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GLULAM BEAMS EXTERNALLY REINFORCED WITH CFRP PLATES

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ABSTRACT

An experimental program was undertaken to investigate the effectiveness of carbon fiber reinforced polymer (CFRP) plates as flexural reinforcement of glued laminated timber (glulam) beams. Beams with and without reinforcement were tested up to failure in a four-point bending configuration. A comparison between the flexural behaviuor of control unreinforced beams with reinforced beams is shown and discussed. The results demonstrated increase in strength, stiffness and ductility when CFRP plate is bonded at tension side of cross section. Research findings indicated that the use of proposed reinforcing solution improves utilization of the compression characteristics of timber. Based on the experimental observations, a theoretical model is developed to predict the ultimate moment capacity and bending stiffness of CFRP-reinforced glulam beams.

KEYWORDS: Glulam, carbon fibers, reinforcement, bending test, analytical modelling.

INTRODUCTION

Glued laminated timber (glulam) is a structural engineered wood product fabricated by gluing together small sections of timber boards into large structural members. The glulam technology allows the production of timber elements in which the natural defects are dispersed, ensuring a final product with more uniform properties, whose dimensions are theoretically unlimited. Capable of spanning greater lengths and carrying greater loads, glulam has advanced the timber as a structural material. Although glulam offers significant improvement over solid timber, its ultimate bending strength often remains limited by strength reducing flaws, such as knots and finger joints, in the tension zone. The tension failure is brittle, random and difficult to predict. Premature tension failures force design codes to apply greater safety factors, leading to increased use of material. Additionally, to satisfy serviceability conditions, long span glulam beams must be relatively high. This is a consequence of timber low modulus of elasticity and its creep behaviour, which imposes a consideration of load duration and environmental conditions to calculate long-term deflection.

The placement of reinforcement with a high modulus of elasticity and high tensile strength in tension zone of flexural members may improve the strength, stiffness, ductility and potentially lower the mechanical variability when compared to unreinforced members. In addition, a reinforcing system may be used to reduce dimensions of the glulam elements and allow utilization of lower grades of wood (Gentile 2000). Traditional reinforcing techniques of timber structures use steel bars or plates. Such methods, however, have drawbacks such as increased dead loads, installation difficulties, limited versatility and corrosion vulnerability. As an alternative, the steel reinforcement can be replaced with fiber reinforced polymer (FRP) plates, sheets or bars. Fiber reinforced polymer is an advanced composite material which consists of many thin and long fibers embedded in a polymer matrix. While the fibers provide strength and stiffness of FRP, the matrix is essential to transfer forces between the individual fibers and to protect the fibers from abrasion and environmental degradation. In civil engineering applications, the two most commonly used fiber types are glass and carbon. The remarkable characteristics such as high stiffness and tensile strength, low weight, easy installation, high durability (no corrosion), electromagnetic neutrality and wide variety of available sizes and shapes, make these composite materials suitable for many structural applications (ACI 440.2R-08 2008).

The reinforcing of solid and glued laminated timber beams using FRP materials has been investigated by many authors, covering the development of new reinforcement techniques, supported on experimental, analytical and numerical approaches. The majority of extant studies are focused on the mechanical performance of FRP-timber system and the failure mode of reinforced members (Gentile 2000; Romani and Blaß 2001; Gilfillan et al. 2003; Svecova and Eden 2004; Borri et al. 2005; Micelli et al. 2005; Schober and Rautenstrauch 2006; Jacob and Barragan 2010; Fiorelli and Dias 2011; Raftery and Hart 2011; Prachasaree and Limkatanyu 2013; Kim et al. 2013; D'Ambrisi et al. 2014). Interaction between FRP composites and the timber element, and bond strength of the timber-adhesive-composite interface have also been investigated (Raftery et al. 2009, Juvandes and Barbosa 2012; de Jesus et al. 2012; Sena-Cruz et al. 2013).

This research examines the flexural performance of glulam beams reinforced with carbon fiber reinforced polymer (CFRP) plates. The reinforcement was externally bonded to the tension zone of the beams. The cross section ratio between the CFRP and the glulam was 0.46 %. An experimental program based on a four-point bending test configuration was conducted on reinforced beams and unreinforced beams that were used as control specimens. Experimental results are discussed in terms of load-deflection behaviour, failure mode, ultimate moment capacity, stiffness and strain profile distribution. In addition, theoretical model is proposed to compute the mechanical properties of tested beams. The accuracy of the analytical results is checked by a comparison with experimental data.

MATERIAL AND METHODS

Timber

Glulam was made from spruce timber (*Picea abies*) which characterized as grade C24 according to EN 338 (2009). In order to minimize the influence of the variability in timber properties, all timber used in the study was sourced from the same stand. Timber stock was kiln dried to approximately 18 % moisture content and then conditioned in an environment of 65 ± 5 % relative humidity and the temperature of 20 ± 2°C for three months. An average equilibrium moisture content of 11.7 % was obtained after the conditioning period and an average timber density of 427 kg.m^{-3} was recorded. The timber was tested in tension, compression and flexure with reference to EN 408 (2010) in order to determine its mechanical properties. Specimens with dimensions of 20 x 20 x 120 mm, 20 x 20 x 300 mm and 20 x 20 x 400 mm were used for compressive, tensile and bending tests, respectively. Twenty identical specimens were prepared for each test type. The tests were performed using a universal testing machine (Instron 1332) with a 100kN load capacity, under crosshead displacement control. The behaviour of timber specimens under different types of loading is illustrated on Figs. 1-3.

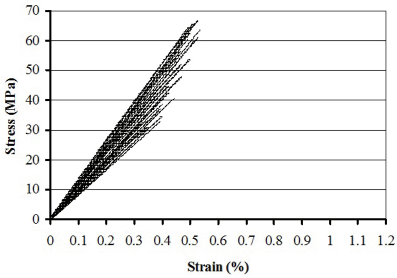

Fig. 1: Stress-strain curves for timber from tensile tests.

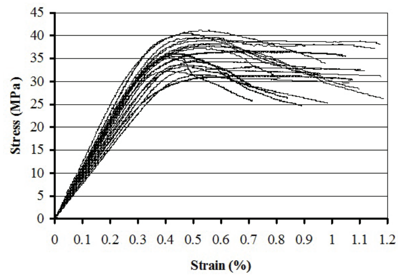

Fig. 2: Stress-strain curves for timber from timber from tensile tests.

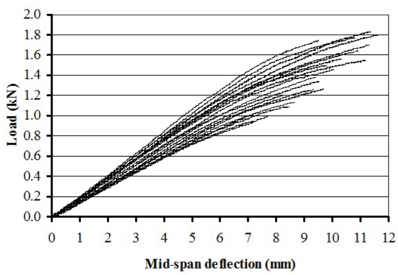

Fig. 3: Load-deflection curves for timber from bending tests.

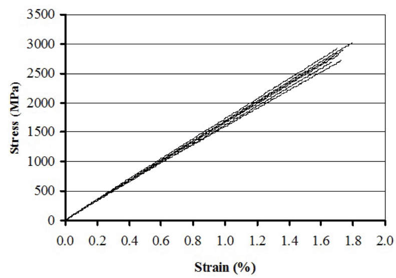

Fig. 4: Stress-strain curves for CFRP plate from tensile tests.

Mechanical properties of timber are given in Tab. 1. In order to take into account the size effect, the tested tensile and bending strength values were adjusted according to EN 384 (2010).

Tab. 1: Experimental mechanical properties of spruce timber.

Property	Value
Tensile strength parallel to grain (MPa)	27.8
Coefficient of variation (%)	25.2
Compressive strength parallel to grain (MPa)	36.3
Coefficient of variation (%)	9.8
Bending strength (MPa)	42.5
Coefficient of variation (%)	20.6
Modulus of elasticity parallel to grain (MPa)	11080
Coefficient of variation (%)	12.6

Fiber reinforced polymer

The FRP material used in this experimental test program comprised carbon fibers in an epoxy matrix. Carbon fibers are much more expensive than glass fibers, but have significantly higher mechanical properties. In addition, carbon fibers display outstanding resistance to thermal, chemical and environmental effects (Andre 2006). The CFRP plate used as reinforcement was Sika CarboDur S613 (60 x 1.3 mm), provided by Sika AG. This is a pultruded carbon fiber reinforced polymer plate designed for strengthening concrete, timber and masonry structures. The plate consists of unidirectional fibers positioned longitudinally, and its fiber volume content is greater than 68 %. The density of this reinforcement is 1.6 g.cm^{-3}.

The CFRP material was tested in tension in accordance with EN 527-5 (2009) in order to determine its tensile modulus of elasticity, tensile strength and strain at break. The tests were conducted with specimens extracted from plate longitudinally. Six specimens with dimensions of 15 x 1.3 x 250 mm were used. Testing was carried out on an Instron 1332 testing machine with a constant crosshead movement. Fig. 4 shows the experimental stress-strain curves. Mechanical properties of carbon fiber plates are given in Tab. 2. Experimental results were very close to those values specified by the manufacturer (Sika AG 2011).

Tab. 2: Experimental results from tensile tests of CFRP plate.

Property	Value
Tensile strength (MPa)	2846
Coefficient of variation (%)	4.5
Modulus of elasticity (MPa)	165543
Coefficient of variation (%)	2.8
Strain at break (%)	1.73
Coefficient of variation (%)	3.2

Adhesive

The implementation of FRP reinforcement requires the use of adhesives. Technical data sheets for FRP materials usually give indications for which adhesive to be used depending on the structure that is being reinforced. The most suitable adhesives for composite materials are based on epoxy resin. Epoxy adhesives have high mechanical properties, superior toughness, good creep and chemical resistance. Because of their good gap-filling characteristic, limited shrinkage during curing, their ability to achieve full cure at ambient temperature and the low clamping pressures that are required, these adhesives are appropriate for applications in FRP-timber bonded connections (Raftery et al. 2009).

Tab. 3: Properties of adhesive Sikadur-30 from manufacturer data sheets (Material and curing conditions: 23°C and 50 % R.H.).

Property	Value	Testing standard
Mixing ratio (resin : harder by volume)	3 : 1	-
Density (g.cm^{-3})	1.65	-
Pot life (minutes)	70	-
Tensile strength after 7 days (MPa)	46.8	ASTM D-638
Elongation at break after 7 days (%)	1.0	ASTM D-638
Compressive strength after 7 days (MPa)	59.3	ASTM D-695
Bending strength after 7 days (MPa)	46.8	ASTM D-790
Modulus of elasticity after 7 days (MPa)	11721	ASTM D-790

The adhesive used in this experimental test program was Sikadur-30, which is a product of Sika AG. Sikadur-30 is two-component, solvent-free, thixotropic adhesive. A base resin and curing agent (hardener) compose two components of adhesive. By mixing these two components in specific ratio, adhesive is ready to be used. The adhesive is designed for use at normal temperatures between 10 to 35°C. The main properties of epoxy adhesive provided by the manufacturer are shown in Tab. 3 (Sika AG 2014).

Beam preparation

The experiment was carried out on a total of 15 glulam beams, including five unreinforced beams (Group A) and ten reinforced beams (Group B). The nominal size of beams was 80 (width) x 210 (height) x 4000 mm (length). Each beam was composed of seven 80 wide and 38 mm thick laminations. Glulam manufacture procedure included strategically balanced lay-ups using better quality laminations (the ones with fewer critical strength reducing defects such as knots, fissures and grain deviation) in the region of the highest stress. The laminations were bonded using phenol-resorcinol adhesive. A pressure of 0.6-0.8 MPa was applied to the closed assembly over the course of 24 h at the temperature of approximately 20°C.

The gluing of CFRP plates was planned very carefully. The timber surface was prepared in a way recommended by a manufacturer so that CFRP plates and timber would be properly bonded. Sanding with sandpaper was applied at those zones at which CFRP's systems were going to be installed so as to eliminate splinters and dust, all of which was accompanied by vacuum cleaning. CFRP plates were cut by diamond cutting disc to a desired length, and were cleaned properly with solvent (Sika Colma Cleaner) in order to remove contaminants. Well-mixed adhesive Sikadur 30 was applied with a trowel across the prepared substrate to thickness of approximately 1 mm. Epoxy adhesive was applied using a special "roof-shaped" spatula across cleaned carbon plate to nominal thickness of 1 mm. Within the open time of the epoxy, the CFRP plate was placed onto the glulam surface. Using a hard rubber roller, the carbon plate was pressed into the epoxy resin until the adhesive was forced out on both sides. All excess epoxy was removed. Neither pressure application on the reinforcement nor clamping was necessary.

Fig. 5: Reinforced glulam beams (Group B).

Fig. 5 shows typical sample of glulam beams reinforced with externally bonded CFRP plates. The reinforced beams were cured at least 7 days at the room temperature of 22±2°C in order to ensure proper and adequate bonding between CFRP plate and glulam.

Beam testing

Testing of the beams was conducted at the Material Testing Laboratory of the Faculty of Civil Engineering, University of Belgrade. Both unreinforced and reinforced beams were subjected to bending test in accordance with EN 408 (2010). The beams were tested in a four-point bending configuration over a simply supported span of 3780 mm (18 times the beams' height). The two loading points were placed at the third span positions. The load was applied monotonically up to the point of failure using a loading cell (capacity of 250 kN), powered by hydraulic jack. In test procedure, the load was transformed from one to two loading points using a steel beam. Test specimens were simply supported on roller bearings. In addition, roller bearings were used at load application points to ensure pure vertical loading and also to provide moment free loading. Steel plates were placed beneath the two load points, as well as at the supports, so that local indentations would be brought to minimum. Lateral restraints were provided so as to prevent lateral buckling of beams. The typical test set-up is shown in Fig. 6.

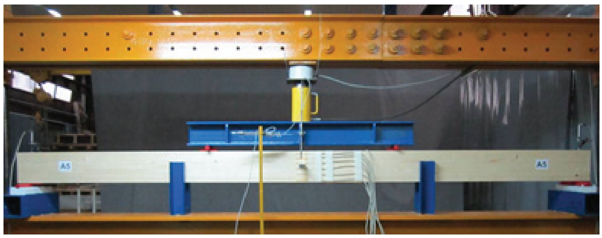

Fig. 6: Test set-up.

Deflections of the beams were monitored by linear variable displacement transducers (LVDTs). One LVDT (HBM W100TK, ±100 mm) measured deflection at mid-span, while two LVDTs (HBM W20TK, ±20 mm) measured local indentation at the supports. Furthermore, strains were monitored by strain gauges (TML PL-60-11, 60 mm long) at the mid-span of the beam across the beam's height. This was necessary in order to record the extent of plasticization in timber, as well as the shift in neutral axis due to compressive plasticization. The strain from strain gauges, deformation data from LVDTs and corresponding load data from a loading cell were all recorded by a computerized data acquisition system (HBM MGC). Data acquisition was carried out with a frequency of 2 Hz.

Monotonic static load was applied at a stroke-controlled rate of 4.5 kN per minute so as to cause the failure of the unreinforced beams in approximately 10 min. The reinforced beams were tested with the same load rate in order to ensure a fair comparison of test results. The failure of the reinforced beams was achieved in 10-15 min. Before the tests were performed, the specimens were conditioned in an environment of 65 ± 5 % relative humidity and temperature of 20 ± 2°C. Soon after testing, moisture content was measured in each beam by a digital hygrometer at different locations. The moisture content in tested beams varied between 10.8 and 11.4 %.

RESULTS AND DISCUSSION

Load-deflection behaviour and failure mode

The load-deflection behaviour up to failure of both unreinforced and reinforced beams is shown in Fig. 7.

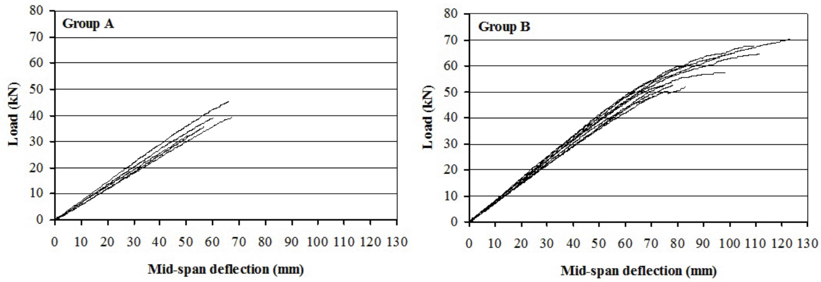

Fig. 7: Load-deflection curves for unreinforced and reinforced beams.

The unreinforced glulam beams (Group A) exhibited quite linear load-deflection behaviour until the point of failure. All unreinforced beams experienced failures brought about by excessive tensile stresses in bottom laminations. Due to brittle nature of the timber behaviour in tension, most beams failed suddenly, without any visible fractures before reaching ultimate load carrying capacity. Once the timber showed signs of fractures, the cracks promptly developed and in some cases propagated along the timber grain, past the load application points and towards the supports (Fig. 8a). Four out of five beams showed failure initiated at defects (e.g. knots), which were located in the zone of maximum bending moment between load application points (Fig. 8b). One beam failed in clear wood in the bottom lamination (Fig. 8c). There were no signs of compressive plasticization at the top lamination in any of unreinforced beams.

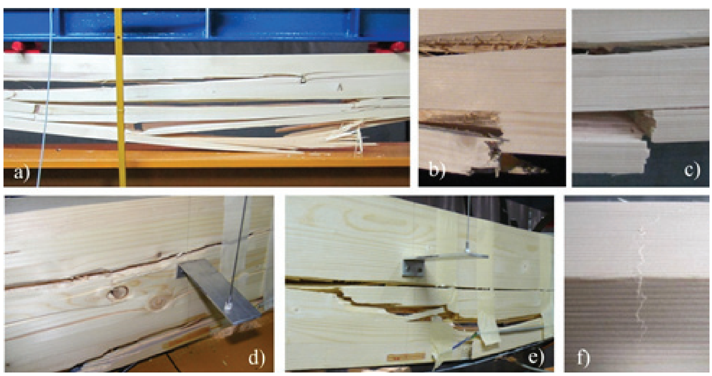

Fig. 8: Failure modes for unreinforced and reinforced beams.

Glulam beams with CFRP plate positioned on the intrados surface (Group B) demonstrated ductile behaviour. The failure mechanism of these beams was characterized by tension failure, with or without partial plasticization in compressive zone. As a comparison, the same failure modes were obtained through the research done by Romani and Blaß (2001) for CFRP plate reinforced glulam. Initially, the load-deflection behaviour showed linear elasticity up until local fractures occurred in the tension zone. Yielding of compressive timber produced a nonlinear response that was ended by a sudden drop of load as a result of tensile timber rupture (Figs. 8d and 8e). The tough compression zone displayed signs of plasticization in the form of buckled fiber, but generally it remained intact (Fig. 8f). With exception of two beams which failed in clear wood, all failures were initiated at defect or discontinuity in the bottom laminations. The presence of shear cracks after the initial tensile fractures was noticed in number of the beams. In some cases, the tension failure in timber was of explosive nature, which caused CFRP plate to peel off as the fracture developed. The adhesion between timber and reinforcement failed only after timber rupture. From visual inspection of the specimens it was concluded that the degree of ductility which was experienced prior to failure was directly dependent on the quality of the bottom timber laminations. Horizontal reinforcement acted as a bridge across the timber defect and contributed to tensile capacity of the beam (Borri et al. 2005).

Ultimate moment capacity and stiffness

The results of experimental tests in regard to maximum load, ultimate bending moment, maximum mid-span deflection and bending stiffness for each beam group are shown in Tab. 4.

For each beam, the ultimate bending moment M_u was calculated according to the Eq.:

$$M_u = \frac{P}{2}\frac{l}{3} = \frac{Pl}{6} \tag{1}$$

where: P - the maximum applied load,
l - the beam span.

Tab. 4: Experimental results from bending tests.

Beam group	Average	Min	Max	SD	CV (%)	Increase (%)
Maximum load (kN)						
A	38.3	32.3	45.4	4.9	12.8	-
B	59.1	48.4	70.1	7.3	12.4	54.3
Ultimate bending moment (kNm)						
A	24.5	20.7	29.0	3.1	12.8	-
B	37.8	30.9	44.9	4.7	12.4	54.3
Maximum mid-span deflection (mm)						
A	60.0	50.5	66.3	6.9	11.4	-
B	93.8	71.0	123.5	17.4	18.6	56.2
Bending stiffness EI (×10^{11} N.mm^2)						
A	6.54	60.6	72.9	5.0	7.6	-
B	7.73	7.20	8.31	0.38	4.9	18.1

SD = Standard deviation, CV = Coefficient of variation.

The apparent bending stiffness EI was calculated from the initial linear portion of the load-deflection curve of each beam and using the mid-span deflection equation for four-point bending:

$$EI = \frac{23}{648}\frac{\Delta P l^3}{2\Delta w} \tag{2}$$

where: $\Delta P/\Delta w$ - the slope of load-deflection curve between 10 and 40 % of maximum load,
l - the beam span.

The effect of the reinforcement on the ultimate moment capacity is clearly demonstrated. The beams of Group B reinforced at the intrados surface achieved an average maximum bending moment of 37.8 kNm compared to the unreinforced beams (Group A) whose moment capacity totalled 24.5 kNm, which represented an increase of 54.3 %. The results in regard to load carrying capacity are in reasonable agreement with the results obtained by other researchers (Romani and Blaß 2001; Gilfillan et al. 2003; Issa and Kmeid 2005) where the ultimate load was increased by

35-70 % when glulam beams were reinforced using CFRP plates for reinforcement ratio between 0.4-1.2 %.

The unreinforced beams (Group A) had an average global stiffness of 6.54 × 1011 N.mm^2. Introduction of material with high tensile modulus of elasticity in the cross section resulted in improvement in stiffness for reinforced beams. The reinforced beams of Group B attained an average stiffness of 7.73 × 1011 N.mm^2, which represented an average increase of 18.1 % when compared with the unreinforced beams. The results concerning the stiffness are in good agreement with the results reported by others (Romani and Blaß 2001; Gilfillan et al. 2003; Issa and Kmeid 2005) where the increase of stiffness varied from 15 to 55 % for glulam beams reinforced with CFRP plates.

CFRP reinforcement in glulam beams reduced the mid-span deflection compared to the unreinforced beams at the same load level in the elastic region. This low deflection is desirable from the standpoint of serviceability limit state in timber structures. On other hand, CFRP reinforced beams underwent large deformations before the global failure occurred when compared to the unreinforced beams. The average increase in mid-span deflection was 56.2 %.

Strain profile distribution

Typical strain distributions across the height of both unreinforced and reinforced beams are shown for different load levels in Fig. 9. These profiles report compressive and tensile strains as negative and positive values on the x-axis, respectively, as well as positions of strain gauges' locations on the y-axis relative to cross section centre.

Strain distribution across the height was quite linear until failure in case of the unreinforced beams. Tensile and compressive strains were almost identical at different load levels. The position of the neutral axis remained unchanged as the applied load was increased, which proved there was no plasticization in the compression zone.

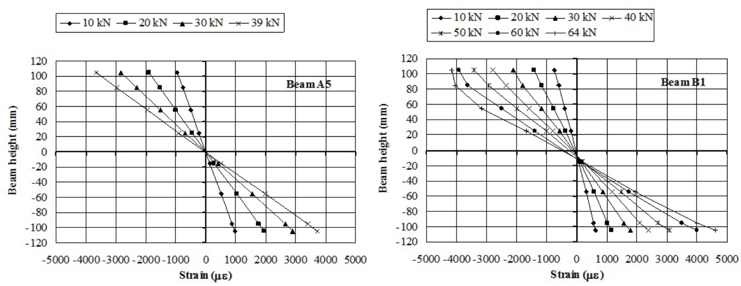

Fig. 9: Strain profiles at different load levels for unreinforced and reinforced beams.

Linear strain distribution across the height was noted for the reinforced beams in the elastic region. A non-linear strain distribution prior to failure could be observed for the reinforced beams where plastic behaviour in the compression zone was reached. Due to the contribution of the CFRP reinforcement, the neutral axis was shifted towards the beam tension zone. No significant variation of neutral axis position was recorded as the applied load increased and plasticization in compression zone occurred. The strain measured on CFRP plate corresponded quite well to the strain measured in the adjacent timber lamination. This indicates that there was no appreciable slip between timber and reinforcement.

The reinforced beams demonstrated a noticeable improvement in the timber strain values. Tensile strain in the reinforced beams was lower compared to the unreinforced beams at the same load level. In addition, the presence of CFRP reinforcement overcame the influence of a local defect in the timber, all of which resulted in the increase in ultimate timber tensile strain. Tab. 5 shows average ultimate tensile strains for beam groups. The average increase of tensile strain at the point of failure for the reinforced beams was 25.5 % when compared to the unreinforced ones.

Tab. 5: Average failure tensile strains in timber.

Beam group	Tensile strain (‰)	Increase (%)
A	3.66	-
B	4.59	25.5

Test results regarding the strain demonstrated that timber compressive properties are more utilized when CFRP reinforcement is included in the beam tension zone. Reinforcement allows for greater strains in the compression region due to larger distance of timber compression fibres from the neutral axis.

Theoretical model

Theoretical model is developed to predict the ultimate moment capacity and bending stiffness of both unreinforced and reinforced beams. The model incorporates realistic behaviour of materials and different failure modes. In other studies, similar models have been proposed by various researchers (Romani and Blaß 2001; Borri et al. 2005; Fiorelli and Dias 2011; D'Ambrisi et al. 2014).

The basic assumptions of the model are listed as follows:

- Strain distribution is linear across the entire height of the beam. Plane sections remain plane during bending.
- Adhesive bond acts perfectly, there is no slip between adjacent timber laminations or between timber and CFRP material.
- Stress-strain relationship for timber in tension is linear elastic, maximum stress value corresponds to the bending strength.
- Stress-strain relationship for timber in compression is linear elastic-perfectly plastic, maximum stress value corresponds to the axial compressive strength.
- Stress-strain relationship for CFRP material is linear elastic.
- Modulus of elasticity of timber is the same in tension and compression zone of the cross section.

For the sake of simplicity, the modelling was conducted in two phases based on the degree of plasticization in the compression zone. Fig. 10 illustrates these phases and corresponding stress/strain state.

If failure of section occurs when the tensile stress in outermost timber fiber σ_t reaches the ultimate strength f_m before the compressive stress of timber σ_c reaches the maximum strength f_c, the depth x of the neutral axis and the ultimate M_u are given by:

$$x = \frac{1}{bh + nA_f}\left(\frac{bh^2}{2} + nA_f h\right) \tag{3}$$

$$M_u = f_m \frac{I}{h - x} \tag{4}$$

with

$$I=\frac{bh^3}{12}+bh\left(\frac{h}{2}-x\right)^2+nA_f\left(h-x\right)^2 \tag{5}$$

where: I - the second moment of area of the homogenized cross section.

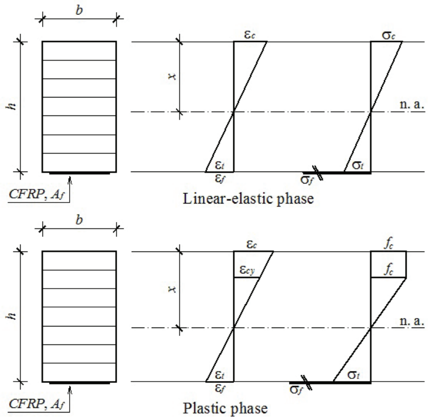

Fig. 10: Idealised strain and stress distributions in CFRP-reinforced glulam.

If the stress of the timber at the compression side σ_c reaches the maximum strength f_c before tensile stress σ_t reaches the ultimate strength f_m, the depth of neutral axis x is a solution of the equilibrium equation:

$$f_c\left[x-\frac{f_c}{f_m}(h-x)\right]+\frac{f_c^2}{f_m}\frac{h-x}{2}=f_m\frac{(h-x)}{2}+f_m\frac{nA_f}{b} \tag{6}$$

while the ultimate moment M_u is given by:

$$M_u=\frac{b}{6}\left[3f_c x^3+\left(2f_m-\frac{f_c^3}{f_m^2}\right)(h-x)^2+6f_m\frac{nA_f}{b}(h-x)\right] \tag{7}$$

In Eqs. 3 - 7 is the homogenization coefficient (ratio between the modulus of elasticity of CFRP material and the modulus of elasticity of timber), A_f is the area of CFRP reinforcement cross section, f_c is compressive strength of timber, f_m is bending strength of timber, while the geometrical characteristics are defined in Fig. 10.

Considering the linear-elastic state, the bending stiffness of the beams (EI) is calculated by multiplying the second moment of area of the cross section by the modulus of elasticity of the timber to which all the constituent parts were transformed.

Eqs. 3 - 7 were initially applied to evaluate the ultimate moment and the bending stifness of the unreinforced beams ($A_f = 0$). These equations were then applied to estimate the ultimate moment and the bending stiffness of the reinforced beams considering the cross section of the selected CFRP plate ($A_f = 78$ mm^2). The mechanical properties of timber and CFRP material used in the analytical calculation were adopted on the basis of material characterization tests. The fact that the tensile stress at failure in bending would effectively be increased with the addition of CFRP plate was taken into account by modification of bending strength of reinforced timber. According to the tensile strain data in Tab. 5, modification factor was taken as 1.25.

Comparisons of the ultimate moment capacity and stiffness results from the experimental testing and analytical modelling are shown in Tab. 6.

Tab. 6: Comparison between experimental results and analytical predictions.

Beam group	Experimental	Analytical	Analytical/ Experimental
Ultimate bending moment (kNm)			
A	24.5	24.7	1.01
B	37.8	35.6	0.94
Bending stiffness EI ($\times 10^{11}$ N.mm^2)			
A	6.54	6.84	1.05
B	7.73	8.17	1.06

Good agreement is achieved between two sets of results. Regarding the ultimate moment capacity, the theoretical model gives excellent prediction for unreinforced beams while prediction for reinforced beams is satisfactorily conservative in relation to experimentally tested beams. On other hand, the theoretical model overestimates the stiffness of the unreinforced beams and reinforced beams but is within an acceptable range.

CONCLUSIONS

This paper presented an experimental investigation and analytical modelling of glulam beams reinforced with CFRP plates. Altogether ten beams externally reinforced at tension side and five unreinforced control beams were tested in flexure. The effect of reinforcing solution was evaluated with regard to load-deflection behaviour, failure mode, ultimate moment capacity, stiffness and strain profile distribution. A theoretical model was proposed so as to predict mechanical properties of reinforced beams. The following conclusions have been drawn:

- Preparation of the reinforced beams confirmed that the proposed reinforcing technique is easy to apply in engineering practice.
- The unreinforced beams showed linear elastic behaviour with brittle tension failures. Tensile reinforcement had an influence on the plastic behaviour of beams, which was not so predominant due to timber low tensile strength. The rupture of CFRP reinforced beams was mainly governed by the defect in timber.
- The bond between reinforcement and timber was effective up to the moment of beam fracture. CFRP plate debonding was only observed as a post failure phenomenon. This means that the epoxy adhesives were quite successful in transferring the loads from timber to reinforcement.
- The addition of CFRP plates at tension side of glulam beams resulted in improvements in the ultimate load carrying capacity and stiffness when compared to the unreinforced beams. The average increase in the ultimate moment capacity of 54.3 % and average increase in the stiffness of 18.1 were attained for 0.46 % reinforcement percentage.
- The reinforced beams experienced lower deflection in elastic region compared to the unreinforced beams. On the other hand, the reinforced beams ensured greater deflection before the failure.
- CFRP reinforcing improved strains in timber. The presence of reinforcement in tension zone allowed for timber compression fibres to reach their yield strains and, hence, better utilization of the compressive properties of timber. In addition, the ultimate timber tensile strain increased in the reinforced beams. This indicates the ability of CFRP plate to reduce the effect of natural defect in timber.

- The proposed theoretical model based on plastic compression behaviour of timber and maximum stress criterion may be used to predict the stiffness and ultimate moment capacity of CFRP reinforced beams. Multiplying the bending strength by factor 1.25 for reinforced beams is found to give good correlation with the experimental results.

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