beams repaired and retrofitted with fiber-reinforced-polymeric (FRP) composites. The paper presents a summary results of an experimental study on the behavior of both Douglas Fir and Glulam wood beams repaired and retrofitted with different composite strengthening systems. In addition, the paper presents a simplified design procedure to predict the capacity of timber beams strengthened with FRP composites. Two types of composites; wet layup laminates and sandwich panels, and two lamination schedules; unidirectional and bidirectional, and two lamination geometries; U-laminate and flat laminates were evaluated. For "flexure/shear" wood beams repaired and retrofitted with bidirectional, carbon/epoxy U-shaped wet layup laminates, a total of eight Douglas Fir (Dug Fir) Larch # 1 wood beams were tested to failure. For "flexure-only" wood beams retrofitted with flat unidirectional laminates, both wet layup and precured sandwich honeycomb composites were evaluated. Experimental results indicated that the use of composites as external repair and rehabilitation elements resulted in an appreciable increase of both strength and stiffness. A practical case study is also presented that provides a step-by-step procedure for analyzing and designing a polymeric composite system for repair of partially damaged wood girders by fire.

1. Introduction

For the past decade or so, polymer composites were introduced to the construction industry as a valuable alternative structural system for repair and rehabilitation of reinforced concrete [7,16–18], steel [14], and masonry [15] structures. These applications were extended for be utilized for potential use as repair and capacity upgrade of wood structural members. One of the first applications was initiated in mid-1990, where E-glass/epoxy laminates were used to restore damaged wooden utility poles. This application was further studied by Ref. [23] and by Ref. [26]. Similar application of strengthening wood piles with composites was also investigated (*e.g.* Ref. [11]. A hybrid glued-laminated wood products were also developed by introducing thin laminates of E-glass/epoxy composites between the wood layers [2]. Gilfillan et al.[6] studied structural behavior of several Irish-grown Sitka timber beams strengthened with both composites and steel. These beams were evaluated under both short- and long-term mechanical loading. Experimental results indicated that an appreciable strength gain was been achieved for beams strengthened with FRP composites. Buell and Saadatmanesh [3] evaluated the behavior of timber bridge beams strengthened with carbon/epoxy composites. The results of their study indicated that the use of carbon/epoxy composite laminates for strengthen timber bridge beams has resulted if a significant increase in both bending and shear strength with a nominal upgrade of beam stiffness. Lyons and Ahmed [12] discussed different factors affecting the bondline strength of wood members including adhesive properties, wood surface conditions and moisture content, as well as the effect of service environment. The results of the study indicated that the roughening wood member surface has a positive impact on bondline strength and that the use of hydroxyl methanol resorcinol (HMR) improves the bond strength in wet conditions. The temperature-dependent creep of different adhesives for wood members and the influence of fillers on temperature stability of epoxy adhesives used for wood-composites application were investigated by Richter and Steiger [25]. Li et al. [10] conducted analytical and experimental investigation on the flexural performance of beams fabricated from two wood species, namely; Tsuga Chinensis and Cunninghamia

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Lanceolata that were retrofitted by carbon/epoxy composite. Similar conclusions were drawn confirming the increase of both the strength and stiffness of the retrofitted beams. The effect of the surrounding environment on the performance of FRP/wood adhesively bonded epoxy joints was evaluated by Ref. [27]. The authors observed that the first twenty months of environmental exposure have a significant effect on bondline failure due to tensile shear strength reduction. Modeling of wood beams strengthened with different CFRP composite laminates was reported by Ref. [8]. The used of FRP near-surface mounting (NSM) strengthening system was evaluated by Ref. [5]. In the study, flexural performance of fir and pine wooden beams strengthened with NSM carbon-fiber-reinforced polymer (CFRP) composite plates and rods were experimentally evaluated. Their experimental results verified the efficiency of both NSM systems in improving both flexural strength and stiffness of the strengthened wood beams.

2. Motivation & objective

Composites have a high potential as an alternative sustainable reinforcing repair system to existing under-deigned, historical and structurally deteriorated constructed facilities to increase the flexural strength and to enhance the ductility of existing and new wood structures (refer to Fig. 1). Increasing the structural capacity of wood members to carry heavier loads allows the structural engineer to reduce the cross-sectional area of the wood member for the same load resulting in a major reduction of wood consumption that lead in saving millions of trees worldwide. Kukule and Rocens [9] confirmed this fact through a study that indicated that the use of prestressed CFRP composite laminates for strengthening wood members can reduce the wood consumption by about 31%. Although, this potential have been verified by several pilot studies, however, its application is relatively limited due to several reasons including lack of design standards, and most important, the limited information on both the short- and long-term structural behavior of wood members externally reinforced with polymer composites. In order to successfully introduce this application to the construction industry, more analytical and experimental verification studies are essential. In addition, there is need for developing reliable simplified design procedures for this potential application. In this study, a total of eight large-scale tests were conducted to assess the flexural behavior of wooden members repaired and retrofitted with external composite laminates and to identify the predominated failure mode of such hybrid structural members. One of the major objectives of this study is to highlight the importance and advantages of adopting the proposed “flexure/shear” strengthening protocol for wood flexural members. As known, wood is an orthotropic material with mechanical properties dependent on grains orientation of the flexure member. Wood members subjected to flexural stresses with grains parallel to the member's longitudinal axis have limited interlaminar shear strength, and hence, the amount of additional external flexural reinforcement (*e.g. using E-glass/, carbon/or Aramid/epoxy laminates*) must be proportional and limited to such strength, otherwise, interlaminar shear failure is unavoidable (refer to Fig. 2). For this reason, and in order to take full advantages of the composite laminates placed at the tension side of the strengthened wood member, it is important to adopt what is referred to in this paper as “flexure/shear” strengthening scheme. In this scheme, a bidirectional [0°/90°] U-shaped composite laminate is proposed. The fiber volume fraction in each orthogonal direction is controlled by the target flexural strength enhancement capacity. Once this capacity is identified, the “parallel-to-grain” fiber volume fraction (0°-direction) can be calculated. Consequently, the resulting interlaminar shear stresses are determined that are used in calculating the “perpendicular-to-grain” fibers volume fraction (90°-direction). The use of this protocol will eliminate or minimize the potential premature interlaminar shear failure prior to reaching the full-capacity of the composite laminate placed at the tension side of the flexural wood member leading an optimum use of composites (see Fig. 2). In

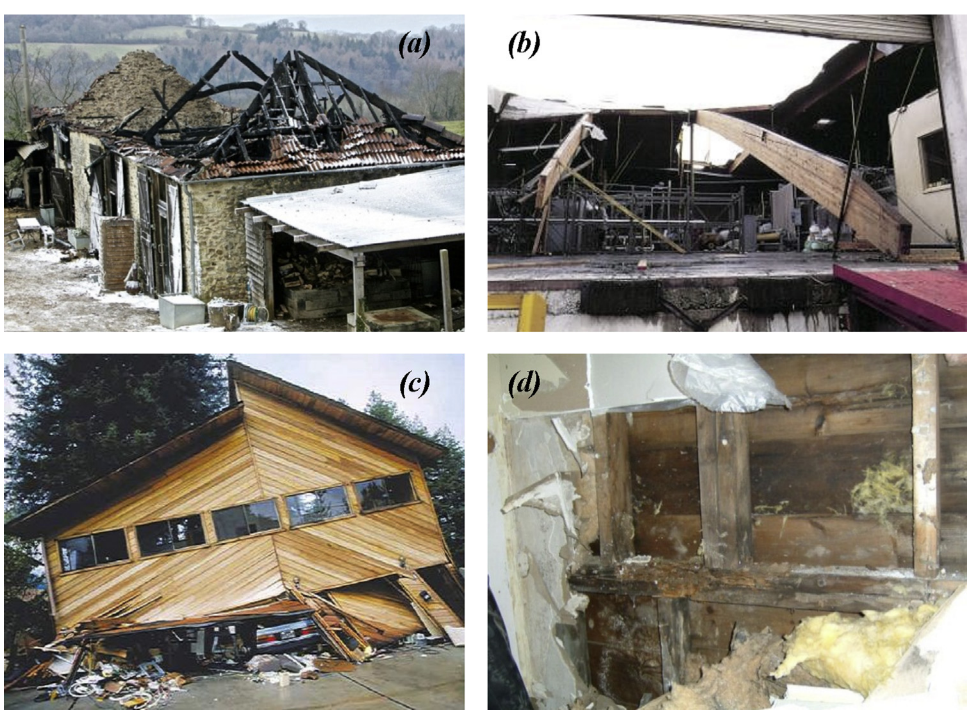


Fig. 1. Examples of Potential Damages of Wood Structures: *(a) Fire Damage, (b) Inaccurate Connection Details and Design Faults, (c) Seismic Capacity Deficiency, (d) Environmental Deterioration.*

Fig. 2. Interlaminar shear failure of wood members subjected to flexure.

addition, the presence of the shell laminate around the sides of the wood member enhances both physical and thermo-mechanical properties of the wood member by covering natural defects such as knots, as well as, by increasing the fire resistance of the member by the formation of additional polymeric char layer in addition of contributing in the stability of the moisture content of the member. Another objective of this study is to explore a new non-conventional composite repair protocol through the use of light-weight prefabricated composite sandwich panels with mechanical shear connectors. The advantages of sandwich structures in flexure have been demonstrated in this study through full-scale experimental evaluation. This protocol increases the quality control of the bondline through the availability of pre-treated surfaces and presence of nails or screws shear connectors that assist in providing the necessary clamping force during bondline curing. If height limitation is not imposed, the added inertial via the sandwich composite section will provide additional flexural stiffness to the strengthened member.

As stated earlier, absence of design codes, standards and manuals for this application is considered one of the major obstacles preventing the wide use of this excellent strengthening alternative. This paper presents a practical design example to illustrate, step-by-step, procedures used in predicting the short- and long-term behavior of glulam girder that was partially damaged during a fire incident and was repaired with FRP honeycomb sandwich composite panel (H-Lam).

3. The H-Lam bolted/bonded sandwich composite strengthening system

Based on the literature review related to this study, it was found that this study is one of the first pilot studies where bonded/mechanically-fastened sandwich composite panels (*designated as H-Lam*[1] *herein*) were used in strengthening of wood structures. This novel system was designed and developed by the author. The use of these newly-developed composite sandwich panels [13] was found to be very successful in upgrading flexural strength of reinforced concrete beams [19], steel bridge members [14]) and was also was utilized as collision protection for RC bridges [24]. The advantages of using these sandwich panels in this application includes: (i) ease and rapid installation, (ii) increase in quality control of the pre-fabricated materials and panels face sheets shop pretreatment surfaces, (iii) light-weight features, (iv) the presence of stiffened holes allows for drilling metal screws or nails that act as both shear connectors and prior to adhesive curing as a temporary clamps, (v) superior fire properties due to higher glass-transition-temperature (T_g) and the use of phenolic matrix, (vi) superior fire resistance of both face sheets and core materials, and (vi) overall all economic advantages. It should be noted that this study took the advantages of phenolic resins with its excellent fire properties for wood members strengthening applications. The sandwich panels used in this study are fabricated from a low-smoke carbon/phenolic facings bonded to an aramid honeycomb core using high-temperature, high-press manufacturing process. The fiber architecture of the two face sheets were in the form of cross-ply $[0°/90°/c]_s$ (c = *half of the core thickness*) thin-laminates with a unit face laminate thickness of 0.75 mm (0.03″). Equal fiber volume fractions for both longitudinal and transversal directions was used, with an overall fiber volume fraction for both directions equal to 70% for face sheet laminates, assuming a zero void ratio. Table 1 presents geometrical and mechanical properties of the H-Lam system used in this study. The typical H-Lam CFRP panels' details and coupons used for extracting the mechanical properties of these sandwich panels are presented in Fig. 3.

In developing this Honeycomb system (see Fig. 4), several design parameters were considered including: (i) resin compatibility with wood, (ii) pre-treatment of the composite surface to be bonded to wood, (iii) providing self-clamping mechanisms to hold the H-Lam in place during the anti-gravity application at the site, (iv) fire resistance, (v) economic considerations and (vi) strength and stiffness requirements. These design targets were achieved by optimum design of face sheets and core; pretreatment of the face sheets, selection of fire retardant matrix and core materials, introduction of central through-the-thickness stiffeners and holes at equal spacing of 25.40 mm (1.00″) along the span of the panel. With this arrangement, the applicator is able to use nails or screws to hold the H-Lam in position after applying the low-viscosity primer and the high-viscosity adhesives in addition its partial role as mechanical shear connectors (refer to Fig. 4). The width of the H-Lam is 152.0 mm (6.0″) and the total thickness of the H-Lam is 10.0 mm (0.40″) with an average face laminate thickness of 0.74 mm (0.029″). The depth/width (d/b) ratio used for evaluating

Table 1
Geometrical and mechanical properties of FRP sandwich composite H-Lam panels* used for Group "***B***" specimens (*Low-Density Honeycomb Nomex™ Core*).

Thickness of FRP face sheets, mm [inch]	0.65 [0.026]
Thickness of honeycomb core, mm [inch]	8.00 [0.314]
Thickness of sandwich panel, mm [inch]	9.30 [0.366]
Width of the panel, mm [inch]	203.20 [8.00]
Longitudinal tensile strength (σ_L), MPa [ksi]	
Average, MPa [ksi]	489.00 [70.92]
Standard deviation, MPa [ksi]	31.07 [4.51]
COV (%)	6.40
Longitudinal elastic modulus (E_L)	
Average, GPa [Msi]	37.80 [5.48]
Standard deviation GPa [Msi]	1.12 [162.44]
COV (%)	2.90

*Results are based on test coupons; COV = coefficient of variation.

[1] USA Patent Pending No. 60-205,609.

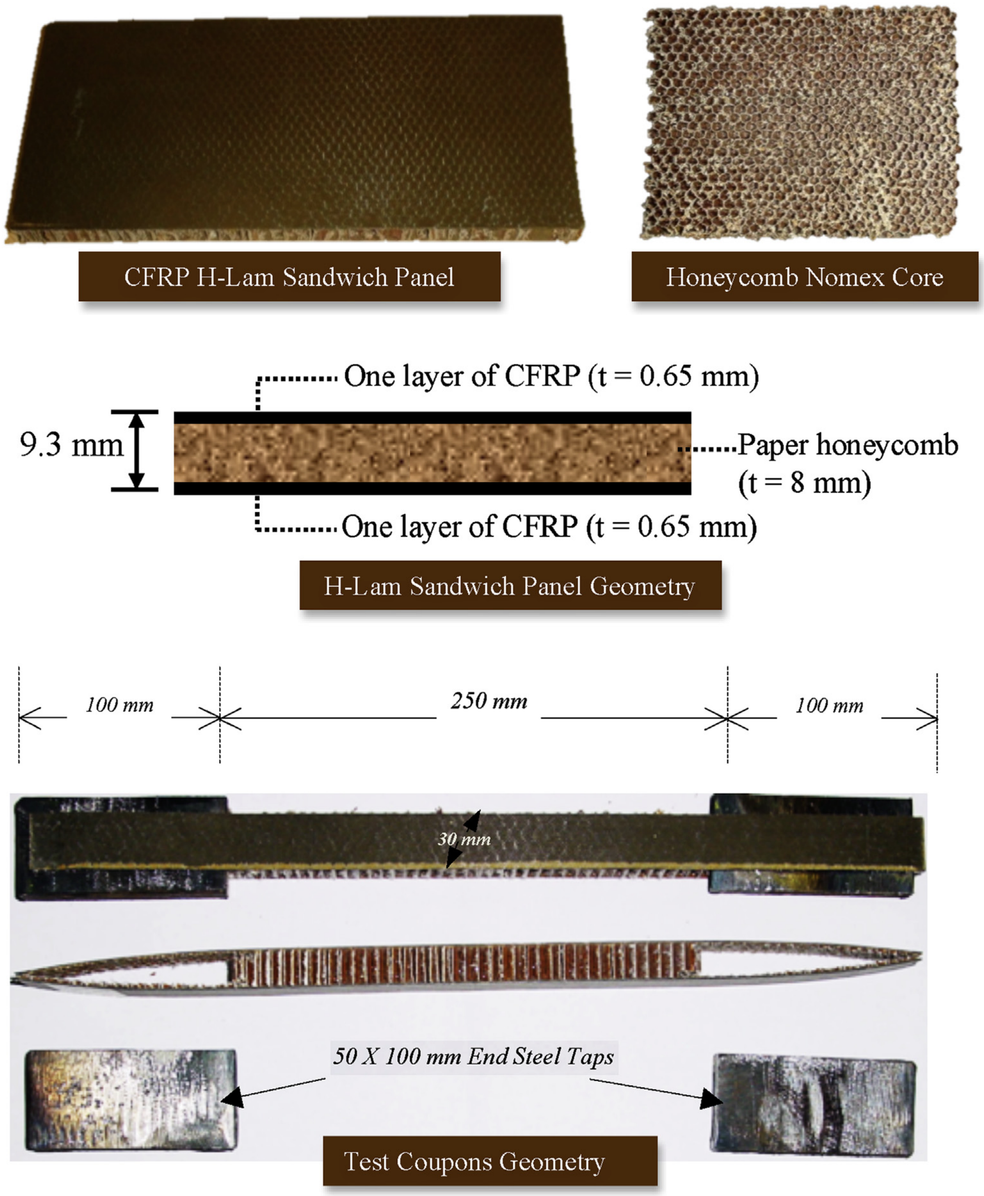


Fig. 3. Geometrical and mechanical properties of cfrp sandwich composite h-lam panels with low-density honeycomb nomex core used for Group "***B***" specimens.

the H-Lam system was 1.0 that according to the NDS requires no lateral bracing. However, the results of current experimental work in progress for wood beams with larger d/b up to 6.0 showed no signed of lateral buckling at failure when the compression edge was laterally restrained.

4. Description of experimental program

As stated earlier, two groups of large scale tests were evaluated in this study, namely: (i) "flexure-shear" strengthening protocol for beams repaired and retrofitted with cross-ply carbon/epoxy wet layup applied in the form of U-shaped laminates that is bonded around three sides of the beam cross-section (*designated hereafter as Group A*), and (ii) "flexure-only" strengthening protocol for beams strengthened with unidirectional carbon/epoxy wet layup and pre-cured honeycomb sandwich flat panels applied at the tension side along the length of the wood beam specimens (*designated hereafter as Group B*). The typical 4-point loading setup and beam specimens' dimensions and geometry are presented in Fig. 5. The unidirectional mechanical properties of the carbon/epoxy wet layup materials, used in this study, were tested in accordance to ASTM 3039 standards (refer to Table 2).

4.1. Group "A" (flexure-shear strengthening scheme): beams strengthened with U-shaped cross-ply [0°/90°]$_{2s}$ composites

For practical reasons, composite laminates were places around three sides of the wood beam. The depth of side laminates (from the beam bottom) 127 mm (5″) which is about two-third of the beam depth. This because in a typical wood building or a bridge, the wood girder carries both the joists and plywood floor, which makes it difficult to place the composite laminate around the girder's entire cross section at all locations along the span without the need of the removal of different elements of the floor system. For this reason, a more practical and conservative case was used in the experimental study.

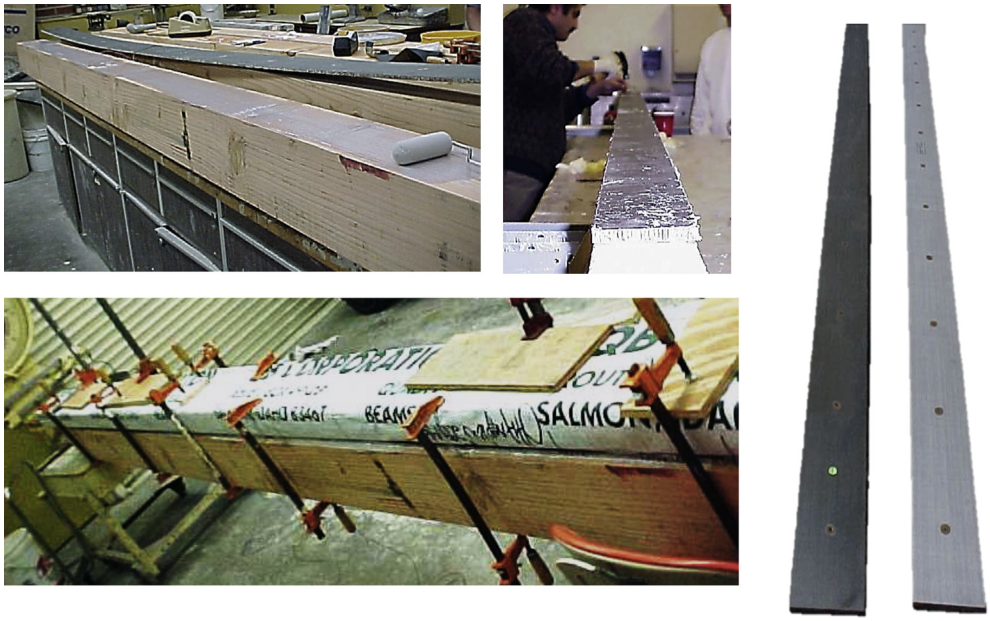


Fig. 4. Application of H-Lam sandwich composites panels on dug-fir beams.

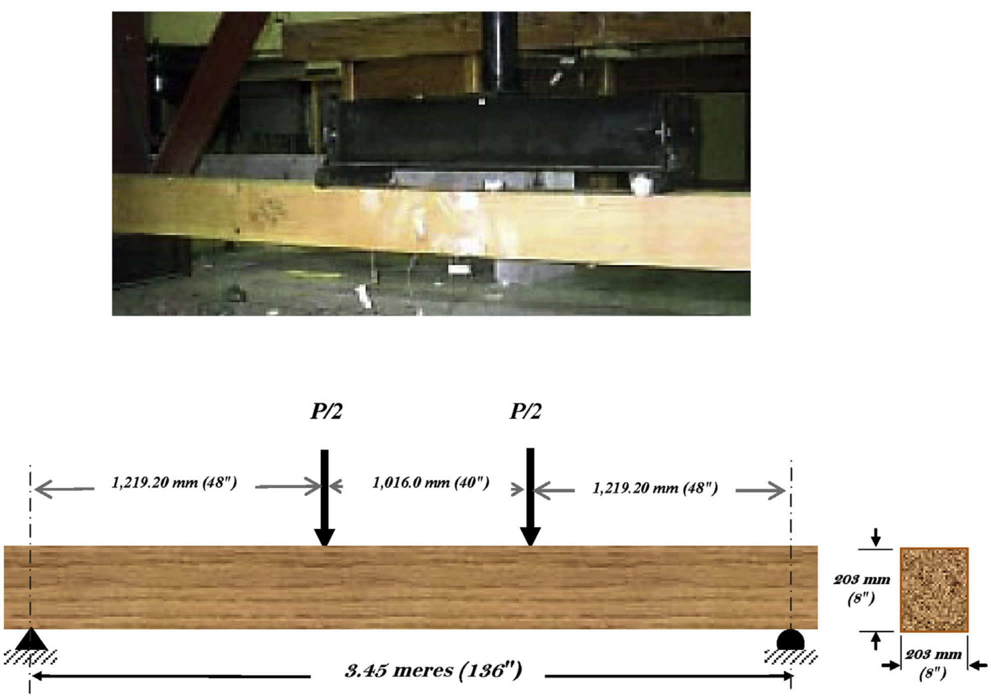


Fig. 5. Wood beams typical dimensions and four-point flexural test setup.

Table 2
Room-temperature mechanical properties of unidirectional carbon/epoxy composites per ASTM 3039.

Coupon sample code	Tensile strength σ_{11}, *MPa [ksi]*	Tensile Modulus E_{11}, *GPa [Msi]*	Rupture strain ε_{11}^{ult} (%)
C1	1241.88 [180.12]	84.8 [12.30]	1.104
C2	1311.59 [190.23]	116.70 [16.92]	1.110
C3	1380.40 [200.21]	109.90 [15.94]	1.102
C4	1350 [195.80]	113.21[16.42]	1.180
C5	1370.06 [198.71]	115.63 [16.77]	1.220
C6	1308.07 [189.72]	129.14 [18.73]	1.280
Average	**1327 [192.47]**	**111.60 [16.16]**	**1.17**

The overall dimensions of Group "A" beam specimens were 20.30 cm (8.0″) X 20.30 cm (8.0″) X 3.45 m (11.33 feet) [*Width* × *Depth* × *Span*] Dug Fir Larch No. 1 wooden beams. All specimens were tested under four-point load regime up to failure (see Fig. 5).

4.1.1. Structural evaluation of as-built (unreinforced) beam test

Initially, two control (as-built) Group "A" specimens were tested under quasi-static loading conditions. The first undamaged, unstrengthened "control" specimen was tested to determine the ultimate moment capacity, flexural stiffness, and failure mode of unreinforced wood. The load was applied via a calibrated 245-kN (55-kip) hydraulic actuator in the form of a linear force-controlled ramp with a loading rate of 8.9 kN/min (2 kips/minute). As the load increased, longitudinal cracks with an average length of 19 mm (¾″) were observed. The first local crack was initiated at a knot located at the bottom side of the Douglas-Fir wood beam below the loading zone (see Fig. 6). The ultimate mode of failure was a combination of interlaminar and tensile failure of wood at the maximum flexural stress region between the two line load application areas. The ultimate load was 31.36 kN (7.05 kips) with a maximum mid-span vertical deflection of 38.00 mm (1.50″). As shown in Fig. 7, this wood beam specimen exhibited linear behavior until a load level of about 22 kN (5 kips), after which the behavior became non-linear up to the failure load. The ultimate load of this control beam specimen was 37.67 kN [8.47 kips] with a corresponding deflection of 118.0 mm (4.65″).

4.1.2. Structural evaluation of pre-damaged (pre-cracked), unstrengthened beam specimens

Wood materials are very susceptible to degradation due to several factors such as fungal attack that occurs due to continuous exposure of moisture and temperature over time. Other potential damages also includes impact and fire which will be discussed in Section 5 of this paper. In order to simulate a scenario of sever loss of a wood member of about one-third of the cross section, cracks extended to one-third of the depth were introduced. In this case, only 66% of the section modulus of the wood flexural member will carry the maximum applied moment. In order to demonstrate the effectiveness of the composite system in restoring damage wood members, a "pre-damaged" specimen was subjected to a similar loading regime. The damage was simulated by 3.18 mm (1/8″) thick-blade saw cuts spaced at 127 mm (5″) at the constant moment area (between the two line loads) with a constant cut depth of 69.85 mm (2.75″) as shown in Fig. 8. Strain gages were bonded to sides and the top surface of the wooden member at several locations. Same data reading schedule and loading rate

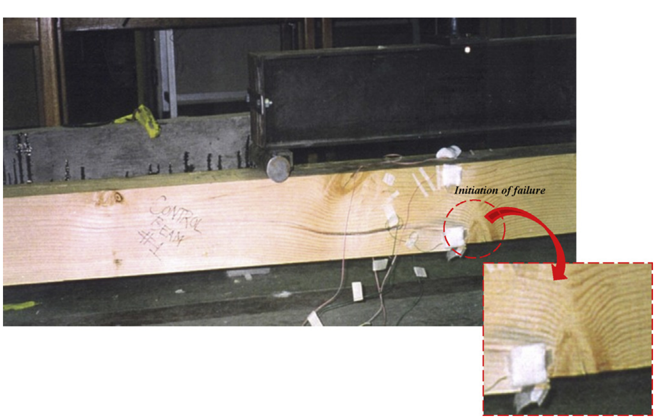


Fig. 6. Failure initiated at a knot located at the bottom side of the mid-span loading zone of the as-built control wood specimen (Group "**A**").

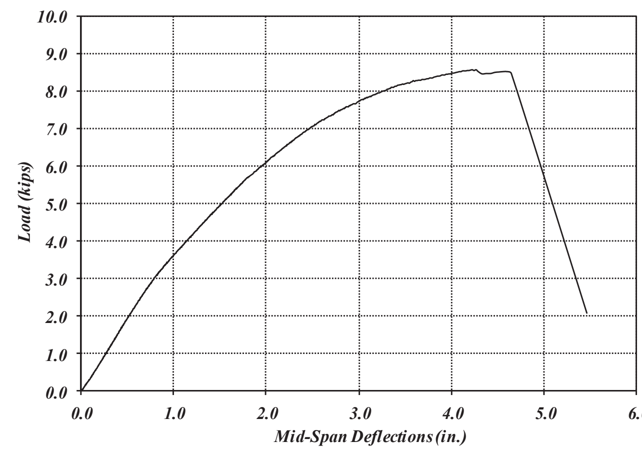


Fig. 7. Load/deflection relation for control (*undamaged*) dug fir wood specimen *[1 inch-25.4 mm, 1 kip= 4.448 kN]*.

were used similar to those for the control specimen. As the load was applied, large deflection was observed. This was expected due to the fact that by slotting the bottom area under the loading points (*maximum constant moment zone*), the effective resisting depth was about 65% of the corresponding depth in the control test. The deflection continued to increase and longitudinal cracks propagated in a faster rate. The ultimate failure load was 31.49 kN (7.08 kips) with a corresponding deflection of 39.37 mm (1.55″). Fig. 9 shows the ultimate failure of the pre-cracked specimen. The load/deflection relation is presented in Fig. 10. As shown in this figure, the behavior was linear up to a load of approximately 13.34 kN (3 kips), after which the behavior exhibited non-linearity with a rapid rate up to failure. One interesting observation is that the ultimate load of the pre-cracked specimen was higher that the ultimate load of the control specimen. This can be attributed to the variation of the wood materials, locations and number of knots (*failure of the control specimen started at a knot location as shown in* Fig. 6).

4.1.3. Structural evaluation of repaired, pre-cracked beam

This test was conducted on another pre-damaged specimen that was repaired with composites. First, the cracks were filed with epoxy (*to about half crack depth*), and then left to cure. The surface was then prepared for applying the primer and the composite laminates. The repair system was composed of bidirectional woven carbon fabrics and high-strength/high-toughness epoxy system $[0^\circ/90^\circ]_{2s}$. The carbon/epoxy laminates were applied to the tension side of the wood member and the sides were extended to cover 75% of the pre-cracked specimen depth (from the bottom). The reason of using this design was based on the observations of crack propagation and ultimate failure mode observed in the control test. The longitudinal fibers were designed to carry the flexural tensile stresses, while the vertical fibers were designed to resist the interlaminar shear stresses. Electrical strain gages were bonded to wood and composite surfaces at different critical locations and data were collected automatically using a computerized data acquisition system as for the other two specimens. The behavior of this specimen was linear up to about 22.25 kN (5.00 kips), after which a slight non-linearity was observed (refer to Fig. 11). It is important to point out that this specimen *did not fail*, since the test was halted due to the hydraulic actuator stroke limitation. The maximum load recorded in this test was 82.5 kN (18.57 kips) with a corresponding mid-span deflection of 77.22 mm (3.04″). The ultimate tensile stress at the mid-span was 36.757 MPa (5331 psi). As compared to

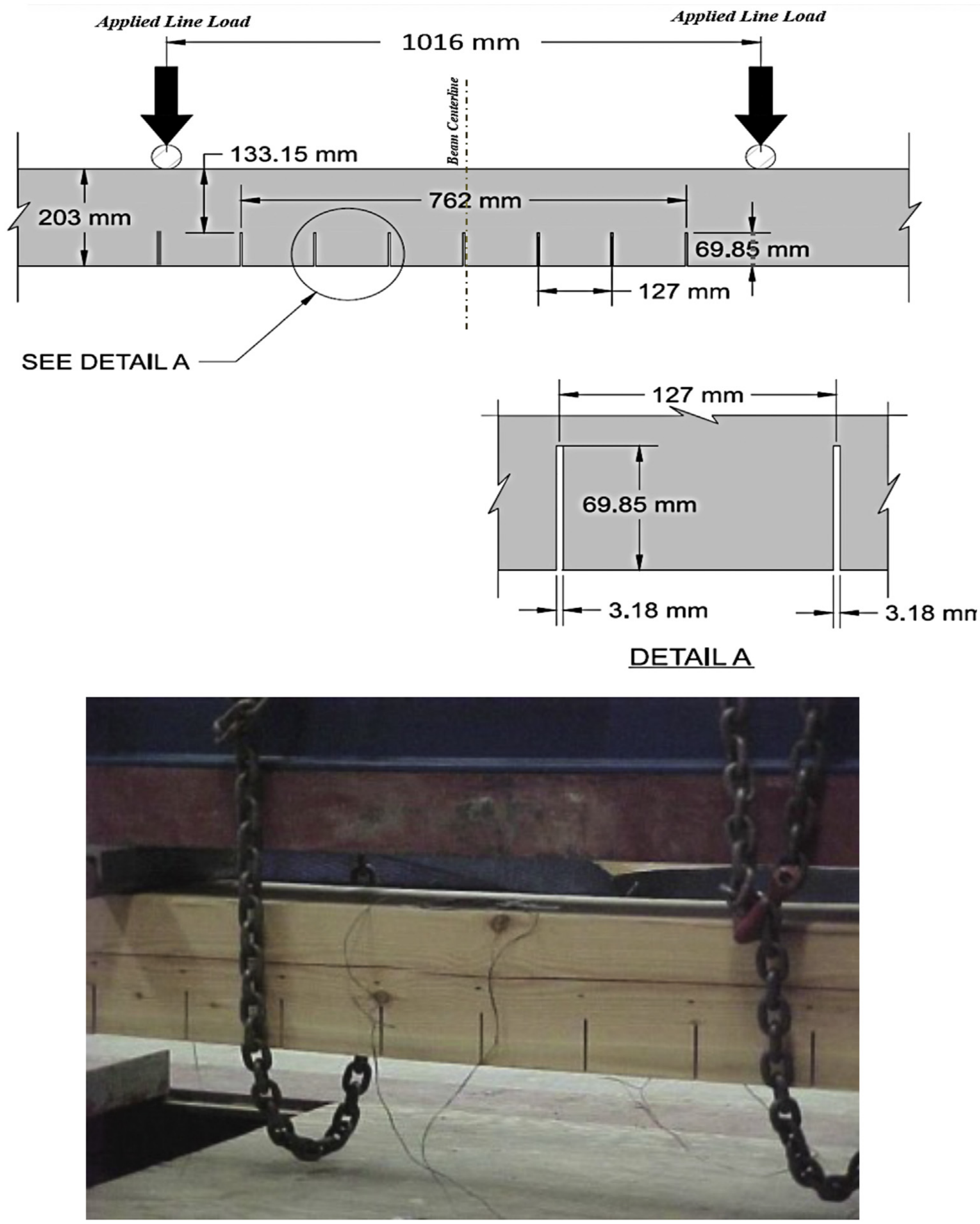


Fig. 8. Mid-span cracks details of the pre-damaged wood beam of Group "**A**".

the strength of the pre-cracked specimen (σ_u = 16.77 MPa/2432 psi), and the control specimen (σ_u = 14.01 MPa/2032 psi) an increase in the ultimate strength of about 120% and 162%, respectively was achieved by adding the composite laminates.

4.1.4. Structural evaluation of retrofitted, undamaged beam

Identical U-shaped bidirectional lamination schedule and application procedure were adopted for externally reinforcing the undamaged wood specimens with carbon/epoxy composites. Electrical strain gages were bonded at both the wood and composite surfaces. The specimen was tested under the same load setup as for the other three specimens. This specimen exhibited stiffer behavior as compared to the other three specimens, and had a bilinear behavior. The first linear part was maintained up to a load level of 57.82 kN (13 kips), while the second near linear stiffness was observed until a load of 102.3 kN (23 kips), after which the yield was achieved and maintained until failure occurred (see

Fig. 9. Ultimate failure of pre-cracked beam specimen.

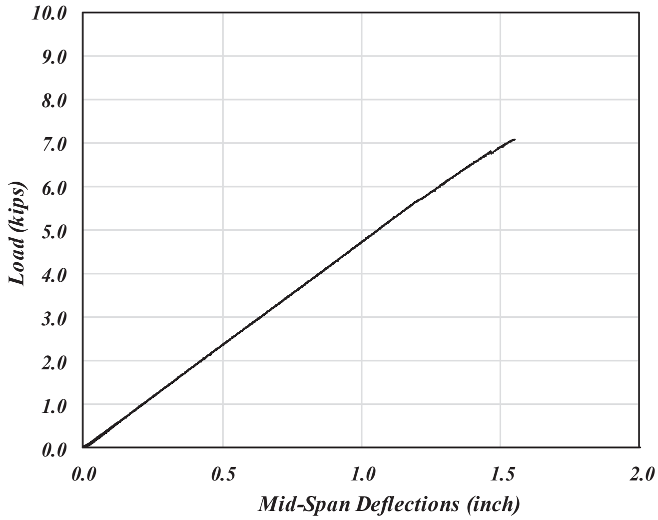


Fig. 10. Load/deflection relation for pre-cracked wood specimen (Group "**A**") *[1 inch-25.4 mm, 1 kip = 4.448 kN].*

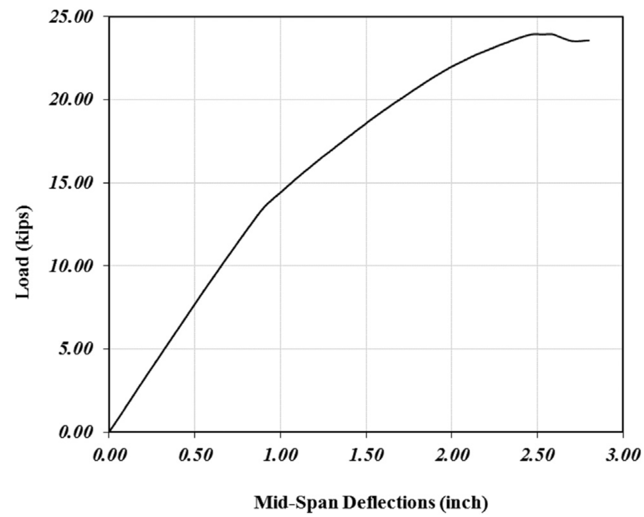


Fig. 12. Load/deflection relation for pre-cracked CFRP repaired wood specimen *[1 inch-25.4 mm, 1 kip = 4.448 kN].*

Fig. 12). The associated failure mode was less sudden and more ductile as compared to the one observed for the control specimen. The ultimate failure mode was initiated by a cohesive failure of the wood at the specimen side exactly under the left point load, followed by a local of the composite laminate at the same location as shown in Fig. 13. The stress/strain curves for pre-cracked, repaired, and the retrofitted wood members are presented in Fig. 14. Based on the test results, the gain in the strength of the retrofitted members as compared to the control members is in the order of 238.27%.

4.2. Group "B" (flexure-only strengthening scheme): beams strengthened with flat unidirectional [0°]$_2$ wet layup and sandwich honeycomb composites

The overall dimensions of Group "B" beam specimens were 20.3 cm (8 ″) × 20.3 cm (8 ″) × 3.70 m (12′) *(Width × Depth × Span)* Douglas Fir Larch No. 1 wooden beams. As one can notice, the span of this group is slightly longer than those of Group "A" described earlier; however, similar geometrical dimensions, wood species and test setup were used. In all cases, the composite laminates were adhesively bonded ton the tension side (bottom) only covering the entire clear span of beam specimens, with primary fibers along the beam longitudinal axis. All specimens were tested under four-point load regime up to failure.

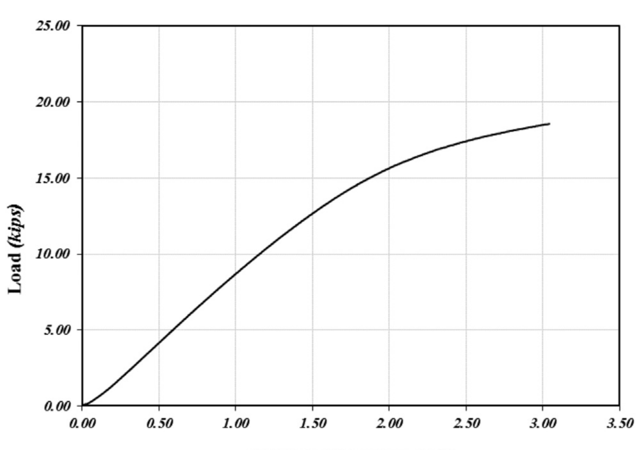


Fig. 11. Load/deflection relation for pre-cracked cfrp repaired wood specimen *[1 inch-25.4 mm, 1 kip = 4.448 kN].*

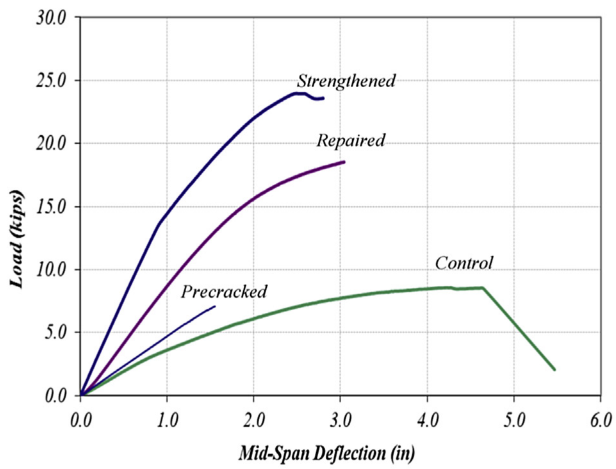
4.2.1. Structural evaluation of as-built (unreinforced) beam test

The general mode of failure of the "as-built" control specimen was brittle with an average ultimate load of approximately 31.4 kN (7.05 kips) with an associated mid-span vertical deflection of about 38.00 mm (1.50″). Near the failure load, major cracks appeared on the sides and the bottom tension side beam with a crack width ranged from 12 mm to 25 mm (1/2″–1.0″). As shown in Fig. 15-a, the major horizontal shear crack that triggered the failure was initiated at a knot location as occurred in Group "A" control specimens.

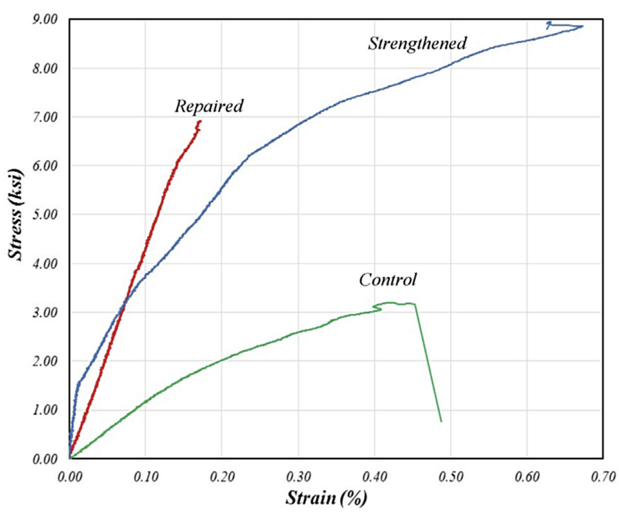
4.2.2. Structural evaluation of wooden beams strengthened with flat unidirectional CFRP laminate [0]$_2$

In this evaluation test, undamaged Dug-Fir beam specimens were laminated with two plies of unidirectional carbon/epoxy composites (see Table 2). The tension (bottom) surface of the beam was roughened with sand papers, and debris and dust were removed via a brush and compressed air. A low-viscosity two-part room-temperature cure epoxy primer was then applied by a roller

Fig. 13. Failure of Group "**A**" retrofitted beam specimen.



(a)



(b)

[1 inch- 25.4 mm, 1 kip= 4.448 kN, 1 ksi=6894.76 kPa]

Fig. 14. (a) Load/deflection and (b) stress/strain behaviors of unstrengthened, repaired and retrofitted beam specimens of **Group "A"**.

to cover the entire surface clear span of the beam specimens. The two-part, low-viscosity epoxy primer was allowed to cure, after which the first unidirectional carbon/epoxy laminate was applied, followed by the second final ply. The laminate was allowed to cure at the laboratory room temperature with an average temperature of 24.50 °C (76.10 °F) and an average relative humidity of 62%. As for all beam specimens evaluated in this study, the load was applied in a four-point loading regime up to failure with a constant rate of loading of 8.9 kN/min (2 kips/minute) using a calibrated 250-kN (55-kip) servo-hydraulic actuator. Load, strain and deflection data were collected automatically via a calibrated data acquisition system. As the load increased, excessive cracking and deflection were apparent. Due to the orthotropic nature of wood, longitudinal interlaminar shear cracks along the side of the beam were observed near the ultimate load. This localized mode of failure is expected in the absence of interlaminar shear reinforcements that were provided in Group "***A***" by the vertical wings of the U-shaped bidirectional laminate that prevented this type of local failure. As the interlaminar shear cracks propagated and widen, sudden tensile rupture of the carbon/epoxy laminate at the bottom face of the beam occurred as shown in Fig. 15-b. The ultimate load of this composite strengthening protocol was a 48.00 kN (10.80 kips) with a mid-span deflection of about 38.00 mm (1.80″). The ultimate capacity of strengthened wood beams with flat CFRP unidirectional laminate $[0°]_2$ increased by 53% as compared to the as-built unstrengthened beams.

4.2.3. *Structural evaluation of wooden beams strengthened with sandwich honeycomb panels (H-Lam) with carbon/phenolic cross-ply face sheets $[0°/90°/c]_s$*

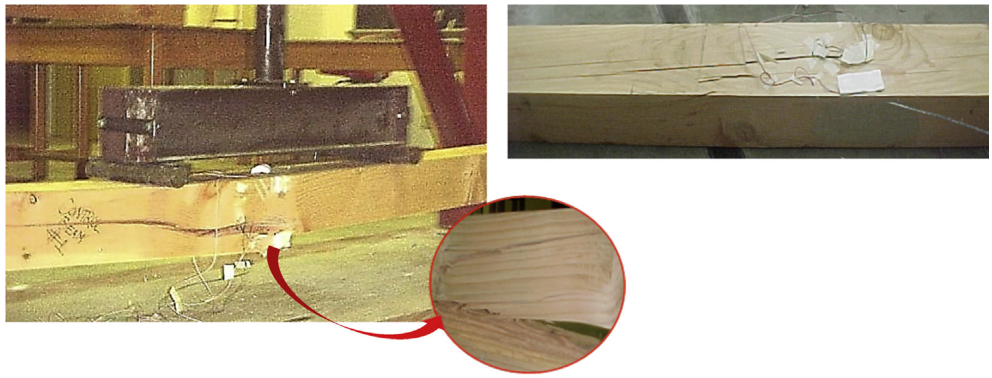
The behavior of this specimen was linear up to about 62.00 kN (14.00 kips), after which, non-linearity was noted (refer to Fig. 16). This specimen performed exceptionally well as compared to other specimens. The ultimate load was about 98.00 kN (22.00 kips), as compare to only 31.00 kN (7.00 kips) ultimate capacity for the as-built unstrengthened beam specimen. This gain in strength can be translated to an increase of over 300% over the strength of the control, as built specimen. The mid-span vertical deflection at the ultimate load was about 76.0 mm (3.0″) as shown in Fig. 16. The failure was initiated by a local interlaminar shear failure close to the left loading point and at about half the depth of the beam (*at the maximum shear load location and at a depth of where maximum shear stresses, near the neutral axis, occur*). This failure propagated rapidly following the path of least resistance, and generated an impulse load, which resulted in a premature shear failure of the H-Lam at near the loading point as shown in Fig. 17. The composite sandwich face laminates and the wood failed as one piece and at the same time (*cohesive failure and not adhesive failure*) as shown in Fig. 18. Fig. 19 shows a sketch describing the failure modes of the wood beams strengthened with "flexure-only" protocol.

5. Summary of experimental results

Load, deflection, and strain data were collected, analyzed and load/deflection (P/δ) and stress/strain (σ-ε) curves were developed and the modes of failure for each beam was identified. The ultimate capacity of the unreinforced wood member was 31.60 kN (7.10 kips) with a mid-span deflection of 38.00 mm (1.5″) at failure. The load carrying capacity of the retrofitted specimen ha increased to 260% and the stiffness was increased by about 90% as compared to the control specimen. The associated failure mode was sudden and was initiated by a cohesive failure of the wood at the specimen side exactly under the left point load. Instantaneously, a local buckling of the side composite thin-walled bidirectional laminate occurred as shown in Fig. 13. Experimental results also indicated that adding external plies of polymer composites to the damaged "pre-cracked" specimen resulted in an increase in the loading carrying capacity up to 180%, with a stiffness upgrade to about 150%. Table 3 presents a summary of the experimental results of Group "***A***" unstrengthened and strengthened wood beams.

In addition, large-scale experimental results indicated that beam specimens retrofitted and repaired with carbon/epoxy laminates exhibited higher toughness. A comparison between the toughness of the different specimens is presented in Fig. 21. From this figure, one can notice the appreciable toughness enhancement of the retrofitted and repaired specimens as compared to the unstrengthened and pre-cracked specimens.

For Group "***B***" beams (flexure-only) strengthened with flat unidirectional wet layup and sandwich honeycomb composites, similar results were achieved. Based on load/deflection curves for the as-built, unidirectional and the H-Lam strengthened beam specimens that were presented in Fig. 16, one can see that distinct high strength, stiffness of the wood beam strengthened with the innovative H-Lam sandwich system. In addition, it is clear from this



(a)

(b)

Fig. 15. Ultimate failure modes for Group "**B**" specimens: (a) unstrengthened beam, (b) beam strengthened with flat unidirectional carbon/epoxy laminates.

figure and Fig. 20 that the toughness of the wood beams strengthened with H-Lam was about four times of the unidirectional flat laminate strengthened specimens and seven times the as-built (*unstrengthened*) beam. A comparison of flexural capacity of as-built and the two FRP strengthened beams is presented in Fig. 21. As shown in this figure, the capacity of the H-Lam strengthened wood beam is over three times the as-built specimen and twice as much as the capacity of the wood beam strengthened with unidirectional flat laminate. The failure of the H-Lam strengthened beam was more ductile that both the flat unidirectional laminate and the as-built specimen. In particular, cohesive failure was observed in the case of the H-Lam specimen where chunk of wood was attached to the top face sheet at the failure. In contrast, the failure of the beam with unidirectional flat laminate was a predominantly in the form of adhesive failure.

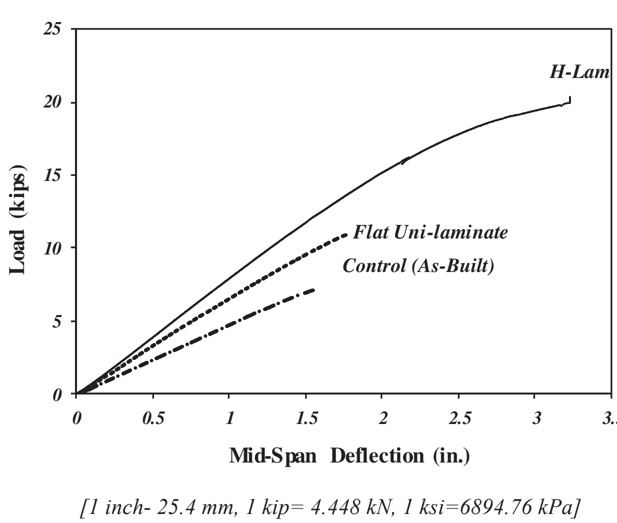


Fig. 16. Load/deflection behavior for unstrengthened (as-built), beams retrofitted with flat unidirectional Laminate and beams retrofitted with H-Lam sandwich panel **Group "B"**.

6. Analytical design procedure: *a case study*

This section presents a practical numerical example for the repair of a partially-damaged glulam wood girder using sandwich composite system evaluated in this study. The example demonstrates analytical procedure for predicting flexural capacity of the repaired glulam girder.

6.1. Background information

An over-hanged Glued Laminated (Glulam) timber girder was a part of the roof structural system of a commercial facility in southern California that was exposed to a fire event. Some portions of few glulam girders were partially due to exposure to direct flames (see Fig. 22).

The typical glulam girders analyzed in this case study has a simply supported span of 21.64 m (71.00′) with an overhang cantilever span of 2.14 m (7.00′) as shown in Fig. 23. The figure also shows the reduced section of the Glue-Lam girder based on visual

Fig. 17. Ultimate failure of the wood beam strengthened with honeycomb sandwich panel (H-Lam).

Fig. 18. Cohesive failure of beams retrofitted with sandwich composite laminates indicating proper interfacial bond between wood and composites.

inspection at the site. As shown in Fig. 24, the reduced section is 79.375 cm × 15.875 cm (31 ¼″ × 6 ¼”). The original dimensions before fire damage were 80.01 cm × 17.145 cm (31 ½″ × 6 ¾”).

6.1.1. Reference code documents

The following design documents were consulted in performing this analysis. It should be noted that the use of older editions of the code was necessary for assessing this existing old building, especially for the mechanical and physical properties of the Glulam that is uncommon today. The design documents are; National Design Specification (NDS) for Wood Construction ANSI/AWC NDS-2012 [20], National Design Specification (NDS) Supplement. Design values for wood construction [21], and National Design Specification (NDS) for Wood Construction e Commentary [22], ANSI/NfoPA NDS-2005.

6.1.2. Data & assumptions

The following are the information on the existing Glulam girder mechanical, physical and geometrical properties as well as the mechanical properties of the composite system used in this case study.

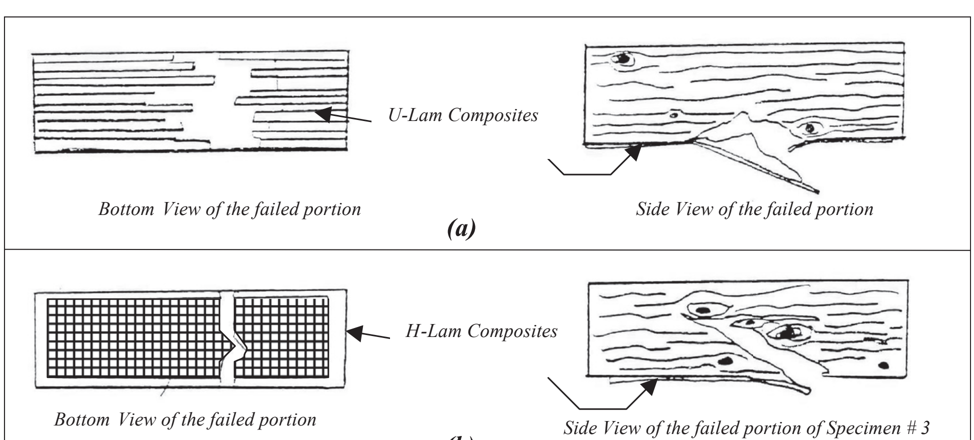


Fig. 19. A sketch illustrating failure modes of wood beams strengthened with (a) unidirectional flat laminate, and (b) H-Lam sandwich panel.

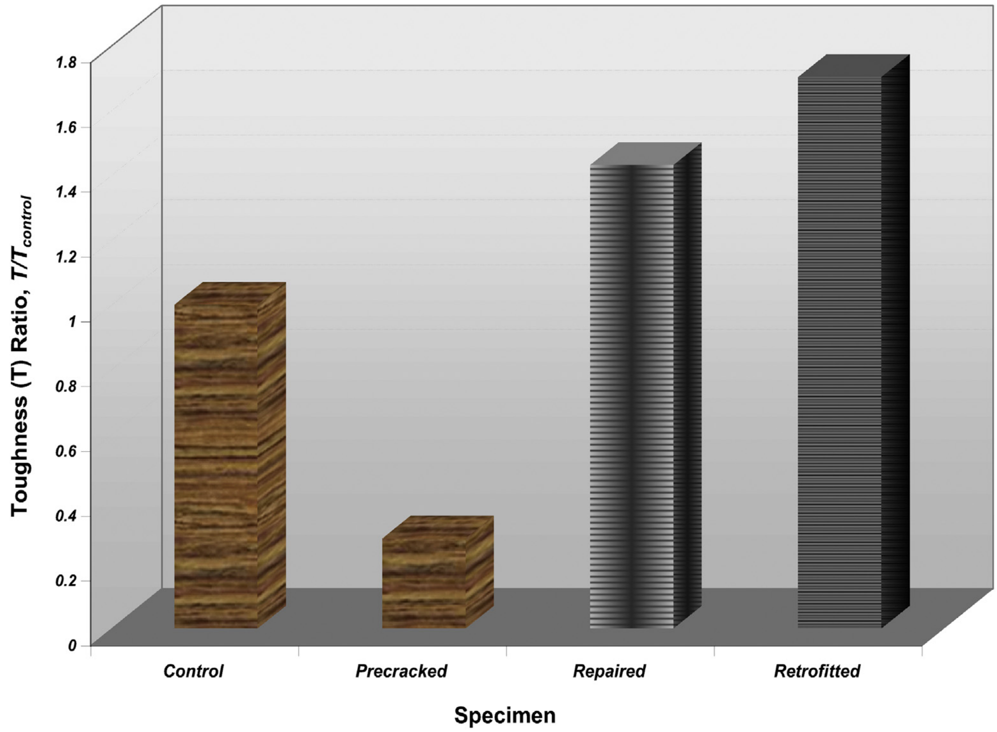


Fig. 20. Toughness comparison relative to control "unstrengthened" wood beam specimen.

Table 3
Experimental summary results for **group "A"** specimens.

Specimen	Ultimate load, *kN [kips]*	Flexural strength, *MPa [psi]*	Max mid-span deflection, *mm [inch]*	Flexural strength increase *(%)*
Control	37.67 [8.47]	16.76 [2432]	118.16 [4.65]	–
Pre-cracked	31.49 [7.08]	14.01 [2032]	39.37 [1.55]	–
Repaired	82.46[a] [18.54[a]]	36.76 [5331]	[77.21] 3.04	119[b]
Retrofitted	106.53 [23.95]	47.41 [6.876]	71.12 [2.80]	238[c]

[a] Specimen did not fail.
[b] As compared to the ultimate strength of the pre-cracked specimen.
[c] As compared to the strength of control "undamaged" specimen.

6.1.2.1. Glue-lam information.
Geometry:

- 24F–V8 Glue-laminated Girder
- Dimensions and Girder Spacing: 80.01 cm × 17.145 cm (31 ½″ × 6 ¾") @ 2.45 m (8′) on center (o.c.),
- Area (A) = 1371.61 cm^2 (212.6 in^2), I_x = 731734.85 cm^4 (17,580 in^4), S_x = 18288 cm^3 (1116 in^3), r_x = 23.096 cm (9.093″), I_y = 33602.36 cm^4 (807.3 in^4), S_y = 3919.79 cm^3 (239.2 in^3) [Source: Table 1C –NDS Supplement (2005)].

  Estimated "As-Built" Mechanical Properties (No Adjustment):

- F_{bxx} (*tension zone*) = 16547.42 MPa (2400 psi),
- F_{bxx} (*compression zone*) = 16547.42 MPa (2400 psi),
- F_{vxx} (*shear parallel to grain*)* = 1137.63 MPa (165 psi)
- E_{xx} = 11.721 GPa (1.7 Msi).

* *(Source: Table 5A – [22] for older quality Glue-Lam)*

Table 4
Geometrical and mechanical properties of FRP sandwich composite (H-Lam) panels used in case study numerical example (*Aluminum Honeycomb Core*).

Thickness of sandwich Panel, mm [inch]	12.70 [0.50]
Width of the panel, mm [inch]	158.75 [6.25]
Longitudinal tensile strength (F_{tu}), MPa [ksi]	1655 [240]
Longitudinal elastic modulus (E_{11}), GPa [Msi]	124.105 [18.00]
Transversal elastic modulus (E_{22}), GPa [Msi]	6.85 [0.98]
Strain at rupture (%)	1.20

6.1.2.2. H-Lam properties (refer to Table 4)

- Width = 17.145 cm (6 ¾″)
- Thickness = 12.7 mm (½″)
- Core Material: Aluminum
- Composite Laminates: 0°
- Ultimate Tensile Strength, F_{tu} = 1655 MPa (240.00 ksi)
- Longitudinal Modulus of Elasticity, E_{11} = 124.105 GPa (18.00 Msi)
- Strain at Rupture = 1.2%.

6.1.2.3. Geometrical information of the structure. Fig. 23 describes the statically system, idealized boundary conditions and geometry of the Glue-Lam Girder under consideration.

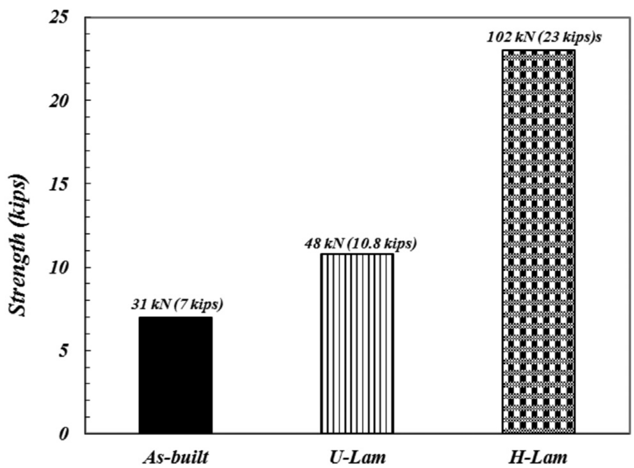


Fig. 21. Flexural strength comparison between for unstrengthened (as-built), beams retrofitted with flat unidirectional laminate and beams retrofitted with H-Lam sandwich panel **Group "B".**

6.1.2.4. Mechanical loading & environmental conditions. The following are the mechanical and environmental conditions that were considered in this analysis.

- Maximum Positive Moment = 509.472 kN-m (375,767 *lb-ft*),
- Maximum Negative Moment = 20.728 kN-m (15,288 *lb-ft*),
- Moment at the Middle of the Charred Portion = 388.084 kN-m (286,236 *lb-ft*),
- Highest Moment (Positive) at the Charred Portion = 463.44 kN-m (341,816 *lb-ft*),
- Lowest Moment (Positive) at the Charred Portion = 282.270 kN-m (208,192 *lb-ft*),
- Maximum Shear = 97.576 kN (21,936 pounds) at left support (point A in Fig. 9),
- Maximum Shear at the Charred Portion = 66.189 kN (14,880 pounds),
- Moisture Content is estimated to be in the range of 6%–16%.

6.1.2.5. Reduced section at the charred portion. Fig. 24 shows the assumed reduced section of the Glue-Lam girder based on visual inspection at the site. As shown the reduced section is 79.375 cm × 15.875 cm (31 ¼″ × 6 ¼″). The original dimensions before fire damage were 80.01 cm × 17.145 cm (31 ½″ × 6 ¾″).

The reduced section modulus is 16,670.6 cm^3 (1017.3 in^3) vs. 18,292.55 cm^3 (1116.28 in^3) for the original undamaged portion of the girder, which constitutes about 9% reduction in the section modulus. Consequently, the reduction in the moment capacity is about 9% at the charred damaged portion. Thus, the section at the charred section has a reduced capacity of only 91% of the designed moment (*demand*) of 463.44 kN-m (341,816 *lb-ft)*, i.e., only 421.73 kN-m (311,052.6 *lb-ft)* with a deficiency of 41.71 kN-m (30,763.5 *lb-ft*). However, each original "undamaged" glulam girder was designed for the maximum moment of 509.47 kN-m (375,767 *lb-ft*) at a distance of 16.20 m (53.00′) from the left support as shown in Fig. 23 at point "***B***". So, the deficiency relative to the original design moment is 45.85 kN-m (33,820 lb-ft). The size factor, C_F, for depth >30.04 cm/12″ (in our case the depth is 79.375 cm/31¼″) is taken as 0.90. It can also be calculated using the following equation:

$$C_F = \left(\frac{12}{d}\right)^{\frac{1}{9}} = 0.89 \cong 0.90$$

6.1.3. Structural upgrade & repair analysis

As mentioned earlier, the reduction in the load carrying capacity at the charred portion of the glue-lam should be restored not only for flexure, but also for shear parallel to the grain (*Interlaminar shear strength*). The following are the design procedures and the analysis for both the H-Lam sandwich system (*for flexure upgrade*) and the carbon/epoxy wet layup laminate (*referred to herein as W-Lam*) for interlaminar shear capacity upgrade.

6.1.4. Flexural upgrade using H-Lam sandwich system

In this analysis, the concept of transformed section is used, coupled with the imposing linear strain compatibility conditions of both the wood materials, and the polymer carbon/phenolic composite H-Lam material. Now, using the information in Sections 6.1.2.1 and 6.1.2.2, and knowing the longitudinal moduli of elasticity of both the glulam species [NDS Table 5A] and the H-Lam sandwich panel, one can calculate the modular ratio (n):

Fig. 22. Fire damages of the Glulam girder (case study).

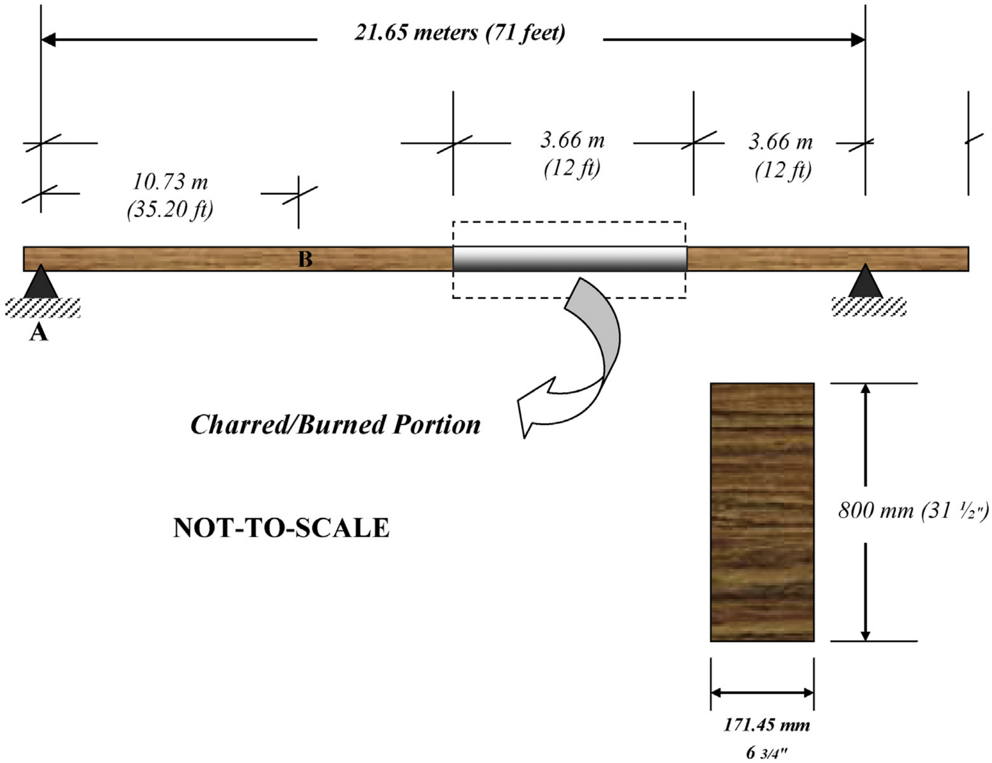


Fig. 23. Dimensions and idealized support conditions for the Glulam girder.

$$E_1 = E_{x(wood)} = 11.72 \text{ GPa } (11,700 \text{ ksi}), \text{ and}$$

$$E_2 = E_{H-lam} = 124.106 \text{ GPa } (18,000 \text{ ksi}), \text{ thus :}$$

$$n = \frac{E_2}{E_1} = \frac{18,000^{ksi}}{1,700^{ksi}} = 10.60$$

The equivalent width of the H-Lam panel can now be calculated by multiplying the original width of the H-Lam by the modular ratio (n):

$$b_{(equivalent)} = n\, b_{(original)} = 10.60 \times 6.25'' = 66.25'' \text{ (168.27 cm)}$$

The location of the neutral axis (N.A.) can be calculated using the equivalent width as follows:

$$h_1 = \frac{\sum y_i A_i}{\sum A_i} = \frac{31.25 \times 6.25 \times 15.625 + 0.022 \times 66.25 \times 31.26 + 0.022 \times 66.25 \times 31.739}{31.25 \times 6.25 + 2 \times 0.022 \times 66.25} = 15.86''(40.28 \text{ cm})$$

Knowing that the ultimate strain of wood is about 0.005 (0.50%), which is considered herein as the limiting state, in order to satisfy the strain compatibility. Based on this assumption, the maximum usable strain at the composite most bottom composite face of the H-Lam can now be calculated using the linear symmetry of the strain diagram shown in Fig. 25. Thus, the maximum effective strain of the H-Lam (ε_{eff}) is calculated as:

$$\frac{\varepsilon_{eff}}{15.89''} = \frac{0.005}{15.39''}$$

from which;

$$\varepsilon_{eff} = 0.00516 \ (0.516\%)$$

Accordingly, the corresponding stress resisted by the H-Lam bottom laminate (F_{ult}^{h-lam}) is calculated as:

$$F_{ult}^{h-lam} = E_{h-lam} \times \varepsilon_{eff} = 18\ Msi \times 0.00516$$
$$= 92,880\ psi\ (640.385 MPa)$$

To calculate the flexural stresses at different levels, the transformed moment of inertia is calculated for the transformed section shown in Fig. 25-b as follows:

$$I_{(transformed)} = 18,462 \text{ in}^4 \left(768,446 \text{ cm}^4\right)$$

Now, one can calculate the flexural stresses at different sections:

i. *Compression flexural stress at the top face of Glue-lam:*

$$\frac{375,767\ lb-ft(12)\times 15.86''}{18,462\ in^4}=3,874\ \ psi\ (26.71\ \ MPa)[Point\ 1\ in\ Figure\ (25-d)]$$

ii. *Tensile flexural stress at the bottom face of Glue-lam (at the interface with H-Lam)*

$$\frac{375,767\ lb-ft\ (12)\times 15.39''}{18,462\ in^4}=3,759\ psi\ (25.92\ \ MPa)[Point\ 2\ in\ Figure\ (25-d)]$$

iii. *Tensile flexural stress at the top face of H-Lam (at the FRP/wood interface):*

$$\frac{375,767\ lb-ft(12)\times 15.39''}{18,462\ in^4}\times 10.6=39,845\ psi\ (274.72\ MPa)<92,880\ psi\ (640.39\ MPa)[Point\ 3\ in\ Figure(25-d)]$$

iv. *Tensile flexural stress at the bottom face of H-Lam:*

[(*Point 4 in* Fig. 25-d)]

The addition of the H-Lam, not only will restore the original capacity, but it upgrades the glue-lam to flexural capacity of 1219.30 kN-m (899,281 *lb-ft*) which is about 200% additional load carrying capacity. This can be calculated, by setting the maximum stress at the bottom face to 640.39 MPa (92,880 psi).

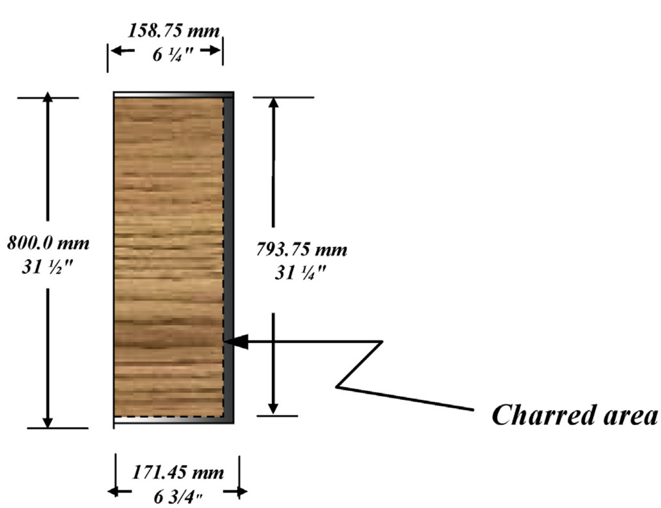


Fig. 24. Dimensions of the reduced section at the charred portion of the Glulam girder based on site inspection.

6.1.5. Interlaminar shear (parallel-to-grain) strength upgrade using carbon/epoxy wet layup (W-Lam) system

At the charred portion of the Glue-lam girder, it is anticipated that the exposure to the high temperature resulting from fire had an adverse effect on the bond-line (glue-line) between the wood laminates. For this reason, and in order to able the girder to transfer the flexural load through the depth of the glue-lam, it is critical to upgrade the interlaminar (*horizontal*) shear strength parallel to the grain (*or to the glue-lines*). This can be accomplished by adding high-strength unidirectional carbon/epoxy laminates bonded to each side of the girder covering 2/3 of the girder's depth which is 53.40 cm (21.00″) from the bottom side. Fig. 26 shows the details of the composite repair system used in this case study.

From the structural analysis described earlier, the maximum shear demand is 97.576 kN (21,936 pounds) at the girder's left support (*point **A** in* Fig. 23), however, at the charred portion, the shear demand is 66.19 kN (14,880 pounds). Now, using the maximum shear value, the maximum shear stress at the centroid is calculated by the following relation:

$$\tau_{\max}=\frac{QV_{\max}}{I_{char}b}$$

where: $Q = 13719.41\ cm^3$ ($837.21\ in^3$), $I_{(char)} = 661558\ cm^4$ (15,894 in^4), $b = 15.88\ cm$ (6 ¼″), and $V_{max.} = 97.576\ kN$ (21,936 *lbs.*), thus:

$$\tau_{\max}=\frac{837.21\times 21,936}{15,894\times 6.25}=184.87\ psi\ (1.27\ MPa)$$

In order to utilize realistic mechanical values for the existing damaged girder, older version of the [22] edition) was consulted. From Table 5A – [22]; F_{vxx} (*shear parallel to grain*) = 1137.64 kPa (165.00 psi). However, and based in the visual inspection and per Table 4A of the 1997 NDS Supplement, it is considered safe to assume that the maximum value of F_v is taken to be 80.00 psi (*as in case Redwood*). Thus, the shear demand of 1.27 MPa (184.84 psi) at the charred portion would not be effective in transferring the applied horizontal shear stresses. For that reason one unidirectional carbon/epoxy ply is applied to the two sides of the girder in

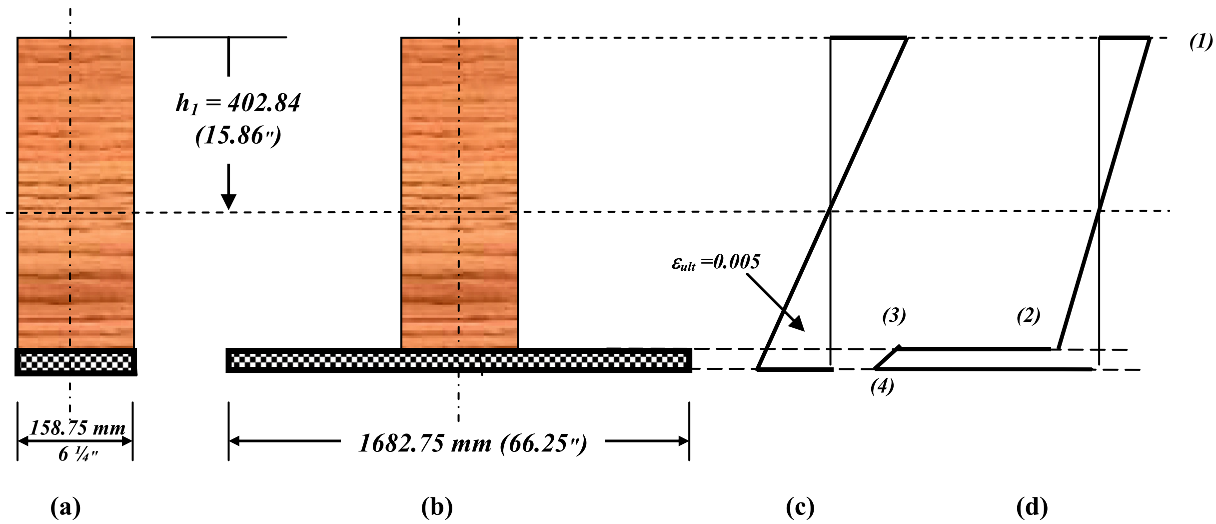


Fig. 25. Transformed section analysis: (a) Original section, (b) Transformed section, strain diagram, and (d) Stress distribution.

order to satisfy this demand. The thickness of the unidirectional laminate is 0.381 mm (0.015″), with an average unidirectional tensile strength of 1.38 MPa (200 ksi). Using a factor of safety (*strength reduction factor*) of 0.75, thus the allowable tensile strength is 1.04 MPa (150 ksi). The shear force resisted by the unidirectional laminate is calculated as follows:

$$V_v^{Uni} = A_{uni} \times F_{alloable}^{uni} = 0.018'' \times 12'' \times 150,000psi = 32,400pounds(144.12kN)[Capacity] > 21,936 \text{ lbs.}(97.58\ kN)[Demand].$$

Thus, a single unidirectional carbon/epoxy laminate [0°] is needed to be applied at each side of the girder with a vertical distance from the bottom side of 53.40 cm (21.00″) to satisfy the shear strength requirement.

The final details of the CFRP H-Lam repair system is shown in Fig. 26.

7. Conclusions

The results of this study revealed that the use of composite external reinforcements contributed to a tremendous increase of both the stiffness and strength of the as-built, undamaged as well as the pre-damaged wood beam specimens evaluated in this study. The highest performance was achieved by the innovative Sandwich honeycomb composite panels (H-Lam), with an increase in the strength over 300% of the as-built wood member. Experimental results indicated that, as expected, the initiation of failure was not due to composite failure nor to failure of the adhesives, rather, the failure for all tested specimens was initiated by the interlaminar shear local failure. For this reason, and based on the results of this full-scale structural evaluation program, it is highly recommended, if not mandated, to use additional wet layup composite laminates (referred to herein as W-Lam) with 90° orientation to the beam longitudinal axis. These 90° laminates are bonded covering at least 2/3 of wood member depth, at both sides of the flexural wood member. This orthogonal lamination detail will further enhance the stiffness and strength performance of such beams that will lead to changing the common brittle failure mode to a more ductile ultimate failure. It is expected that by using the W-Lam in conjunction with the H-Lam system could increase the ultimate capacity by at least 50% in top of the 300% increase resulted from applying the H-Lam system alone that was confirmed in this study. In addition, the results of this study indicated that, adding external composite laminates to the damaged “pre-cracked” specimen resulted in not only restoring the original capacity of the damaged members but also increased both strength and ductility of the wood member. For example, the flexural capacity and the stiffness of the pre-cracked wood members evaluated in this study has increased to 180% and 150%, respectively of the flexural capacity and the stiffness of the unrepaired pre-damaged wood members. That can be translated into major saving in preserving historical and existing constructed facilities that are partially-damaged due long-term environmental exposure, or natural and man-made events such as fire, impact loading, wind and earthquakes.

8. Recommendations & final remarks

This study identified several potential research areas including research on evaluating the effectiveness of the use of polymer composites in enhancing the ductility and strength of wood column members. Also, further studies on the durability, creep and fatigue behavior of this strengthening system are also need to ensure the long-term reliability of this strengthening technique.

One of the major obstacles that may limit the use of this successful application of polymer composites, is the lack for standardized design codes and both national and international specifications. Till today, no national or international document has been published to provide a guide for architects and engineers for using this sustainable technique that save our forests and protect our environment. For concrete applications, for example, The American Concrete Institute (ACI) and the International Code Council (ICC) in USA have developed design guides and acceptance criteria, respectively, for strengthening reinforced concrete structures, yet, no professional engineering organization or even an

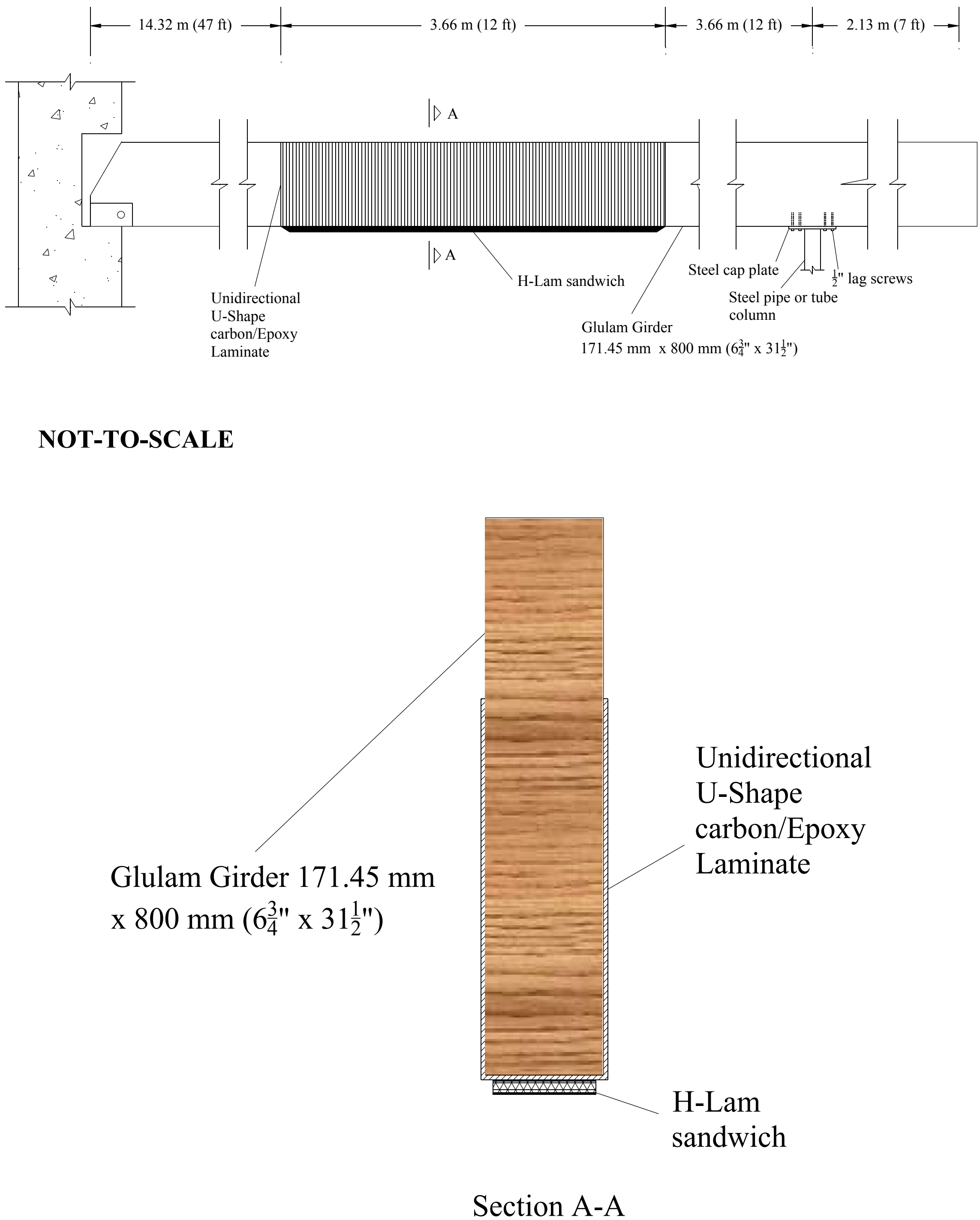


Fig. 26. FRP composite retrofit details of fire partially-damaged glulam girder.

industrial organization has taken the lead to initiate any standard documents for this application. It is highly recommended that a national and international standards be developed to expand and control the use of such successful technique.

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