TIMBER BEAMS POST-TENSIONED WITH HARDWIRE SHEETS

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Abstract: Ever increasing traffic demands and environmental exposure of timber bridges have reduced their performance over the years. The large number of these structures in many jurisdictions makes development of effective strengthening techniques essential for planning the maintenance of these structures. An experimental program was undertaken at the University of Manitoba to determine the effect of post-tensioning on performance of these beams. The emphasis of this work was primarily on the flexural behaviour and on the effect of this technique on the stiffness of the timber beams. A total of six salvaged Douglas-Fir creosote treated timber beams were tested in three point bending till failure. The dimensions of beams were 130 mm wide x 330 mm deep x 4500 mm long. The beams were reinforced with GFRP sheets at an angle of 45˚ in the shear zone to prevent shear failure. The beams were post-tensioned with hardwire sheet on the tension side. Four beams were post-tensioned with medium density hardwire sheets and the other two beams were post-tensioned with high density hardwire sheets. The dimensions of the sheets were 120 wide x 0.74 mm thick x 5000 mm long. The results showed small increase in the stiffness of the post tensioned timber beams on average by 3% and 6%, for the mediums and high density sheets, respectively. The increase in strength was more significant, on average 60%.

1 INTRODUCTION

Many timber bridges in Canada have degraded due to aging, flood damage or increased heavy traffic loads which they were not designed for. These bridges are in need of either rehabilitation or replacement in the near future. It was found that strengthening to extend service life of aging bridges is a more economical option compared to full replacement. New materials and techniques are required to maintain these timber bridges and extend their service life. This paper presents an experimental study of strengthening and stiffening options for aging timber bridges using post-tensioning.

In the last two decades, many studies were performed on strengthening timber beams with fiber reinforced polymers (FRP) [Gentile et al. (2002), Svecova and Eden (2004), Buell and Saadatmanesh (2005), and Yang et al. (2008))]. Triantafillou and Deskovic (1992) tested three specimens of 800 mm long and 45 mm wide specimens in three point bending. Two specimens had a thickness of 60 mm while the third one had a thickness of 80 mm. The beams were reinforced with prestressed thin CFRP sheets consisting of unidirectional carbon fibres at a volume fraction of about 60% bonded together with an epoxy matrix. It was found that the CFRP sheets increased the strength, stiffness and ductility characteristics of the wood. Also the experimental results were verified with accurate analytical models.

Silva-Henriquez et.al (2010) tested 45 glulam beams in four point binding to failure. The beams were 130 mm wide x 305 mm deep x 6.7 m long. A total of 15 beams were prestressed with GFRP laminate (121 mm wide x 3mm thick) on the tension side. Another 15 beams were reinforced with GFRP laminate on the tension side. The remaining 15 beams were control beams without any reinforcement. The results showed that the strength of the prestressed glulam beams was increased by 38% compared with the reinforced GFRP glulam beam and was increased by 95% compared with the control beams. Also the stiffness for both the prestressed and reinforced GFRP beams was increased by 8% compared with control beams.

Walker (2006) tested 58 timber beams with dimensions of 75 mm x 250 mm x 2400 mm. The beams were reinforced using five different reinforcement configurations. Eighteen were used as control beams; 14

beams had 1 GFRP bar; 6 beams had 2 GFRP bars; 16 beams had 1 GFRP bar and dowels; and 14 beams had 2 GFRP bars and dowels. The research showed that the control and strengthened beams sustained an average load of 38.6 kN and 50.8 kN respectively which represents an increase of 31.6%.

2 EXPERIMENTAL PROGRAM

2.1 Materials

2.1.1 Timber

The Ontario Department of Transportation provided a total of 7 creosote-treated Douglas-Fir beams (130 mm x 330 mm x 4500 mm) for this research. These beams were graded using the National Lumber Grades Authority (NLGA 2003).

2.1.2 Hardwire

Hardwire is a family of reinforcements made from ultra-high strength twisted steel wires, as shown in Figure 1. Two different types of hardwire sheets were used for flexural strengthening in the test. Four beams were reinforced using medium high density hardwire ST-12-12, while another two beams were reinforced using high density hardwire ST2-36-12. Table 1 and Table 2 list the material properties of the hardwire sheets. The ST2 hardwire is a high carbon steel cord with a micro-fine brass or AO-brass (adhesion optimized) coating.

Figure1: Hardwire tape

Table 1: Properties of the Cord

Single Roving (Cord) Properties							
Description	Filament Diameter (in)	Cord Diameter. (in)	Break (N)	Break (Lbs)	Break (Kips)	Strain to Failure	Length per lb (ft)
ST2	2@0.012	0.024	445	100	0.1000	2.25%	1328

Table 2: Mechanical Properties of the Hardwire Sheet

Description				Typical Composite Properties			
Density	Hardwire Item Number	Cord Type	Standard Cord Coatings	Laminate Density (lb/ft^3)	Laminate Thickness (in)	Sheet Stress (ksi)	Effective Modulus (msi)
Medium	ST2-12-12	ST2	Brass,AO Brass	101.6	0.029	41.71	3.22
High	ST2-36-12	ST2	Brass,AO Brass	180.8	0.029	125.17	8.61

Tension test of the sheet was conducted at the McQuade Structures Laboratory at the University of Manitoba to determine its ultimate load. The medium high density sheet was tested in accordance with ASTM Standard D198 (ASTM 1999). The test setup and the sheet after failure are shown in Figures 2 and 3 respectively. The results showed that the ultimate load and the strength for the medium density sheet was 25 kN and 283 N/mm^2, respectively.

Figure 2: Experimental Setup for Tension Test

Figure 3: Sample after Failure

2.2 Test Specimens

Six beams out of seven with dimensions of 130 mm x 330 mm x 4500 mm were post-tensioned at the McQuade Structures Laboratory at the University of Manitoba by a local contractor. The remaining beam was a control beam without any reinforcement. Four beams out of the six were post-tensioned using medium density sheet (120 mm wide x 5000 mm long) and GFRP sheets at a 45˚ angle at each end while the other two beams were post-tensioned using high density sheet (120 mm wide x 5000 mm long) and GFRP sheets at a 45˚ angle at each end. To obtain unreinforced stiffness values, these beams were tested initially under a load of 15 kN.

The post-tensioned hardwire sheets were prestressed using stressing bed (Figure 4). A Fibreglass Evercoat (FIB 622) epoxy was used to bond the sheets to the timber. A sheet was bonded to two steel anchor plates and prestressed between two steel abutments. The beams were flipped upside down to perform the work easily in the stressing bed. Two layers of wax paper were put on both ends of the beam to act as a bond breaker between anchored sheets and the beam. Bonding the prestressed sheet to the timber beam was accomplished by applying a 2 mm layer of epoxy to the tension face of the beam and then raising the beam until it was in contact with the prestressed sheet along the full length of the beam. Once the height of the beams was fixed, the beam was adjusted and positioned such that the sheet was centered within the edges of the beam.

The hardwire sheet was loaded axially by a jack at the stressing abutment. For the medium density sheet, the prestressing level was 70% of the ultimate tensile strength specified by the manufacturer, representing a load of approximately 17 kN. For the high density sheet, the prestressing level of 70% of the ultimate tensile strength specified by the manufacturer was approximately equal to 30 kN. Additional epoxy was applied to fill any voids between the sheet and the epoxy as shown in Figure 5. Heavy loads were then applied on top of the sheet to prevent de-bonding of the sheet as shown in Figure 6. A GFRP sheet with a 45˚ angle at each end was applied by bonding it to the timber using the epoxy resin as shown in Figure 6. This sheet also served as anchorage for the post-tensioning sheets.

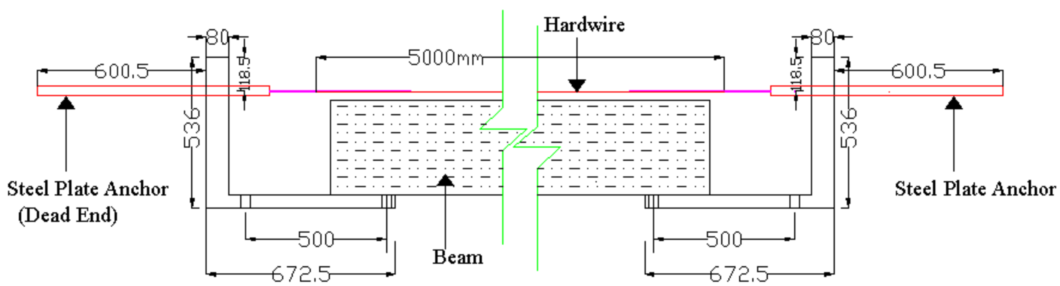

Figure 4: Cross Section of Stressing Bed

Figure 5: Applying Layer of Epoxy Resin to Bond the Sheet

Figure 6: Post-tensioned Timber Beam

2.3 Test Setup

All the beams were tested in accordance with ASTM standard D198 (ASTM 1999). The beams were simply supported on rollers with a span of 4500 mm and tested in three point bending. A monotonic static load was applied by a servo-hydraulic testing machine with a displacement rate of 3 mm/minute. A 500 mm long bearing plate was used to distribute the applied load. Lateral support was provided at either end. A view of the test setup can be seen in Figure 7.

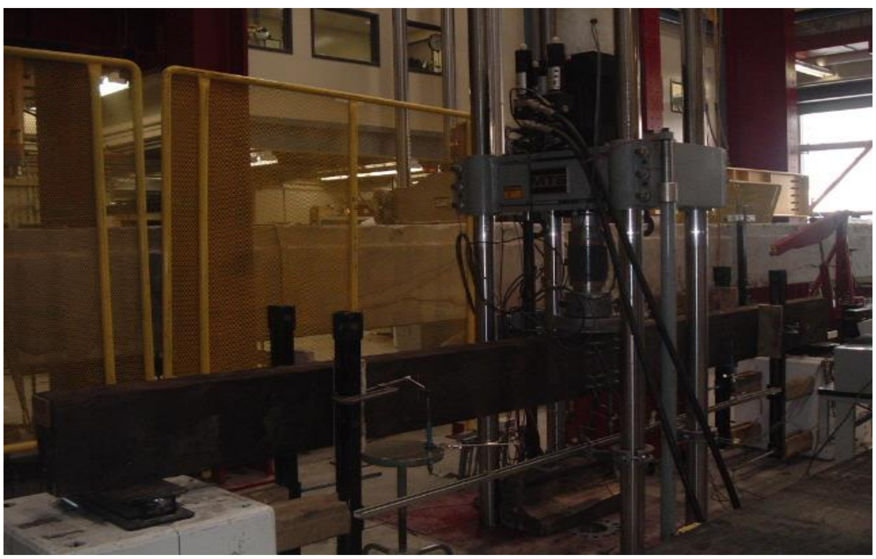

Figure 7: Timber Beam Test Setup

2.4 Beam Instrumentation

The beams were instrumented with four linear variable displacement transducers (LVDTs) to measure the vertical displacement at midspan and quarter spans. Four 200 mm pi-gauges were used to measure the strains in the beam at midspan at different heights. The four pi-gauges were placed symmetrically, 65 and 115 mm from the mid height of the beam as shown in Figures 8 and 9. All readings from the LVDTs, pi-gauges, machine load and the stroke were recorded using a data acquisition system.

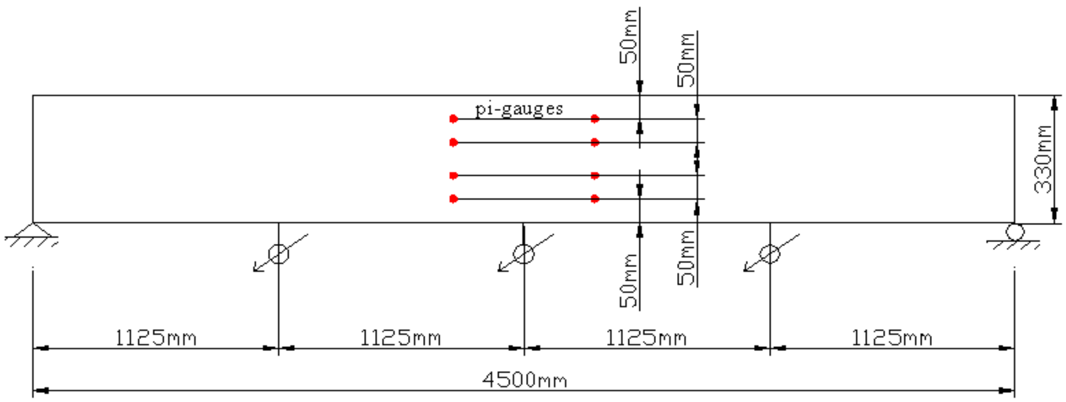

Figure 8: Schematic Drawing of Beam Instrumentations Used During Experiment

3 EXPERIMENTAL RESULTS

This paper will present the experimental results in terms of strength increase for the beams and their stiffness.

3.1 Load–Deflection Behaviour

All six beams were tested prior to strengthening up to a load of 15 kN in order to determine the initial stiffness. The load-deflection curves for timber beams post-tensioned with hardwire sheets are shown in Figure 9. The load deflection curves are generally linear up to 75% of the failure load. The average failure load for these beams was found to be 137.2 kN while the failure load for the control beam was 85.4 kN. Test results showed that the strength was increased on average by 60.7% while there was a slightly increase in the measured stiffness of the beams before and after strengthening. The failure loads, strength and the stiffness values are listed in Table 3.

3.2 Failure Mode

All post-tensioned timber beams were failed in flexure and one control beam failed in a combination of flexure and shear. The common mode of failure was tension parallel to the grain. The sheets did not rupture in any of the beams due to the high strength of the reinforcement. The sheet de-bonded along the mid length of the beams at the failure load, as can be seen in Figure 10.

3.3 Beam Stiffness

All timber beams were tested in bending up to a load of 15 kN prior to strengthening so that the effect of strengthening on bending stiffness could be evaluated for each beam. The apparent stiffness was calculated from the initial linear-elastic portion of the load-deflection curve of each beam by using equation 1.

[1] $$EI = \frac{\Delta P L^3}{48 \Delta \delta}$$

Where: ΔP is the given range of applied load,
L is span length,
$\Delta\delta$ is the given range of deflection for load range ΔP,
E is the modulus of elasticity,
I is the moment of inertia.

Table 3 presents the results of flexural tests in terms of stiffness, failure load and strength increase. The results show that the post-tensioned timber stringers had stiffness increase on average by 4%. This increase in stiffness can be more pronounced if larger post-tensioning force was applied. However, the first set of the sheets could not be stressed to more than 20 kN. With larger post-tensioning force, the stiffness could be further increased. That is why the second type of the sheets that was used for beams P1 and P2. The high density sheets had capacity of 90 kN, and the limiting factor became the compressive strength of the timber parallel to the grain. Aside from the limiting factors mentioned above, this strengthening method was also new, and as a result the know-how for performing this type of post-tensioning needs to be further extended. It is for example important that the thickness of epoxy between the reinforcement and the beam is kept to a minimum; otherwise the losses are too high.

Table (3) Experimental Results for Beams Post-tensioned with Hardwire Sheets

Beam	Grade	Hardwire type	Stiffness of unreinforced beam N.mm^2	Stiffness of reinforced beam N.mm^2	% Change	Failure Load (kN)	Strength Increase %
P1	NO.2	High density	4.98E+12	5.23E+12	4.87%	148.48	73.93%
P2	NO.1		5.48E+12	5.89E+12	6.90%	134.25	57.26%
P3	NO.2	Medium density	5.97E+12	6.16E+12	3.10%	119.50	39.98%
P4	NO.1		5.63E+12	5.69E+12	1.06%	167.63	96.36%
P5	NO.1		5.63E+12	5.87E+12	4.10%	148.70	74.18%
P6	NO.2		5.01E+12	5.201E+12	3.70%	104.48	22.38%

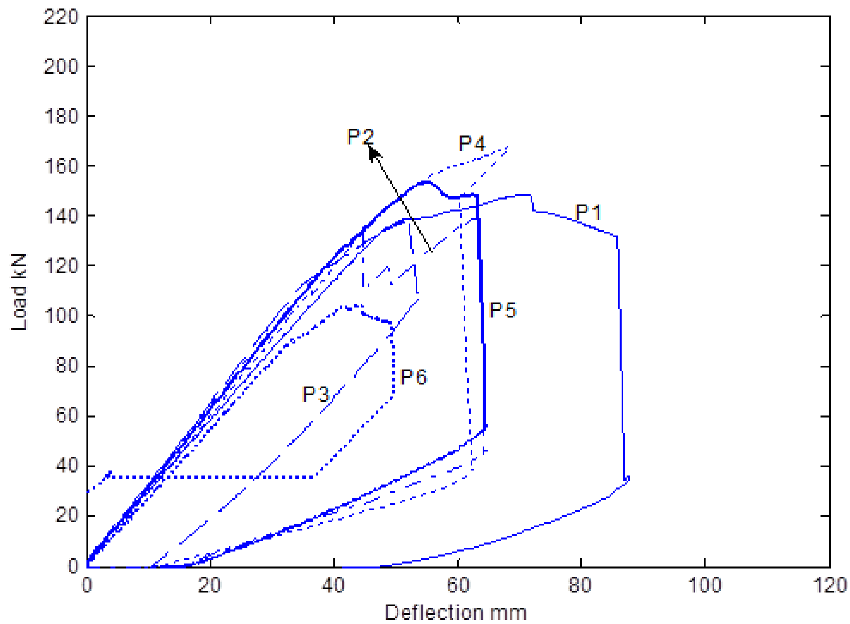

Figure 9: Load-Deflection Curves for Timber Beams Reinforced with Prestressed Hardwire

Figure 10: De-bonding of the Hardwire Sheet in the Tension Zone

4 CONCLUSIONS

This paper has shown that hardwire sheets may become a useful method for increasing the flexural capacity of Douglas-fir timber bridge beams. It was found that the strength and the stiffness were increased on average by 66% and 6% for post-tensioning with high density hardwires sheets, while they were increased by 58% and 3% for post-tensioning using medium density hardwire sheets.

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