Flexural Strengthening of Timber Beams Using Glass Fibre Reinforced Polymer

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ABSTRAT: This research was conducted to investigate the bending behavior of timber beams strengthened with glass fibre reinforced polymer (GFRP) rods. Seven timber beams of Yellow Meranti species with the dimensions of $100 \times 200 \times 3000$ mm were tested. One of the beams was used as a control beam (unstrengthened) while the remaining six beams were strengthened before tested to failure under four point loading. The bending behavior of the beams was studied through their load-deflection characteristics and strain distribution across the depth of the beam upon loading and also the failure modes. The results showed that the strengthened beams performed better than the control beam. The ultimate load has increased between 20 - 30% for the strengthened beams when compared to the control beam. The stiffness increased between 24 - 60% for the strengthened beams. The beam is categorized as under-reinforced, balance- reinforced, and over-reinforced if the percentage of GFRP rod is less than 0.32%, between 0.32 - 0.35%, and greater than 0.35%, respectively. GFRP rod bonding system has a good potential to be used in the new construction or rehabilitation process of timber structures.

1 INTRODUCTION

Timber is one of the oldest building materials ever used by mankind. Steel and concrete replaced wood as the major building construction material in the 20th century. However, in the 21st century, wood is still being used especially in United States for many short-span bridges (Buell & Saadatmanesh 2005). Timber is also used in various constructions such as residential buildings, commercial structures and sport complexes. A number of nations are also involved in the structural use of timber in multi-storey constructions. On the contrary, in Malaysia, due to lack in technical knowledge in advanced timber engineering and commitment in quality assurance of structural timber members, local Public Works Department (PWD) has been replacing old timber structures especially bridges with concrete during the 7th Malaysian Plan, 1996–2000 (Tan et al. 2002). In addition, the PWD is currently suspending the use of prefabricated timber roof trusses for projects under their direct supervision.

Timber is well known for its high strength to weight ratio**,** which means that timber is easy to construct and move because there is no formwork required. The use of timber will lower the construction cost because no heavy machinery is needed. In addition, construction time can be shortened because they do not require extra time for hardening which, in turn, is needed for concrete structures. Due to its insulation from sound and electricity, as well as resistance to corrosion and oxidation, timber is also popular in light construction.

However, timber structures may also encounter problems due to design failure, excessive loading, and infestation by termites. One of the causes of design failure is the varying strength of timber in which the strength of timber is much depends on its moisture content and the orientation of the timber grain; even timbers taken from the same log will have different strength. All these factors will affect the load capacity of the timber structures. Thus, methods of enhancing the timber should be developed in order to increase the load capacity of timber structures. One of the methods employed is by strengthening it with glass fibre reinforced polymer (GFRP) bonded by resin. Special adhesive should be used to bind timber and GFRP rod firmly.

Research on studying and developing the techniques for the strengthening of timber structures has been done and still ongoing. Previous researches have contributed significantly in encouraging the

usage of FRP and serve as reference for future researchers. Triantafillou & Plevris (1992) have reinforced wood beams by placing a thin unidirectional carbon fibre-reinforced sheet on the tension face of the beam. They found that the bending strength increases almost linearly with an increased quantity of fibres up to a critical value, beyond which the moment becomes almost constant. They also state that even a small amount of fibres, as low as 1% of the cross-sectional area, of thin carbon FRP bonded to wood beams can result in a strength increase in the order of 60% initially, and even higher strengths when wood compressive yield occurs. Dolan et al. (1997) have carried out research on prestressed glued-laminated timber beam. This research indicates that both strength and stiffness are increased by using small volumes of pretensioned Kevlar yarns. Apart from that, there was another research done by Fiorelli & Dias (2002) to evaluate the structural behavior of wood beams strengthened with FRP. The research was focused on the experimental and theoretical analysis of timber beams of the specie Pinus caribea var. hondurensis which were reinforced with GFRP and CFRP. The results of this research showed that the flexural stiffness (*EI*) determined experimentally was greater than the theoretical values. These values are in favor of structural safety. It shows that the increase of stiffness varied from 15% to 29% for beams strengthened with glass fibre and with carbon fibre. An other project was done by Buell & Saadatmanesh (2005). This project determined whether applying FRP composites in the form of either a fabric or laminate to timber beams would increase the load capacity of those particular beams. The results show that applying carbon fabric to the timber beams provides significant increase to the bending and shear capacity, and nominal increase in the stiffness of the beams. Statistically, the ultimate bending strength for all reinforced beams was increased between 40 - 53 %. Furthermore, those beams were held together after ultimate failure where no catastrophic failure when the beams were wrapped with carbon fabric.

The behavior of timber stringers reinforced with GFRP sheets was studied by Gomez and Svecova (2008). The stringers were reinforced for shear and bending. The proposed reinforcement leads to improvement of stiffness by 5.5 – 52.8%. The ductility of timber beams strengthened with FRP was investigated by Yang et al. (2008) and they found that the FRP reinforcement turns the brittle tensile failure to the ductile compression failure of timber beams. Alam et al. (2009) have strengthened fractured timber beams using steel and FRP. The results showed that these reinforcements are very effective in enhancing flexural strength but the CFRP reinforcement endows the greatest flexural strength. The latest inovative development of the usage of GFRP in strengthening works was conducted by Manalo et al. (2010) where composite sandwich beams were tested in the edgewise and flatwise positions. The results exhibited that the sandwich beams tested in the edgewise position failed at a higher load with less deflection compared to specimens tested in the flatwise position. In the meantime, Ferrier et al. (2010) have developed an inovative hybrid beam made of glulam and short fibre-reinforced concrete planks with or without internal reinforcement consisting of steel or FRP bars. The results showed that the hybrid beam performed higher bending stiffness and ultimate load capacity compared to that of a glulam of similar dimensions.

Timber, in general, has very little of any plasticity. Under failure load, the deflections of timber continue to increase linearly until the member breaks either with tension fibre or compression fibre splitting. At this point the member loses its load carrying capacity. This type of failure occurs with no warning and is therefore catastrophic (Gardner 1989). However, in this research it is expected that the non catastrophic failure will occur when solid timbers are strengthened using FRP.

Many researches have been done on glued-laminated timber. However, research on strengthening on solid timber is very limited. In this research, the focus is on the determination of the increase in load carrying capacity and stiffness for the solid timber beam strengthened with GFRP.

The strength and stiffness of timber member can be enhanced through strengthening process. There are many methods to strengthen the timber structure such as the application of steel plate attached to the tension side of the beam. Although the bending capacity increased but there is a disadvantage where the steel plate tends to become rusty after some period of time. With the recent technology, the FRP becomes the right choice of strengthening material.

The objectives of this research are to study the bending strength and the stiffness of the beams strengthened with GFRP and also to investigate the failure modes of the strengthened beams

This study will focus on bending behavior of the strengthened timber beams with respect to unstrengthened beam (control beam). All beams were tested under four-point loading.

2 RESEARCH SIGNIFICANCE

When a beam is strengthened at tension zone, the mode of failure for the timber structures may change from tension failures to compression failures. In other words, this method increases the tensile capacity of the structure, as well as fully utilizing the compression capacity of timber. As a result, it can be applied in new construction projects, as well as in the rehabilitation of existing timber structures. The rehabilitation and strengthening of the existing timber structures will definitely increase its load capacity and save the cost of timber structure replacement. In addition, effective strengthening techniques will also reduce the size or depth of the timber beams that are required for construction (Haiman & Zagar 2002). Bigger section of timber beams are getting difficult to be obtained, thus small section of timber beams after strengthened can have equivalent capacity as big section.

Yellow Meranti, a widely distributed wood species in Malaysia, is not a high-performance material for structural usage because of its low strength and durability. Since Yellow Meranti is cheap and used in furniture industry, research has been conducted at UTM to study the feasibility of utilizing the low-grade Yellow Meranti for structural usage by reinforcing it with FRP composites.

3 RESEARCH MATERIALS

3.1 *Yellow Meranti timber beams*

The main research material was Yellow Meranti timber beams which obtained from Segamat, Johor. The size of the beam is $100 \times 200 \times 3000$ mm. Yellow Meranti, also known as yellow seraya is the important commercial light hardwood in Malaysia which is exported as sawn timber and logs (Desch 1981). Table 2.2 in MS 544: PART 2 (2001) categorized the strength of Yellow Meranti is in strength group 6 and it requires preservative treatment before being used in construction. However, for this research, untreated Yellow Meranti was used.

Yellow Meranti has density between 575 - 735 kg/m^3 at air dry (MTC Wood Wizard 2006). The average density is 680kg/m^3 at 19% of moisture content (MS 544: PART 2: 2001). The durability of the heartwood can be classified as moderately durable while sapwood of Yellow Meranti is susceptible to powder-post beetle attack. In terms of working qualities, Yellow Meranti gives a good finish in most operations by hand and machine tools.

3.2 *Glass fibre reinforced polymer (GFRP)*

The GFRP rods were obtained from Polymer Composite Asia at Nilai, Negeri Sembilan. Three sizes of rods were used i.e. 6.35 mm, 9.53 mm and 12.7 mm.

Fibres are the main component in a fibre reinforced composite material. The primary function of fibres or reinforcements is to carry load along the length of the fibre. Glass fibres are one type of fibres that is considered strong in strength but lack in rigidity due to their molecular structure. Whereas, the polymer has low tensile strength but high rigidity. Combining these two materials will provide good composite element. GFRP is a brittle material and has low elasticity modulus when compared to steel (Shin 2003). GFRP are available in very long lengths, easy to handle and installed in construction due to its light weight. All FRP materials have the same behavior when subject to tension load. FRP will react elastically in the beginning until final brittle rupture.

3.3 *Adhesives*

The adhesive used to bond the timber and GFRP rods was Sikadur®-30 which is a product from Sika Kimia Sdn Bhd. It consists of component A which is white in color and component B which black/grey in color. By mixing these two components where the ratio of A:B is equal to 3:1 either by weight or volume for at least 3 minutes, the mixture becomes smooth in consistency and uniform grey color. It is normally used at temperature between 8°C to 35°C. Table 1 shows the characteristics of Sikadur®-30.

Table 1. Characteristic of Sikadur-30

Characteristics	Sikadur-30
Elastic Modulus (N/mm^2)	12,800 (Static)
Compressive Strength (N/mm^2)	85 – 95 (7 days at +35 ^{o}C)
Tensile Strength (N/mm^2)	26 – 31 (7 days at +35 ^{o}C)
Shear Strength (N/mm^2)	16 – 20 (7 days at +35 ^{o}C)
Shrinkage(%)	0.04
Pot life (minutes)	40 (at 35 ^{o}C)

4 LABORATORY WORKS

Tests were conducted to determine the bending behavior of strengthened and unstrengthened timber beams. The dimension of timber beam was $100 \times 200 \times 3000$ mm and seven timber beams were taken randomly. The clear span was 2700 mm. All beams were tested in accordance to American Society for Testing and Materials (ASTM: D198-27, 1992).

4.1 *Testing of moisture content of timber beams*

Since the strength of the timber beams varies with moisture content, it is necessary to determine the moisture content of each timber beam. The preferred result is that all timber beams will have the same moisture content. This is to ensure that the effect of moisture content to the strength can be eliminated. Hence the variation in strength of timber comes from the effect of strengthening. The testing of moisture content was conducted in accordance to ASTM Standards (ASTM: D 4442 – 92, 1992).

4.2 *Static bending test of Yellow Meranti specimens*

The static bending test was carried out based on BS 373:1957. The dimensions of the sample are 50 mm × 50 mm × 1000 mm. Twelve samples were prepared for this test. The distance between the supports and the distance between the applied point load and the support are shown in Figure 1.

The load was applied at such a rate that loading head moves at a constant speed of 0.13 in/min (0.1 in/min in ASTM D143-83). The sample was supported by pin and roller support so that the sample is free to follow the bending action and not be restrained by friction to avoid any longitudinal stresses due to friction force.
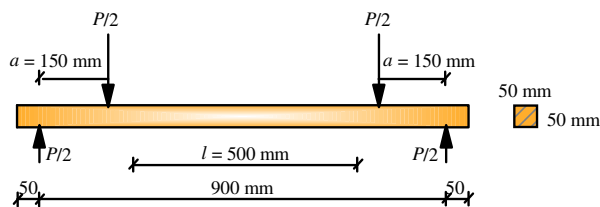

Figure 1. Static bending test: four point loading method

4.3 *Tensile test of GFRP rods*

The tensile test was carried out in accordance to ASTM D3039. The standard did not mention in details on how to prepare the samples. However, Valter et al. (2009) have suggested to use a conical polymeric head that fits a conical hole inside the anchoring device at grip heads. Nine samples were prepared for this test. The test was conducted in order to determine direct tensile strength, elasticity modulus and stress-strain relationship for GFRP rods due to the lack of information from manufacturer.

4.4 *Strengthening of timber beams*

One of the beams was not strengthened and this becomes control beam. Another six beams were strengthened using different area of GFRP. Figure 2 shows

the cross section for all the specimens strengthened with GFRP rods. For control beam, the code is C-B whilst for the beam strengthened with GFRP rods with diameter of 6.35 mm, 9.53 mm and 12.70 mm, the codes are GR-635-1B, GR-953-1B and GR-127-1B for one rod and GR-635-2B, GR-953-2B and GR-127-2B for two rods.

Each timber beam will be cut to prepare one or two grooves along the bottom part of the beam which is parallel to grain. The sizes of the groove for each beam are not equals depending on the diameter of the rods. The purpose is to minimize the amount of Sikadur®-30.

The grooves were cleaned with small broom to remove the dirt. This step is important to ensure a good bonding exists between GFRP rods and timber. After that, the GFRP rods from each size were cut into 3 m length. Then the strain gauge of BFLA-5-3L was attached at the mid-length of each GFRP rod using special adhesive called cyanoacrylate (CN); a product from TML, Japan. A coating material (N-1) was used to protect the strain gauge from direct contact with Sikadur®-30. The first layer of Sikadur®-30 was applied inside the grooves and the GFRP rods were placed to fit the groove; eventually another layer of Sikadur®-30 was placed to completely cover the GFRP rods.
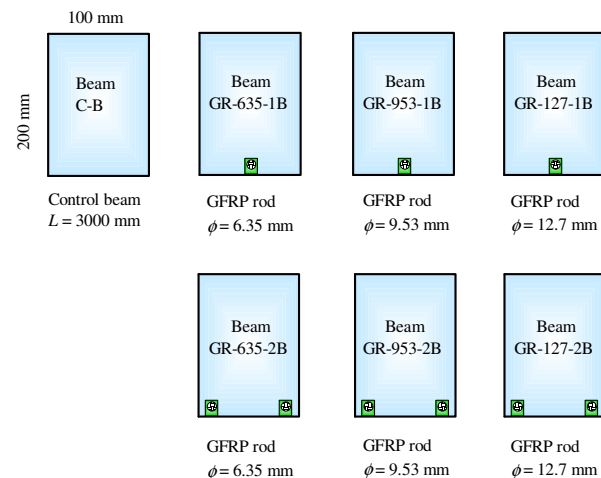

Figure 2. Cross section of beam strengthened with GFRP rods

All beams will be put aside for more than seven days for curing process to ensure the bonding between FRP and timber is well established. After seven days, six strain gauges were attached at mid-span along the depth of each timber beam. The typical samples of the timber beam strengthened with GFRP rods are illustrated in Figure 3.

Figure 3. Timber beam strengthened with GFRP rods

4.5 *Bending test of timber beams*

The control beam was tested first until failure and then followed by the strengthened beams. All timber beams were tested under four-point loading where the half shear span to depth ratio (*a/h*) was 6, which is between 5 and 12 as recommended by ASTM D198 (1992). The clear span for the timber beam was 2700 mm. The deflections of all the beams were measured using three linear variable displacement transducers (LVDT) which were placed below the beams. Figure 4 shows the setting of the testing. All beams were laterally braced in order to prevent from lateral torsional buckling. The beams were tested at Structures and Materials Laboratory, Civil Engineering Faculty, UTM.

A plywood of 100 mm length, 100 mm width and 16 mm thick and a rubber pad of 100 mm length, 100 mm width and 5 mm thick were used as bearing plate before a steel roller was put on the rubber pad. The plywoods were rounded corners to minimize stress concentration in test specimens and to obtain a relatively uniform load level under the loading plates, as suggested by Lam & Craig (2000)**.** This arrangement is very important as the preliminary testing on beam showed that the timber beam experienced crushing due to local bearing stress which comes from point load. This local bearing stress generated further compressive stress at compression zone in the timber beam. Load spreader made by short steel beam of 0.6 m long and 0.15 m height was put on steel rollers where the distance between them was 0.3 m. A steel plate of 100 mm $\times$ 100 mm $\times$ 10 mm thick was put on the load spreader followed by a load cell and steel plate. An auto hydraulic jack was used to apply the load where it was attached to a steel beam and then supported by steel column.

The testing rig was Magnus Frame which was self-reacting frame built-up with steel channel section. The sample was supported on two steel bases through lubricated pin and roller support. The whole system of this Magnus Frame was anchored and erected on strong floor. The capacity of the Magnus Frame was 200 kN.

A loading rate of 2.0 kN/min was applied to each timber beam until failure. The maximum load that the beam can sustain before failure was known as ultimate bending load. When the timber beam failed, the shape for cracking was drawn on a paper. Immediately after the test, samples from that particular timber were taken to determine its moisture content.
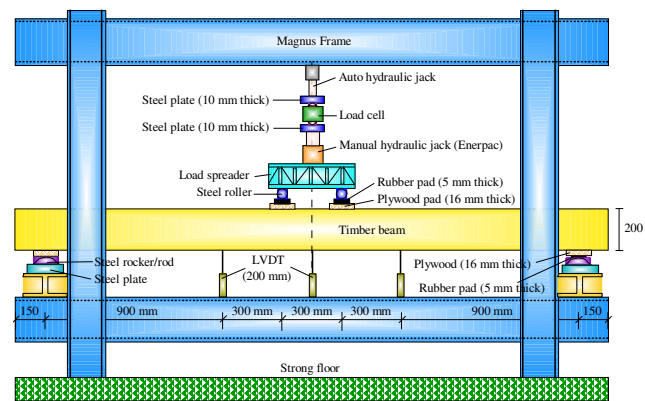

Figure 4. Configuration for bending test

5 RESULTS AND ANALYSIS

All the results obtained from lab work were analyzed and discussed. The strengthened beams were compared with control beam in order to determine the behavior of strengthened timber beams in terms of strength and stiffness. In addition, the modes of failure for all beams were also discussed.

5.1 *Moisture content of timber*

The moisture content of the timber beams ranged between 23 to 65% with the average is 35.1% upon the arrival of the timber beams. These results indicate that the moisture content of the beam is not consistent. Subsequently, all the timber beams were put into a cabin with temperature around 40°C to 50°C for one month in order to reduce the moisture content. At low temperature but long exposure to this temperature allows the timber beams dry slowly without defect from drying process. The beams were put aside at room condition to stabilise its moisture content for one month. Then the moisture content of the beam was tested. The results show that the moisture

contents are almost consistent and the average value was reduced to more or less 16%.

5.2 Bending strength of Yellow Meranti specimens

Twelve specimens were tested successfully and Figure 5 shows the relationship between load and deflection at mid-span. The following properties were computed and the details results were presented in Table 2.

Fibre stress in the wood at proportionality limit,

$$\sigma_p = \frac{3P_p a}{bh^2} \quad (1)$$

Fibre stress in the wood at maximum load,

$$\sigma_u = \frac{3P_u a}{bh^2} \quad 2)$$

Bending modulus of elasticity (Gere and Timoshenko, 1984),

$$E = \frac{P_p a(3L^2 - 4a^2)}{48\Delta_p I} \quad (3)$$

where b = width of beam (= 50 mm), h = depth of beam (= 50 mm), a = distance between point of application of load and support (= 150 mm), L = span length (= 900 mm), P_p = load at proportionality limit, P_u = ultimate load, Δ_p = deflection at mid span at proportionality limit, I = Moment of inertia (= $bh^3/12$).

The proportionality limit was taken as that point in the load-deflection curve at which the curve deviates from the straight line. Generally, the specimens behaved bilinearly where their stiffnesses were reduced when the beams start to crack or crush until total failure occurred.
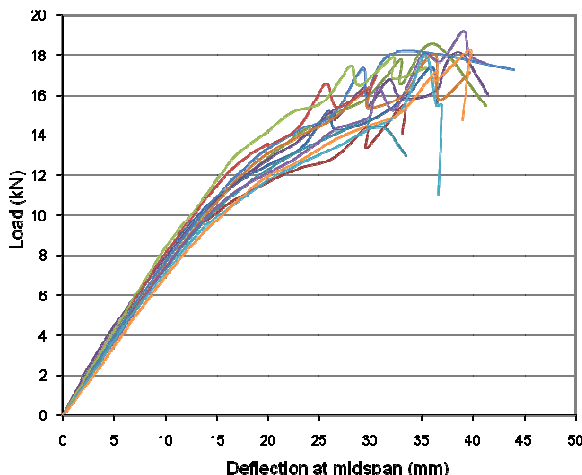
Table 2. Bending strength and elasticity modulus

Sample	P_p (kN)	Δ_p (mm)	P_u (kN)	σ_p (N/mm^2)	σ_u (N/mm^2)	E (kN/mm^2)
1	8.0	9.90	17.4	28.80	62.64	11.345
2	7.4	9.93	15.5	26.64	55.80	10.463
3	7.6	9.90	18.6	27.36	66.96	10.778
4	7.8	9.90	18.1	28.08	65.16	11.062
5	7.1	9.65	14.9	25.56	53.64	10.330
6	7.4	9.94	18.0	26.64	64.80	10.452
7	7.6	9.87	18.2	27.36	65.52	10.811
8	8.0	9.88	17.0	28.80	61.20	11.368
9	8.6	10.2	17.4	30.78	62.64	11.769
10	7.3	9.88	19.2	26.28	69.12	10.374
11	7.0	9.85	18.0	25.20	64.80	9.978
12	8.9	13.2	18.3	32.04	65.88	9.466
		Average		27.80	63.18	10.683

The results for bending stress at proportionality limit, bending stress at ultimate limit and elasticity modulus are almost consistent for all specimens where the average values are 27.8 N/mm^2, 63.18 N/mm^2 and 10.7 kN/mm^2, respectively.

5.3 Stress-strain curves for GFRP rods

Tensile test for GFRP rods was undertaken since the supplier did not provide any data for mechanical properties of the rods. The data was analysed and interpreted into stress-strain curve to find the ultimate load, maximum stress and strain, and elastic modulus of GFRP rods. Figure 6 shows the stress-strain curves for all samples of the rods. From these curves, it is concluded that the GFRP rods behave almost linear elastic under tensile load. Due to the brittleness of GFRP rods, all rods experienced a sudden failure immediately after reach their ultimate tensile strength. The slope of the stress-strain curve represents the elastic modulus of GFRP rods. All samples failed within the gauge length region and no slip occurred at the grip heads thus the results were valid.
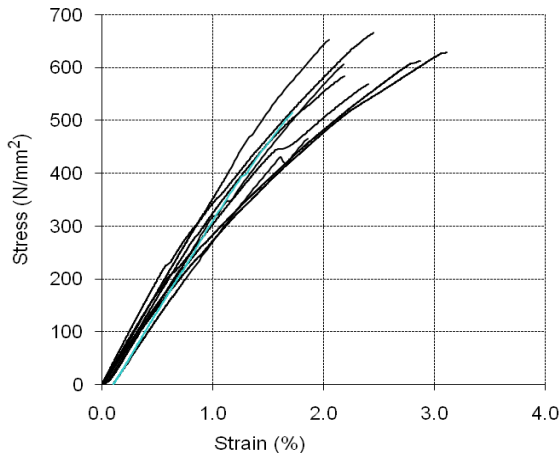

Figure 5. Load-deflection curves of static bending test

Figure 6. Stress-strain curves of GFRP rods

Table 3. Results of tensile test on GFRP rods

Rod	Sample	Max. load (kN)	Max. stress (N/mm^2)	Max. strain (%)	Young's modulus (N/mm^2)
	Sample 1	17.9	568.3	2.40	28262
ϕ 6.35	Sample 2	17.2	583.6	2.19	33342
	Sample 3	20.1	665.4	2.45	33545
	Sample 1	43.0	612.3	2.87	26540
ϕ 9.53	Sample 2	44.6	628.9	3.11	25855
	Sample 3	43.2	606.0	2.18	33396
	Sample 1	82.7	653.0	2.05	35981
ϕ 12.7	Sample 2	58.9	464.8	1.86	28970
	Sample 3	65.3	515.3	1.72	34534
	Average		588.6	2.3	31158
	SD		64.9	0.45	3754

The tensile strength was calculated using the following equation:

$$\sigma_u = \frac{F_u}{A} \tag{4}$$

where σ_u = tensile strength, F_u = maximum tensile force, A = cross-sectional area

The tensile rigidity and Young's modulus were calculated using the following equation:

$$EA = \frac{\Delta F}{\Delta \varepsilon} \text{ and } E = \frac{\Delta F}{A \times \Delta \varepsilon} \tag{5}$$

where E : = Young's modulus, EA = tensile rigidity, ΔF = difference between loads at 20% and 50% of maximum tensile force, $\Delta \varepsilon$ = strain difference between ΔF

The ultimate strain was taken as the strain corresponding to the ultimate tensile capacity. It can also be calculated from the following equation:

$$\varepsilon_u = \frac{F_u}{EA} \tag{6}$$

where ε_u := ultimate strain

The detail results are shown in Table 3. The average value for stress and strain is 588.6 N/mm^2 and 2.3%, respectively. This result is in good agreement with the result obtained by Yatim (2002) where the GFRP rods failed at a strain between 1.5 – 2.4%. As a comparison, the ultimate stress and strain for mild steel material is 275 N/mm^2 and 0.2%, respectively. Hence GFRP rods have higher ultimate stress and strain than steel.

The result shows that the average value of elasticity modulus is 31158 N/mm^2, which is within the range of 22000 – 50000 N/mm^2 (Giap 2007).

5.4 *Bending behavior of timber beams*

The results of load-deflection curves for all beams are shown in Figure 7. Initially, all the beams behaved linearly elastic, and as the loads were increased, the beams exhibited non-linearity until failure. There are some slight drops observed in the curve as a result of small cracking during testing.
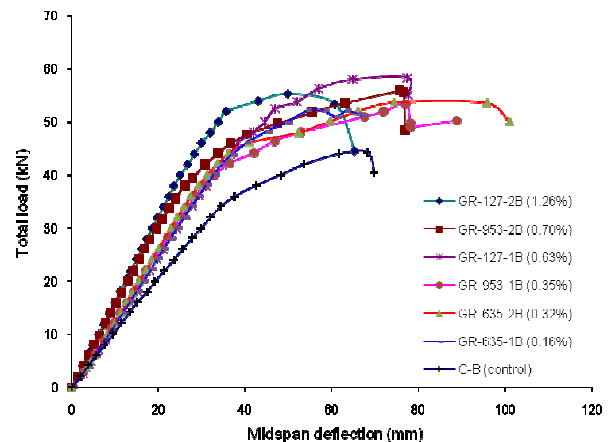

Figure 7. Load-deflection curves for all tested beams

Figure 7 clearly indicates that all the strengthened beams have higher ultimate load carrying capacity than the control beam. When there is significant

failure i.e. crushing or simple tensile crack occurred on the beams, the corresponding load will be taken as ultimate load carrying capacity. Besides that, the strengthened beams with GFRP also indicated enhanced flexural stiffness. In other words, the strengthened beams experienced lower deflection than the unstrengthened control beam at the same load level. This low deflection phenomenon is desirable in the aspect of serviceability limit state in design to ensure comfort ability of timber structures. In timber design, generally the deflection will govern the design. For beam where clear span is 2700 mm, the allowable deflection is 8.1 mm. For all beams the failure occurred after the deflection is more than 30 mm. At this stage, the beams were already considered as failed. Hence, the bending capacity of the timber beams is not fully utilised. Therefore if the timber beam has high bending capacity but low stiffness, it will not enhance the timber structure in structural design point of view.

5.5 *Ultimate load carrying capacity*

The ultimate load carrying capacity of all beams and their corresponding deflections are shown in Table 4.

The strength of timber beams increased as the percentage of FRP used is increased except beam GR-953-2B and GR-127-2B. This result is unexpected since the beam strengthened with larger area of GFRP rods should have a higher ultimate load carrying capacity. After an observation, it was deduced that the beam used for GR-953-2B might be weaker due to the existence of sapwood along its length and parallel to its grains. This is the only beam which failed due to shear along the interface between sapwood and heartwood. In the meantime, beam GR-127-2B had an initial crack at its cross section at both ends and also a knot at one of vertical faces.

Table 4. Results summary of bending test

Beam	Area of GFRP (%)	Stiffness (E) (kN/mm²)	P_{uls} (kN)	δ_{uls} (mm)	% increase in E	% load increase in ULS
C-B		6.079	44.3	68.31	0.0	0.0
GR-635-1B	0.16	7.547	52.7	63.34	24.1	19.0
GR-635-2B	0.32	8.030	53.7	74.47	32.1	21.2
GR-953-1B	0.35	7.819	53.3	77.08	28.6	20.3
GR-127-1B	0.63	7.595	58.3	77.47	24.9	31.6
GR-953-2B	0.71	9.329	55.8	75.62	53.5	26.0
GR-127-2B	1.27	9.752	55.3	49.91	60.4	24.8

It is believed that even if the defects did not exist, the increment in the ultimate load of these two beams are expected to be very small since this beam strengthened with 0.63% of GFRP was already over-designed where the ultimate load was governed by compression capacity. Referring to Table 4, the ultimate load increased between 19% – 31.6% when the timber beams were strengthened using GFRP rod for area between 0.16% – 0.63 %. Generally, a small percentage of GFRP (0.16%) is sufficient to enhance the bending capacity of timber beams; hence the cost of the strengthening can be reduced.

5.6 *Stiffness*

The straight line at the initial part of the curves shown in Figure 7 indicated that the timber beam behaves linearly elastic. For each curve, the loading data up to 30 kN were taken for determination of stiffness to ensure the data within that region is totally linear. Linear regression analysis for that region was carried out to determine the gradient of the slope.

The bending modulus of elasticity was calculated from the following equation:

$$\Delta = \frac{Pa}{24EI}\left(3L^2 - 4a^2\right) \tag{7}$$

$$E = 6042.3m \tag{8}$$

where *a* is the half shear span, P/Δ is the gradient of load-deflection curve which can be replaced by *m*.

The stiffness, *E* is shown in Table 4. Generally, the stiffness was increased as the percentage of GFRP rods increased. It seems that strengthening using one rod was not very efficient to enhance the stiffness. The effect of number of rods can be confirmed by comparing beam GR-635-2B and GR-953-1B where both beam had almost the same percentage of GFRP area. The ultimate loads are similar as expected but the stiffnesses are different. It was found that the increase in stiffness for beam with two rod (32.1%) is higher than that of beam with one rod (28.6%). In overall conclusion the increase in stiffness for strengthened beams were between 24% - 60% when the timber beams were strengthened using GFRP rod for area between 0.16 – 1.27%.

5.7 *Strain distribution diagram*

The strain distributions across the depth of the beam at every 10 kN load increment and at failure load are shown in Figure 8. From the investigation, it is obvious that the strain distribution is linear as the load

increased and when the strain at the top fibre reached about 0.3% the compression zone starts to crush. From the compression test that has been carried out on Yellow Meranti specimen, it was found that the maximum compressive strain from direct compressive test was 0.83%. However, this value cannot be used to determine whether the compression zone has failed or not because the compressive strain value from direct test is not similar to the compressive strain due to flexure.

For the tension zone, the beams start to crack when the flexural tensile strain is more than 0.6% and again this value is less than the maximum tensile strain from direct tensile test which is 0.87%. Although the maximum tensile and compressive strain are almost same but the values are different when the beam is tested under bending. From the flexural strain values as well as from the crack or crush pattern, the failure mode is identified as to fail in tension or compression zone, whichever come first.

The strains at first crack or crush load for all beams are shown in Table 5. These strains will determine the failure mode of the beams. This failure modes have been confirmed by observing the crack or crush pattern during testing. Generally, the tensile strains were decreased as the percentage of GFRP rod increased. It shows that the present of GFRP rod was able to reduce the tensile strain (maximum reduction was 47%) and increased the compressive strain (maximum increment was 54%) in the timber beams. Thus, the tension zone of timber beams was successfully strengthened if the percentage of GFRP is greater than 0.35%. Above this value, the failure is controlled by compression zone and the ultimate load was not increased significantly unless the compression zone is strengthened. Although there is a deviation in the results, it is believed that it was due to the nature of timber itself. However, this result can only be validated through further investigation.
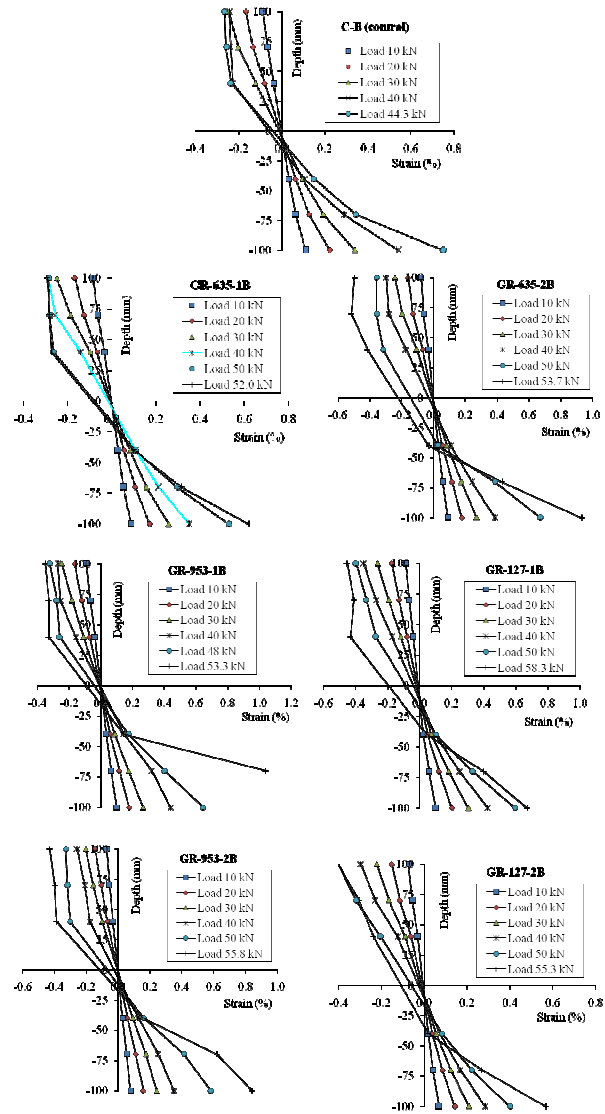

Figure 8. Strain distribution diagrams for all beams

Table 5. Strain at first crack or crush load

Beam	Area of GFRP (%)	Tensile strain (%)	Compressive strain (%)	Failure type based on strain value
C-B		0.75 > 0.60	0.26 < 0.30	Tensile crack
GR-635-1B	0.16	0.62 > 0.60	0.29 < 0.30	Tensile crack
GR-635-2B	0.32	0.67 > 0.60	0.36 > 0.30	Tensile crack and crushing
GR-953-1B	0.35	0.65 > 0.60	0.32 > 0.30	Tensile crack and crushing
GR-127-1B	0.63	0.43 < 0.60	0.35 > 0.30	crushing
GR-953-2B	0.71	0.58 < 0.60	0.33 > 0.30	crushing
GR-127-2B	1.27	0.40 < 0.60	0.40 > 0.30	crushing

5.8 *Mode of failure*

Table 6 shows the details of mode of failure. Control beam failed in bending with simple tensile crack which shows that the tension capacity is less than the compression capacity for an unstrengthened timber beam. Beam GR-635-1B failed with the same failure mode as control beam, which indicates that the reinforcement is still inadequate to increase the tension capacity more than compression capacity. Hence the beams can be considered as under-reinforced.

Table 6. Modes of failure for all timber beams

For beam GR-635-2B, minor crushing occurred shows that the beam was successfully strengthened by GFRP rod of 0.32%. Moreover, the beam failed in crushing followed by simple tensile crack failure. Since the failure in the tension and compression zone occurred almost simultaneously, the beams can be considered as balance-reinforced. Beam GR-953-1B failed with similar pattern to beam GR-635-2B as expected since the area of GFRP for beam GR-953-1B which is 0.35% is almost close to the area of beam GR-635-2B. The other beams show crushing failure occurred first before tension failure. This phenomenon shows that any further increases of GFRP area more than 0.35% yields to be over-reinforced beam. Horizontal shear failure occurred in beam GR-953-2B before the ultimate load was achieved and this phenomenon is due to the separation between heartwood and sapwood. Otherwise beam GR-953-2B is expected to fail in pure bending. Regardless how the beam failed, the GFRP rods were still strongly intact with the timber for all beams and the adhesive was not cracked. Although few cracks were observed on the adhesive but it caused by the wood splitting.

For all beams, before the total failure occurs, sounds of cracking are heard. This characteristic has make timber a suitable construction material that show signs prior to failure. Strengthened timber beams behaved elasto-plastically and held together with the reinforcement after failures occurred.

6 CONCLUSIONS

All Yellow Meranti timber beams were successfully tested. Timber beam strengthened with GFRP rods had an increase in its ultimate load carrying capacity. The percentage of increase is between 20 % - 30 %. The strengthening of timber beams with GFRP enhanced the stiffness of the beam with a percentage of increase between 24% - 60%. No debonding or delamination occurred between GFRP rods and timber

beams. It shows that the load carrying capacity is totally dependent on the strength of timber and GFRP. The failure mode was governed by the strength of timber beams since no rupture occurred to the GFRP rods. Sikadur®-30 showed good performance as a bonding agent between GFRP rods and timber beams. The timber beams failed in tension when the flexural tensile strain is more than 0.6% whilst the beams failed in compression when the flexural compressive strain is more than 0.3%. The failure mode of the strengthened timber beams changed from tension failure to compression failure when the percentage of GFRP rod is greater than 0.35%. The beam is

Beam	Area of GFRP	Failure mode
C-B		Failed in bending with simple tensile crack (under-reinforced)
GR-635-1B	0.16%	Failed in bending with simple tensile crack (under-reinforced)
GR-635-2B	0.32%	Crushing and simple tensile crack occurred simultaneously (balance-reinforced)
GR-953-1B	0.35%	Minor crushing followed by simple tensile crack (balance-reinforced)
GR-127-1B	0.63%	Crushing followed by simple tensile crack (over-reinforced)
GR-953-2B	0.71%	Crushing followed by simple tensile crack and lastly sudden horizontal shear failure occurred (over-reinforced)
GR-127-2B	1.27%	Crushing followed by simple tensile crack (over-reinforced)

categorized as under-reinforced, balance- reinforced, and over- reinforced if the percentage of GFRP rod is less than 0.32%, between 0.32% - 0.35%, and greater than 0.35%, respectively. The strengthening of timber beam with GFRP rods is a suitable method that can be applied in either new construction or rehabilitation of existing timber structure. However, more tests are required to validate this result before it can be used in design.

Further research on the use of CFRP plate to strengthen the timber beams is recommended since the strength of CFRP is higher than GFRP and also easy to attach to timber beams without the need of the grooves. Although CFRP is another strengthening material, one must consider the cost because they are quite expensive.

ACKNOWLEDGEMENTS

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