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Schober, K.-U., Harte, A. M., Kliger, R., Jockwer, R., Xu, Q., & Chen, J.-F. (2015). FRP reinforcement of timber structures. *Construction and Building Materials*, *97*, 106-118. https://doi.org/10.1016/j.conbuildmat.2015.06.020

Published in:
Construction and Building Materials

Document Version:
Peer reviewed version

FRP reinforcement of timber structures

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Abstract

Timber engineering has advanced over recent decades to offer an alternative to traditional materials and methods. The bonding of fibre reinforced plastics (FRP) with adhesives to timber structures for repair and strengthening has many advantages. However, the lack of established design rules has strongly restrained the use of FRP strengthening in many situations, where these could be a preferable option to most traditional techniques. A significant body of research has been carried out in recent years on the performance of FRP reinforced timber and engineered wood products. This paper gives a State of the Art summary of material formulations, application areas, design approaches and quality control issues for practical engineers to introduce on-site bonding of FRP to timber as a new way in design for structural repair and rehabilitation.

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1. Introduction

Fibre reinforced polymer materials combining high strength fibres and a resin matrix have a wide variety of industrial applications due to their high strength-to-weight ratio and ease of handling. Their versatility is reflected in the construction industry where they have been widely used for many years, especially for strengthening of concrete structures [1],[2]. More recently, the techniques have also been extended to timber structures [3]-[6].

FRP reinforcement in the form of pultruded rods or plates and woven fabrics has an expanding and particularly effective application in structural repair of timber structures. They may be inserted at critical locations for enhanced loading capacity, or for load transmission when damaged beam-ends are cut off and replaced with new timber or epoxy mortar. The reinforcement techniques are usually based on the use of adhesives on site, and use procedures that are common for the repair or the upgrading of concrete and metallic structures. Such techniques minimize the disturbance to the building and its occupants during the intervention. However, some concerns have prevented the wider use of adhesives, particularly in historical timber structures, where sufficient reliability cannot yet be guaranteed. One reason is that a long service life has not yet been fully proven for synthetic adhesives, since the oldest bonded joints are only around sixty years and greater ages cannot be simulated by existing accelerated ageing tests [7].

Fibre reinforced polymers and structural adhesives have been used to repair or strengthen structural members for many years. This approach is nevertheless difficult to extend to less common wood species, less favourable environmental conditions, new adhesive formulations and other variables. The lack of established design rules available to engineers and other decision makers has significantly restrained the use of FRP strengthening techniques in many situations where these could otherwise be a preferable option to most traditional techniques or to the total replacement of timber members.

In this chapter, the materials used in FRP reinforcement of timber structures are discussed, a design approach for bondline delamination is presented, current and potential applications of FRP for reinforcement of timber structures are described, design rules are outlined and finally relevant quality control procedures for on-site bonding are summarised.

2. Materials

2.1 FRP reinforcement materials

FRP materials are composites comprising fibres that provide the load-bearing capacity and stiffness, embedded in a polymeric resin that transfers loads between fibres and provides protection for the fibres. They are available in a wide variety of forms, and have properties that vary considerably depending on the fibre material, volume fraction and orientation. Typical properties of the common fibres and polymers are given in Tab. 1.

Tab. 1 Fibre and polymer properties [1],[8]

Material	Modulus of Elasticity [GPa]	Tensile Strength [MPa]	Failure Strain [%]	CTE [10^{-6} °C^{-1}]	Density [g/cm^3]
E-glass	70-80	2000-4800	3.5-4.5	5.0-5.4	2.5-2.6
Carbon (HM)	390-760	2400-3400	0.5-0.8	-1.45	1.85-1.90
Carbon (HS)	240-280	4100-5100	1.60-1.73	-0.6 - -0.9	1.75
Aramid	62-180	3600-3800	1.9-5.5	-2.0	1.44-1.47
Basalt	82-110	860-3450	5.5	3.15	1.52-2.7
Polymer	2.7-3.6	40-82	1.4-5.2	30-54	1.10-1.25

CTE: Coefficient of thermal expansion; HM: High modulus; HS: High strength

For structural reinforcement, two main forms of FRP are generally used, namely, pultruded rods or plates and fabrics. For internal reinforcement, pultruded rods and plates are bonded into slots or grooves formed in the timber element. For external reinforcement, FRP plates or fabric materials are used.

2.2 Adhesives

The reinforcement of timber with FRP is normally implemented by adhesive bonding. For bonding of pultruded FRP plates, the adhesive is applied to the timber substrate and the FRP is then applied to the adhesive under pressure. FRP fabric reinforcement is generally applied using a wet lay-up method, whereby the fabric is impregnated with the adhesive first and then this is applied to the timber.

Epoxy based adhesives have been used in most cases for on-site repair jobs, most formulations were developed for other materials. These adhesives are generally too rigid for bonding timber and there is

no chemical bonding or suitable mechanical anchorage in wood. The bond surface is prone to failure because of dimensional changes in the wood induced by moisture content variations, even for indoor applications such as those defined under Eurocode 5 Service Class 2. More recently, epoxy formulations have been developed specifically for use with wood. On-site application of adhesives may be difficult and the quality of adhesive bond is not easy to evaluate. Since properties of reinforced elements very much depend on construction quality, relevant procedures for applying and controlling adhesive quality[10],[11],[12] have to be followed.

The selection of the adhesive for bonding of FRP to timber has to be undertaken with great care. The adhesive must be capable of bonding with both the FRP and timber and should have adequate strength. Five categories of adhesive are available: epoxies, polyurethanes, polyesters, phenolics and aminoplastics [12]. While many of these adhesive types have been shown to provide satisfactory bonding performance when used in a controlled environment [13],[17], two-part cold-cure epoxy adhesives have generally been found to be most suitable for on-site bonding as they have good gap-filling properties, are thixotropic and have low curing shrinkage. Additional information on adhesives for on-site bonding: characteristics, testing, applications can be found in [18].

A wide variety of epoxy formulations is available and compatibility with the adherents must be verified by the manufacturer. On site, the moisture content and the surface quality of the timber are difficult to control and it may not be possible to achieve the desired bonding pressure. Thick bond lines in the order 1-3 mm are common. Careful surface preparation is essential in order to achieve good bond strength and durability. Surfaces to be bonded should be dry, free from contaminants, such as release agents and dust, and have sufficient surface roughness. For the FRP this involves abrasion followed by solvent cleaning or else removal of the peel ply if one is provided. Surface preparation should be carried out immediately prior to bonding while preparation and application of the adhesive should be in accordance with the manufacturer's instruction. The importance of using experienced operatives cannot be underestimated. Quality control measures should be implemented at each stage of the process [19].

2.3 Bond behaviour of FRP-timber interface

Central to successful reinforcement is the integrity of the bond between the FRP and timber substrate [1],[20]. Over the last two decades, a number of studies were undertaken by different researchers on

bond behaviour of FRP-to-wood and many useful results were acquired, including failure modes, stress distribution and local bond-slip relationship.

In the early studies on the behaviour of FRP to wood bonds, the test configuration, shown in Fig. 1(a), was a modified form of the ISO 6238 and ASTM D905-03 block shear test [13],[14],[21]-[24]. In those tests, the shear strength of the bond was derived as an average stress over the bonded plate length. As the surface of the FRP was sandwiched between two pieces of timber, FRP plate surface strains were difficult to monitor, not to mention the shear stress distribution and bond-slip responses.

SILVA et *al*. [25] conducted four-point bending tests on timber-FRP joints with the strengthening techniques of near-surface mounted (NSM) and externally bonded reinforcement (EBR) which was convenient to get the FRP strain distribution, shear stress distribution and bond-slip responses. On the basis of SILVA's tests, JUVANDES and BARBOS [26] analyzed the maximum anchor strength of the composite and the maximum composite strain, and proposed the effective bonding length for the EBR and NSM reinforcements.

WAN et *al*. [27] introduced single-lap FRP-to-timber joint shear tests, shown in Fig. 1(b), in which the strength of the bond between the FRP and timber was examined. The shear tests were conducted on softwood (Pine) which was strengthened with carbon FRP with the main test variables being the FRP bond length and the growth characteristics of the timber. Extensive strain gauging of the FRP has enabled the onset and propagation of debonding cracks to be monitored. Failure modes of the joints and effective bond length were identified based on the test results. WAN et *al*. [20] also conducted a series of tests on 86 single-shear FRP-to-timber joints to study on FRP-to-timber bonded interfaces. The test parameters included adhesive type, FRP plate type and timber species. Test results showed that all softwood joints failed predominantly in the timber while the hardwood joints exhibited failure at different interfacial positions. Load-slip, strain and bond stress distribution, and bond stress-slip responses were consistent with those of FRP-to-concrete bonds. A test-based theoretical bond stress-slip law utilizing the *J*-integral method was proposed and could be implemented in analytical and numerical models.

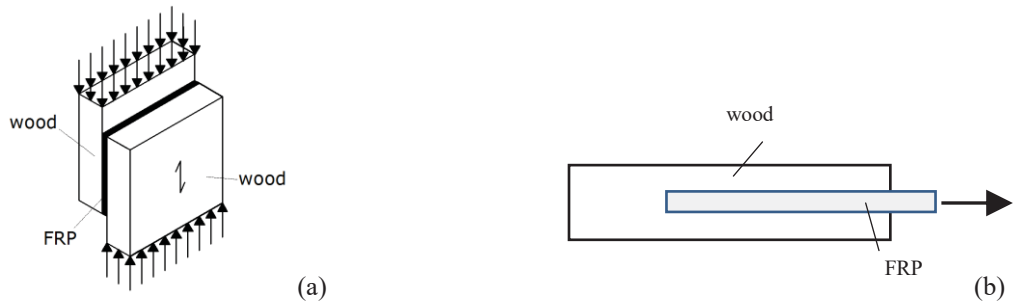


Fig.1 (a) Shear block test configuration; (b) single lap joint shear test

The FRP composites mentioned above were all FRP sheets or strips. There was much less research on bond behaviour of wood and FRP bars. LORENZIS et *al*. [28] conducted pull-out tests to study the bond performance of FRP rods epoxied into glulam timber. The test variables were bonded length, surface configuration of the rod and direction of the wood fibres with respect to the longitudinal axis of the joint. The observed failure modes were cohesive failure in timber for rods glued-in parallel to the grain and adhesion failure for rods glued-in perpendicular to the grain.

In addition to the experimental investigations, finite element simulation on bond behaviour of FRP-to-timber was developed in recent years. VALIPOUR and CREWS [29] proposed a novel force-based element in the framework of the total secant approach for nonlinear analysis of timber beams strengthened with FRP sheet (bar), including bond-slip effects. The formulation takes account of material nonlinearities and preserves the continuity of slip shear. Further, a composite Simpson integration scheme with a finite difference scheme was employed to calculate the bond shear forces along the element. It was concluded that for the considered cases, bond shear-slip between the FRP sheet (bar) and timber has a minor effect on the ultimate loading capacity as well as the load-deflection response of timber beams strengthened with FRP sheets (bars). The assumption of perfect bond between the FRP and timber beam was observed to be acceptable in most cases.

In summary, the research on the bond behaviour of FRP-to-timber is still in its infancy. Further investigations should be carried out to determine the influence of the various factors affecting the bond between FRP and wood including timber properties and specimens, thicknesses (diameters) and specimens of FRP sheets (bars), and thickness of adhesive. The local bond-slip relationship for FRP-to-timber which can be directly used for finite element analysis is definitely the research focus in the future.

2.4 FRP damage and delamination by loads

2.4.1 Global fracture criteria for composite design

A realistic design approach to account for the fracture and delamination in FRP strengthened timber structures needs appropriate material models describing physically-based failure criteria of the anisotropic and non-linear properties of these composite structures. Global fracture criteria with complete stress interaction were the first failure criteria developed [30]. The main advantage is the ease of use for analytical, numerical and design approaches due to a single scalar equation of failure for unidirectional laminates. These hypotheses contain no information about the fracture mode of composites in three-dimensional stress states; the fibre-parallel fracture plane is unknown. Stress combinations like (τ_{21}, σ_2) and (τ_{31}, σ_3) are set to equal and do not consider the importance of loads perpendicular to the fibre within the fibre plane which are critical due to the dimensions of most structural elements.

The most common criterion used in FE-codes has been developed by TSAI and WU, shown in Eq. (1). The symmetry of material properties is considered by strength coefficients which can be easily transformed in their invariance's.

$$\begin{aligned} F = \; & \frac{D_0}{F_{11}}\sigma_1 + \frac{D_{90}}{F_{22}}\sigma_2 + \frac{D_{90}}{F_{22}}\sigma_3 + \frac{\sigma_1^2}{F_{11}} + \frac{\sigma_2^2}{F_{22}} + \frac{\sigma_3^2}{F_{22}} \\ & + F_{12}\sigma_1\sigma_2 + F_{12}\sigma_1\sigma_3 + 2\,F_{23}\sigma_2\sigma_3 + \frac{\tau_{12}^2}{f_{v,90,0}^2} + \frac{\tau_{13}^2}{f_{v,90,0}^2} + \frac{\tau_{23}^2}{f_{v,90,90}^2} = 1 \end{aligned} \tag{1}$$

with

$$F_{11} = 1/\left(f_{t,0}\,f_{c,0}\right) \qquad F_1 = 1/f_{t,0} - 1/f_{c,0}$$

$$F_{22} = 1/\left(f_{t,90}\,f_{c,90}\right) \qquad F_2 = 1/f_{t,90} - 1/f_{c,90}$$

$$F_{12} = 0 \qquad 2\,F_{23} = \frac{2}{f_{c,90}\,f_{t,90}} - \frac{1}{f_{v,90,90}^2}$$

$$D_0 = f_{c,0} - f_{t,0} \qquad D_{90} = f_{c,90} - f_{t,90}$$

$f_{t,0}$ tensile strength in fibre direction

$f_{c,0}$ compressive strength in fibre direction

$f_{t,90}$ tensile strength perpendicular to fibre direction

$f_{c,90}$ compressive strength perpendicular to fibre direction

$f_{v,90,0}$ shear strength in laminate area

$f_{v,90,90}$ shear strength perpendicular to laminate area

σ_1, σ_2, σ_3 are normal stresses and $\tau_{23} = \tau_{32}$, $\tau_{13} = \tau_{31}$, $\tau_{12} = \tau_{21}$ shear stresses related to the (x_1, x_2, x_3) coordinate system of the unidirectional laminate area, where (x_1, x_2)-area describes the laminate area and x_3 the thickness direction (Fig. 2).

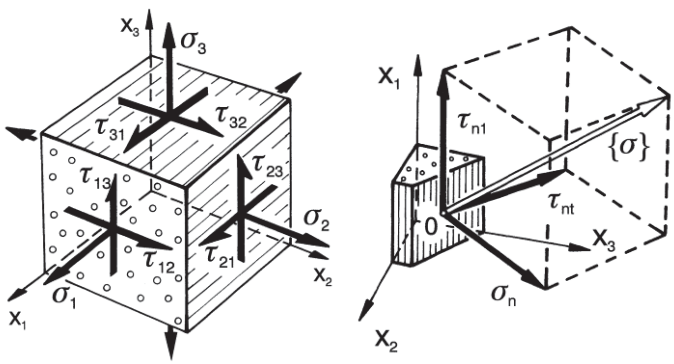


Fig. 2 Stresses in unidirectional laminates

With this hypothesis most of fracture modes can be described but fibre fracture (FF) and interlaminar fibre fracture (IFF) cannot be identified. Furthermore, the strength increase cannot be represented by a maximum criterion but only by an interactive criterion. Therefore, for FRPs the combination of the general Tsai-Wu criterion with the hypothesis of maximum stresses is recommended, otherwise stress states greater than the single value of the strength will be accepted.

2.4.2 Failure-mode based strength criteria for composite design

Each failure mechanism is governed by one mode-associated strength. Failure conditions are required to be simply formulated, numerically robust, and physically-based and to allow a simple determination of the highest mode stress effort (design driving). The most important investigations have been made by PUCK [31] in a further development of the classical MOHR theory and HASHIN's failure criteria. The key statement in his proposal is that failure in a fibre parallel fracture plane under tension perpendicular to the plane is caused by the tension stresses and also by the shear stresses τ_{nt} and τ_{n1} (see Fig. 2), while compression forces perpendicular to the fibre plane increase the shear resistance. A further enhancement of this criterion was the introduction of an additional term of the internal COULOMB friction to account for stress stiffening below the fracture limit due to compression. For the case of tension reinforcement, the general fracture criterion of PUCK is given by an easy to modify 7-

parameter model identifying FF and IFF for design issues. A more sophisticated version has been developed in the form of the mathematically simpler Puck-Knaust-IFF-failure criteria [31] given in Eq. (2), with $F(\sigma)$ identifies the fracture plane.

$$F = \frac{\sigma_2^2 + \sigma_3^2 - \sigma_2\sigma_3 + \tau_{23}^2}{f_{t,90} f_{c,90}} + \left(\frac{1}{f_{t,90}} - \frac{1}{f_{c,90}}\right)(\sigma_2 + \sigma_3) + \frac{\tau_{21}^2 + \tau_{31}^2}{f_{v,90,0}^2} + \underbrace{\frac{1}{f_{t,0,IFF} f_{c,0,IFF}}\sigma_1^2 + \left(\frac{1}{f_{t,0,IFF}} - \frac{1}{f_{c,0,IFF}}\right)\sigma_1}_{F(\sigma_1)} \tag{2}$$

2.4.3 Practical application for engineering problems

The definition of failure is an issue during failure analysis. In particular, initial failure like delamination has to be defined in the right way. The classification of failure criteria regarding their original aim has to be assessed before their use:

- Analysis range (first ply or post failure)
- Physical model (micro, macro or component level)
- Mathematical approach (limit, polynomial tensor or physically based)

One of the biggest problems for circulation of knowledge on failure of FRP is the non-availability of respective post-processing software codes. The TSAI-WU criterion is implemented in most software e.g. ANSYS® or ABAQUS® being able to analyze the composite structures. Due to the same mathematical layout of the parabolic criterion of the PUCK-KNAUST and the TSAI-WU criterion in form of tensor polynomials it is possible to implement the PUCK-KNAUST criterion in commercial FE-codes for an appropriate design of composite structures by enhancing the included failure criteria. The complete mathematical expression of the modification and implementation for engineering design problems is given in Tab. 2.

Tab. 2 Strength indices for PUCK-KNAUST modified TSAI-WU criteria for FRPs

Strength index	FF modification	IFF modification	Strength index	FF modification	IFF modification
$f_{t,x}$	$f_{t,0}$	1,3 $f_{t,0}$	$f_{v,xy}$	> 10^4 MPa	$f_{v,0,90}$
$f_{c,x}$	$f_{c,0}$	1,3 $f_{c,0}$	$f_{v,yz}$	> 10^4 MPa	$\sqrt{-1/3\, f_{t,90}\, f_{c,90}}$
$f_{t,y}$	> 10^4 MPa	$f_{t,90}$	$f_{v,zx}$	> 10^4 MPa	$f_{v,90,90}$

$f_{c,y}$	> 10^4 MPa	$f_{c,90}$	C_{xy}	0	0
$f_{t,z}$	> 10^4 MPa	$f_{t,90}$	C_{yz}	0	–1
$f_{c,z}$	> 10^4 MPa	$f_{c,90}$	C_{zx}	0	0

Due to the mathematically similar definition of the TSAI-WU-criteria and the PUCK-KNAUST-criteria by tensor polynomials, the implemented algorithm in FE-codes can be modified using Tab. 2 for the specific of fibre reinforced polymers and own stress and failure analyses.

2.4.4 Numerical aspects of bond-line delamination

Decohesion along interfaces plays an important role in a wide variety of failure processes in structures when using chemical bonding as the optimal form of combining two surfaces with each other. The analysis of delamination initiation is based on stresses and interaction criteria of the interlaminar stresses in conjunction with a characteristic distance as a function of geometry and material properties. Crack propagation is predicted by a fracture mechanics (FM) approach. This avoids the difficulties associated with a stress singularity at the crack front but requires the presence of a pre-existing delamination. When used in isolation, neither the strength-based approach nor the FM approach is adequate for a progressive delamination failure analysis.

For the combination of anisotropic materials like wood with FRP reinforcement in an interface damage law, the interaction between shear and normal stresses has to be considered. This problem can be solved by using a delamination analysis and an exponential interface damage law [33]. The definition of traction and separation depend on the finite element and the applied material model. Decohesion response is specified in terms of a surface potential $\Phi(\delta)$ relating the interface tractions and the relative normal and tangential displacements δ_n and δ_t across the interface. In most of the computations all cohesive surfaces are taken to have identical cohesive properties which simplifies the surface potential [34] to

$$\Phi(\delta) = e\,\sigma_c\,\bar{\delta}_n\left[1-\exp\left(-\frac{\delta_n}{\bar{\delta}_n}-\frac{\delta_t^2}{\bar{\delta}_t^2}\right)\right] \tag{3}$$

e Euler constant

σ_c maximum normal traction at the interface

$\bar{\delta}_n$ normal separation where the maximum normal traction is attained with $\delta_t = 0$

$\bar{\delta}_t$ shear separation where the maximum shear traction is attained

The potential leads by derivation on the displacements to the stresses if $\delta < \delta_{max}$ where the traction components T are coupled to both normal and tangential crack opening displacements. The main advantage of the cohesive zone modelling is that, when it is known where fracture may occur a priori, a cohesive zone may be placed anywhere along element interfaces in that area, to take these effects into account. Furthermore, using decohesion elements, both onset and propagation of delamination can be simulated without previous knowledge of crack location and propagation direction and is therefore suitable for structural design and evaluation of composite beams.

2.5 FRP damage by fire

When FRP is subjected to elevated temperatures, the strength and stiffness are reduced due to the loss of mechanical integrity of the polymer matrix [3]. The two-part epoxy adhesives normally used in bonding have glass transition temperatures less than 100°C. For externally bonded FRP reinforcement, the exposure to high temperatures in a fire situation can therefore lead to both loss in the reinforcing effect of the FRP and degradation of the bond between the FRP and timber. In order to prevent this situation, the innate insulating properties of timber can be used to protect the reinforcement by providing adequate timber covering to the FRP.

MARTIN and TINGLEY [35] undertook a test program in which FRP reinforced and unreinforced control glulam beams were subjected to standard fire exposure and mechanical loads equal to or in excess of the design loads. The beams were reinforced with three different types of FRP plates, bonded either externally to the soffit or between the bottom two laminations. The results showed that internally reinforced beams had a 44% higher fire endurance compared to the externally reinforced specimens. No difference in fire performance was found between the different types of FRP. WILLIAMSON [36] showed that glulam beams with externally bonded FRP can be designed to achieve a 1-hour rating. Fire exposure testing on glulam beams with beam end repairs comprising short epoxy resin replacement beams attached to the main beams by means of steel reinforcing bars [37] showed that these repairs could give one hour fire resistance. In this case, the steel bars had a minimum cover of 60 mm to the surface.

3. Applications

3.1 Beam end reinforcement

In order to make the interventions minimally intrusive, the connecting elements are confined to the corners of the beam and their length is reduced to the necessary minimum. The operating procedures depend on the specific requirements of the site. This starts with the propping of the beams followed by removal of the decayed portion of the timber, usually terminated in an inclined cut (Fig. 3). The new section of wood is shaped to match the external dimensions of the original decayed section. After the preparation of internal holes or external grooves between the original and newly introduced wood (Fig. 4), the holes or groove are partially filled with adhesive and the reinforcing elements are inserted. The last stage is the insertion of a final wood fillet, to hide the grooves (Fig. 5), followed by removal of the beam supports after the complete curing of the adhesive.

Fig. 3 Inclined cut of decayed end

Fig. 4 Linking grooves

Fig. 5 Insertion of a final wedge

Epoxy resins have been used in specific cases to repair timber that has deteriorated from decay or insect attack and certain structural deficiencies in existing construction. The compressive strength and filling capabilities of epoxies can aid in repairing timber structures. The tensile, shear, and bond strength of epoxies as structural wood adhesives are, in certain cases, limited and are further subject to variability due to conditions of use.

Mechanical reinforcement should be used in conjunction with epoxies for repairs intended to develop shear capacity. For structural rehabilitation and restoration the decayed ends will be cut off and replaced by timber or polymer concrete where the shear and tension forces resulting from loads are transmitted by GFRP reinforcement with a PU or EP resin

Fig. 6 Shear reinforcement with GFRP

filling compound between the new and the existing part of the structure (Fig. 6). In some cases the decayed part of the structure will be replaced by a polymer concrete (PC) supplement using rapid-setting organic polymers as binders and small grade aggregates such as sand or gravel. The application of PC in structural rehabilitation and restoration has several advantages. Historically significant structures can be protected by minimum disturbance of the construction and minimum replacement of decayed parts. The appearance of the timber structure will not be changed. The section design can be easily executed by timber formwork on the level of the necessary construction height. All work can be undertaken from the top side so suspended ceilings will remain unaffected. The floor below the reconstruction work can be used with some restrictions. The method is suitable for inhabited floors and restoration of complicated timber joints. The full load-carrying capacity is achieved after one day with an increase of the structural performance by 140% for timber-on-timber supplements and 30% for timber-on-PC supplements as reported by SCHOBER et *al.* [33],[34].

3.2 Tension reinforcement perpendicular to the grain

The tensile strength of the timber perpendicular to the grain is considerably lower compared to the tensile strength parallel to the grain. High tension stresses perpendicular to the grain stresses occur at notches, holes or curved beams and beams with variable height. Efficient reinforcement techniques are required for increasing or maintaining the load-carrying capacity of these structures. The reinforcement should offer high strength and stiffness and should create a more ductile failure of the structure. Reinforcement made of FRP offers high strength and stiffness along its fibre direction; however, it commonly does not lead to more ductility of the structure. There is relatively little information on research and applications regarding FRP reinforcement of regions with tension perpendicular to the grain reported in literature due to the existence of other reinforcing techniques offering more ductility, like e.g. self-tapping screws.

In general, the reinforcement should be applied close to the region of high tensile stresses perpendicular to the grain. As discussed by HALLSTRÖM and GRENESTEDT [38], the use of internal reinforcement may be necessary for internal members that are not accessible at their external sides. For wrapping of FRP fabrics around the timber member good accessibility of the member at all sides is crucial.

Studies on glued laminated timber (glulam) beams with holes reinforced by means of glass fibre reinforced polymers (GFRP) are reported by HALLSTRÖM and GRENESTEDT in [38],[39]. It was aimed

at reducing the stress singularity in the corner of the hole by changing the anisotropy by means of the reinforcement. However, the theoretical decrease of stresses at the corner of the hole was considerably lower than the level of increase of load-carrying capacity observed in the tests. The effect of load redistribution from the timber to the GFRP was confirmed in comparative FE simulations.

Results of tests on notched glulam beams reinforced by means of GFRP plates are reported by COUREAU et *al.* [40]. The reinforced notched beams showed higher load-carrying capacity compared to reference tests on unreinforced beams. Higher load-carrying capacities were achieved in the tests with an increase in the width of the FRP plates. Failure of the reinforced notched beams was accompanied by delamination of the FRP plates. Debonding of the FRP laminates was also observed in tests performed by JOCKWER [41]. In the case of perpendicular to the grain reinforcement, debonding occurred in the lower beam section due to insufficient bond line area, whereas in the tests reinforced with an angle of 45° to the grain debonding occurred in the upper beam section near the support due to the high relative stress difference between the timber in compression perpendicular to the grain and the CFRP in tension in fibre direction. In both types of configuration of the CFRP, it was not possible to prevent crack initiation in the timber at the notch corner by the reinforcement. The two configurations are illustrated in Fig. 7.

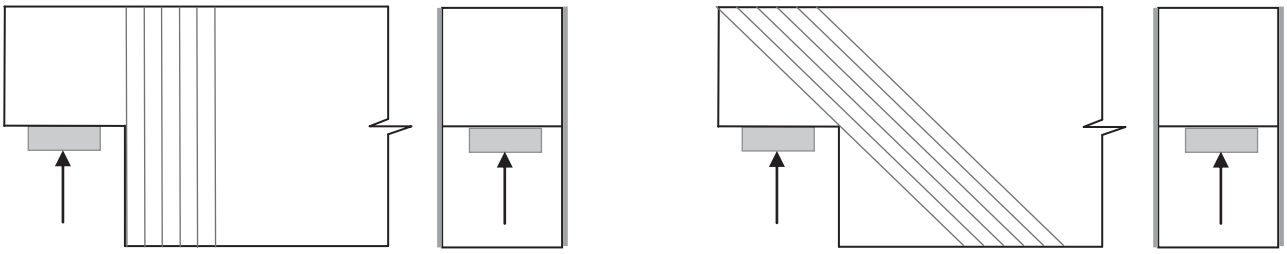


Fig. 7 FRP reinforcement of notched beams

Tests on curved and pitched cambered beams reinforced by means of GFRP are reported by ENQUIST et *al.* [42]. The reinforcement perpendicular to the grain led to a considerable increase in load-carrying capacity compared to unreinforced beams. Tensile failure in the GFRP and debonding occurred in the tests. Radial reinforcement repairs for curved bending members can also be accomplished by placing GFRP rods in oversized holes filled with epoxy PC. Here, the radial stresses are transmitted through the epoxy PC in shear.

The need for further research on the application of GFRP as reinforcement perpendicular to the grain includes [42]:

- Long-term tests to cover the effects from variation in moisture and ambient conditions
- Tests on full-size beams in order to cover scale effects.

3.3 Bending reinforcement

The reinforcement of timber members in bending using pultruded FRP rods, strips, plates or other structural shapes and fabric wraps has been the subject of a large number of research programmes during the last 50 years. The reinforcement can be deployed internally by bonding rods or strips in grooves cut into the tension and compression faces of the member or externally by bonding FRP plates to the tension face but not on the compression face due to the risk of buckling failure in the FRP. Experimental campaigns to investigate the reinforcement of solid timber beams and glulam with CFRP reinforcement by several researchers [43]-[47] have demonstrated that the use of a small percentage of reinforcement in the order of 1.5-2.5% can result increases in the bending strength and stiffness of up to 90% and 100%, respectively. In addition, reinforced beams were shown to have less variability in their properties than unreinforced beams. With increasing amounts of tensile reinforcement, the load-deflection response becomes ductile due to compression yielding at high strain levels.

Due to the high costs associated with CFRP materials, several investigators [47]-[50] have studied the flexural reinforcement of timber members with GFRP. As for the case of CFRP, large increases in strength were reported when using small percentages of reinforcement. However, the increases in stiffness were less significant due to the lower stiffness of the GFRP material. Decreased variability in the performance of the reinforced members was found. Limited ductility was achieved, which increased with increasing percentage of reinforcement. More recently, flexural reinforcement of timber using basalt fibre reinforced polymer (BFRP) has been undertaken with promising results [52].

3.4 Shear reinforcement

Various studies on the reinforcement and repair of regions with high shear stresses in glulam beams are reported in the literature. The studies were often performed more in order to compare different reinforcement techniques and to evaluate the reinforcing effect rather than to validate or verify design procedures. In general the tests can be roughly separated whether the reinforcing was applied externally or internally. External reinforcement was commonly made by means of FRP plates, fabrics or roving.

Internal reinforcement was commonly made by FRP rods. The different techniques and relevant parameters are shown below.

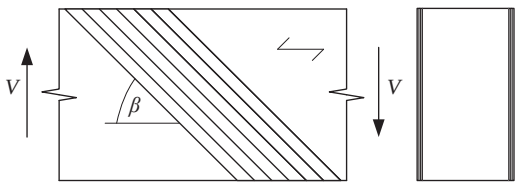


Fig. 8 Externally applied FRP plates

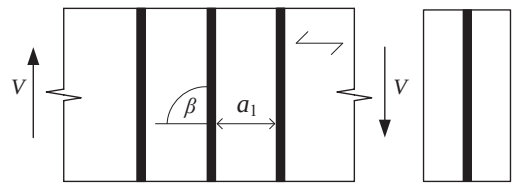


Fig.9 Internal applied FRP rods

A good summary of the literature regarding shear strengthening of timber beams by means FRP is given by ANDRE [4]. The main focus in this work is put on the properties of different fibre materials. The behaviour of mechanically connected GFRP plates as shear reinforcement of timber beams was evaluated by AKBIYIK et *al.* [53]. However, it is not possible to restore the stiffness of the undamaged beams by this reinforcement method.

An approach for the calculation of the stresses in the timber of a beam reinforced by means of FRP panels is proposed and evaluated by TRIANTAFILLOU [5],[54]. The approach is verified in a series of experiments on small size specimens. The main focus of the study was the required area fraction of the FRP panels and the relative height of the reinforcement in order to reduce the shear stresses in the timber. The approach is valid only for undamaged timber beams with no checks in the cross-section and, hence, not adequate for the repair of structures. As discussed in [5],[54] the reinforcing effect of the FRP depends on its stiffness in the grain direction of the timber. Hence, the use of multidirectional FRP fabrics or unidirectional FRP roving with small angle β relative to the grain direction of the timber are beneficial both for strength and stiffness of the reinforced beam.

The advantages of internal reinforcement by means of FRP rods are the good aesthetics of the wooden surface of the beam and the possibility of reinforcing beams with reduced accessibility from the sides. Hence, FRP rods are a good method for repairing beams in situ [55]. A large test series on damaged beams of railway bridges reinforced by means of GFRP rods installed with $\beta = 90°$ is reported by RADFORD et *al.* [6] and BURGERS et *al.* [56]. A considerable increase of the shear stiffness of damaged beams was observed in the tests. Compared to external FRP wraps internal FRP rods show a lower stiffness but also less material use and an easier installation. The latter point is of special importance if beams are only accessible from the bottom surface. GFRP rods with $\beta = 90°$ were tested in small and medium scale glulam beams by GENTRY [57]. A general good reinforcing effect and an increase of

approximately 50% of the load-carrying capacity in shear were observed. The method of reinforcing beams by means of FRP rods with $\beta = 90°$ is also referred to as the Z-spiking method.

Tests on glulam beams reinforced with GFRP dowels in shear and bending were performed by SVECOVA and EDEN [58]. The internal shear reinforcement was applied with different distance a_1 between the bars partly along the whole length of the beam and partly only in the region of high shear force near the support. The rods were installed perpendicular to the grain at $\beta = 90°$. It was possible to increase the load-carrying capacity, to reduce the variability of the beams, to increase the deflection before failure and hence, introduce some kind of ductility to the beams. A dowel distance equal to the beam depth is recommended by the authors based on tests.

An example of an application of CFRP wraps in practice was given by LAUBER [59]. A rafter was reinforced in shear by wrapping it with CFRP fabric in a U-shape. The advantage of fabric compared to plates is better flexibility during installation and its possibility to adapt to other connecting elements. An exact surface preparation of the existing beam is necessary in order to smooth the surface and to create good bond between timber and CFRP. It is emphasized that constant ambient conditions and constant moisture content of the structure are required in order to prevent delamination and to guarantee the long term strength.

SONTI and GANGARAO [60] found FRP wraps to be adequate for increasing the strength and stiffness of timber beams in infrastructural application. WIDMANN et *al*. [61],[62] performed a test series on full size beam specimen with shear cracks reinforced by means of CFRP roving with $\beta = 45°$. A considerable increase of the stiffness of the damaged beam was observed at that fibre angle.

3.5 Pre-stressed FRP

The reinforcing efficiency of FRP materials can be improved by prestressing. In passive or slack reinforcement, the load-carrying capacity of the FRP is often not reached as failure commonly occurs in the timber element. In the case of tensile reinforcement of beams, a tensile force is usually applied to the FRP sheet or rod by means of hydraulic jacks before pressure bonding to the timber element. The eccentric prestress induces significant compressive stresses in the bottom of the beam, which oppose the tensile stresses due to the external loads. In this way, the bending strength of the beam is increased. In addition, higher compressive stresses in the timber under the action of the external loads give rise to a plastic response. Prestressing of the beam also influences the deformation of the member. When the

jacking force is released, the beam cambers in the upward direction. This deflection can be offset against the deflection due to the external loads thereby giving an apparent increase in flexural stiffness.

Several researchers have investigated the feasibility of prestressing timber beams with FRP plates. As far back as 1992, TRIANTAFILLOU and DESKOVIC [63] undertook a small scale testing programme, in which one timber beam was reinforced with 2.5% prestressed CFRP plate bonded to the beam soffit. A strength gain of 40% compared to 16% for passive reinforcement was achieved. More recently, DAGHER et *al.* [64] tested 6.7 m long glulam beams with 1% GFRP plate reinforcement bonded to the soffit using a PRF adhesive. A prestressing force of 30% of the ultimate tensile strength was applied to the FRP plate using hydraulic jacks. The strength of the prestressed beams was found to be 95% higher than unreinforced beams and 38% higher than passively reinforced beams. The average precamber of the beams was 10.9 mm, which can be offset against the allowable deflection. RODD and POPE [65] investigated the behaviour of a GFRP prestressed glulam beam with a bumper lamination glued to the bottom face of the FRP plate. NEGRÃO et *al*. [66]reported increases in bending resistance of 30% in solid timber beams when prestressed with 0.3% CFRP. The precamber was about 30% of the allowable deflection.

A key limitation is the fact that the prestressing force needs to be anchored at the ends of the beams. Load transfer takes place over a short length at the end of the bond line and delamination may occur due to the local stress concentration. BRUNNER and SCHNUERIGER [67] described a gradiented prestressing device developed by EMPA as a possible solution to this problem. In this method, the curing of the epoxy adhesive is carried out in a controlled fashion using heat to accelerate curing. Starting at mid-span and gradually moving towards the support while reducing the prestressing force, ensures that the prestressing force is anchored over a longer length and the force at the end is reduced. An alternative approach [64] is to release the jacking force immediately after applying the bonding pressure.

Prestressing of FRP using hydraulic jacks may not be suitable for on-site applications. An alternative method proposed by Negrão et *al*. [65] involves the pre-cambering of the timber before installing the FRP reinforcement as shown in 9. This is achieved using an adjustable prop located at the centre of the beam. This approach has the additional advantage of inducing a triangular bending moment distribution in the beam due to the prop force resulting in a low, constant shear stress in the glue line. This method may not be suitable for large beams as the prop force required may be too great. Another method, most recently developed at Chalmers University of Technology, is the stepwise prestressing of the laminate, which greatly reduces stress concentrations at the ends of the laminate, making mechanical anchorage unnecessary, [68][67]. This method differs from the gradiented pre-stressing device developed by EMPA in that it is based on the stepwise introduction of the pre-stressing force in the laminate, rather than the gradual release of the pre-stressing force. Special equipment was also developed to pull the laminate in such a way that the entire pre-stressing force is gradually introduced into the laminate in a discrete or continuous manner by distributing the total force over ten steps. Numerical and experimental studies show that it is possible to reduce the shear and peeling stresses in the bond line to levels below 1 MPa and 0.2 MPa respectively, for a pre-stressing force of 100 kN. These values are well below the shear and tensile strength perpendicular to the grain of wood and adhesives.

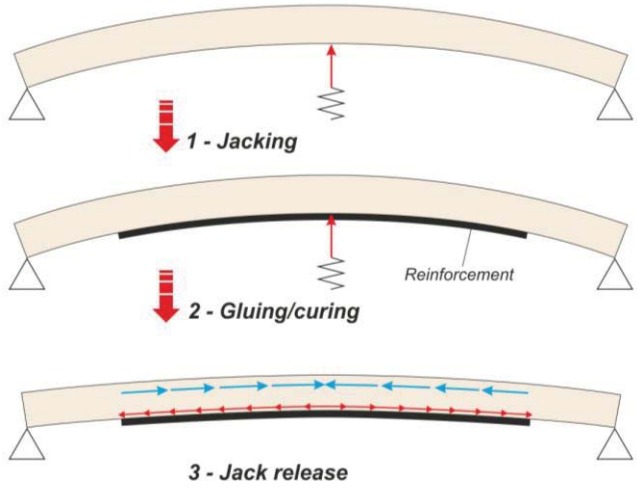


Fig. 10 On-site procedure for FRP prestressing [65]

Design models for FRP prestressed timber beams have been presented by a number of researchers. BRUNNER and SCHNUERIGER [67] describe a calculation model for the case of FRP prestressed plate bonded to the bottom of the beam using an iterative approach for the moment capacity and a linear elastic perfectly plastic model for timber in compression. TRIANTAFILLOU and DESKOVIC [63] considered the same situation but used a bilinear model with a falling branch post-yield for compressive response. BRADY and HARTE [69] presented a closed form expression for the moment capacity of prestressed glued-laminated beams incorporating a bumper lamination covering the FRP, where the timber in compression was modelled using a bilinear model with a falling branch post-yield.

McConnell et *al.* [70] presents a theoretical model for glulam members post-tensioned with BFRP tendons.

The long-term performance of prestressed timber requires further examination before this method can be recommended for use.

4. Design

4.1 Flexural strengthening

Flexural reinforcement can be placed on the tension and compression faces of the member and can be in the form of externally bonded plates (EBP) or near surface mounted (NSM) rods, plates or strips. Externally bonded plates are not recommended for compression reinforcement due to the likelihood of buckling.

Analysis of reinforced timber flexural members is based on a classical strength of materials approach. The analysis is based on the following assumptions:

- The member cross-section is symmetric in the plane of bending
- Plane sections remain plane
- De-bonding or slippage does not occur between the FRP and wood
- The FRP material is linear elastic to failure in tension and compression
- The timber is linear elastic to failure in tension and nonlinear in compression

Various constitutive models have been used to model the non-linear behaviour of timber in compression. The bilinear Bazan-Buchanan model [71] assumes linear elastic behaviour up to the yield point followed by a falling branch with a negative slope. This model has been found by a number of authors [45],[48],[69] to match well with experimental results. In many cases, data on the slope of the falling branch is not available and, in that case, a simplified linear elastic, perfectly plastic model has been used [44],[49]. A quadratic approximation has also been used successfully [46].

In order to determine the ultimate moment capacity, all possible failure modes must be considered. In a large number of test programmes over the last twenty years, it has been found that failure of the FRP reinforcement is unlikely to occur and, in practise, only two failure modes need to be considered. These are:

– Mode 1: Failure of the timber in tension while in compression the response is linear elastic

– Mode 2: Failure of the timber in tension after the onset of compressive yielding

These two scenarios are illustrated below for beams with NSM reinforcement on both faces.

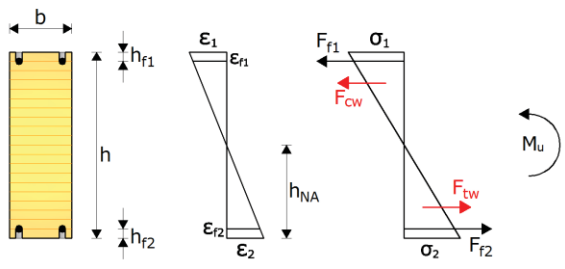


Fig.11 Failure mode 1

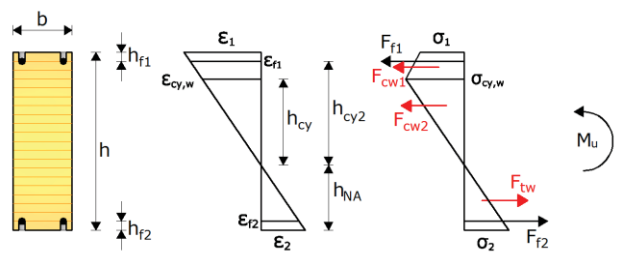


Fig. 12 Failure mode 2

For failure mode 1, the maximum tensile strain in the timber, ε_2, reaches the ultimate value while the maximum compressive strain, ε_1, is less than the yield strain.

$$\varepsilon_2 = \varepsilon_{tu}\ ;\ \varepsilon_1 \le \varepsilon_{cy} \tag{4}$$

where ε_{tu} is the ultimate tensile strain for the timber and ε_{cy} is the timber compressive yield strain. The analysis is based on strain compatibility and force equilibrium. Equilibrium of the axial forces acting on the reinforced section requires that.

$$F_{f1} - F_{f2} + F_{cw} - F_{tw} = 0 \tag{5}$$

where F_{f1} and F_{f2} are the forces in the compressive and tensile reinforcement, respectively, and F_{cw} and F_{tw} are the total compressive and tensile forces in the wood, respectively.

As all of the strain terms are linearly related, Eq. (5) reduces to

$$E_f A_{f1} \cdot \frac{h - h_{NA} - h_{f1}}{h - h_{NA}} - \left(E_f - E_w\right) A_{f2} \cdot \frac{h_{NA} - h_{f2}}{h - h_{NA}} + \frac{1}{2} \cdot E_w b \cdot \left(h - h_{NA}\right) - \frac{1}{2} \cdot E_w b \cdot \frac{h_{NA}^2}{h - h_{NA}} = 0 \tag{6}$$

where the dimensions are defined in Fig. 11, E_f and E_w are the elastic moduli of the FRP and timber, respectively, A_{f1} and A_{f2} are the areas of tensile and compressive reinforcement, respectively, and ε_{f1} and ε_{f2} are the corresponding strain terms. Solving this quadratic equation gives the location of the neutral axis, h_{NA}. Increasing the axial stiffness EA of the FRP in tension causes the neutral axis to move down, while increasing that of the FRP in compression causes the neutral axis to move up. Knowing

the location of the neutral axis, the ultimate moment capacity of the reinforced section is determined by taking moments of the forces about the neutral axis

$$M_u = F_{f1} \cdot \left(h - h_{NA} - h_{f1}\right) + F_{f2} \cdot \left(h_{NA} - h_{f2}\right) + \frac{2}{3} F_{cw} \cdot \left(h - h_{NA}\right) + \frac{2}{3} F_{tw} \cdot h_{NA} \tag{7}$$

For failure Mode 2, the maximum tensile strain in the timber, ε_2, reaches the ultimate value while the maximum compressive strain, ε_1, is greater than the yield strain (Fig. 12).

$$F_{f1} - F_{f2} + F_{cw1} + F_{cw2} - F_{tw} = 0 \tag{8}$$

This again gives a quadratic expression which can be solved to give the location of the neutral axis. The ultimate moment capacity is then determined by taking moments of the normal forces about the neutral axis

$$M_u = F_{f1} \cdot \left(h - h_{NA} - h_{f1}\right) + F_{f2} \cdot \left(h_{NA} - h_{f2}\right) + \frac{2}{3} F_{cw1} \cdot h_{cy} + \frac{2}{3} F_{cw2} \cdot h_{cy2} + \frac{2}{3} F_{tw} \cdot h_{NA} \tag{9}$$

The extent of compression yielding in the timber depends on the relative magnitudes of ε_{tu} and ε_{cy} and on the amount of tension and compression reinforcement. Increasing the amount of reinforcement on the tension side leads to more ductile behaviour and increased strength. Ideally, the timber beam should be unloaded prior to application of the FRP reinforcement. If this is not possible, the existing strain in the structure before FRP strengthening takes place must be taken into account.

Fundamental to the design process is the selection of appropriate design values for the material properties. While some countries have developed national guidelines, there is currently no European Standard specifying the design of FRP structures or FRP reinforcement for timber, steel or concrete structures. The Italian National Research Council has developed guidelines [2],[3] for FRP reinforcement of existing structures. Using a limit state approach, the design value of a property is expressed as

$$X_d = \eta \cdot X_k / \gamma_m \tag{10}$$

where η is a conversion factor to account for environmental and creep effects, X_k is the characteristic value and γ_m is the partial material factor. Some of the recommended values for the conversion factor η from CNR-200 [2] are summarised in Tab. 3. CNR-DT200 provides different partial material factors

for FRP depending on the failure modes and the types of FRP system used. For FRP rupture, γ_m is given as 1.1 for certified systems and 1.25 for uncertified systems.

Tab. 3 FRP design conversion factors η [2]

FRP	Exposure			Loading mode	
	Internal	External	Aggressive	Creep	Fatigue
Glass / Epoxy	0.75	0.85	0.95	0.30	0.50
Aramid / Epoxy	0.65	0.75	0.85	0.50	0.50
Carbon / Epoxy	0.50	0.70	0.85	0.80	0.50

A major challenge in the design of reinforcement for existing timber structures is the determination of the properties of the timber for use in design calculations. This is particularly difficult for older structures. Non-destructive testing carried out during the assessment of the structure prior to reinforcement is necessary in order to get characteristic values for design. For timber, the design value of a property is expressed as

$$X_d = k_{\text{mod}} \cdot X_{k(n)} / \gamma_m \tag{11}$$

where k_{mod} is the modification factor for service classes and load-duration and $X_{k(n)}$ is the characteristic value of the property from on-site tests. It has been found by several researchers [47],[48],[50] that reinforced timber beams fail in tension at a higher stress than unreinforced beams. This is because the reinforcement in the tension zone bridges defects such as knots and constrains crack opening so that the timber can carry a higher load before failure. Gentile et al. [48] found that an enhancement in tensile strength of between 18 and 46% could be achieved depending on the unreinforced timber strength.

For the serviceability limit state, the materials are assumed to behave elastically. The flexural stiffness *EI*, of the reinforced beam can be determined using the transformed section method. Depending on the situation, a gravity load test may be carried out on the unreinforced beam to measure unreinforced flexural stiffness. From this the mean modulus of elasticity E_w for the timber parallel to grain and the modular ratio *n* can be calculated, where

$$n = E_f / E_w \tag{12}$$

In the transformed section method, an equivalent section in which the FRP reinforcement is transformed to an equivalent area of timber is considered. The bending stiffness for the reinforced section is then found by multiplying the second moment of the transformed section by the modulus of

elasticity of the timber. The maximum increase in flexural stiffness can be achieved by placing half of the reinforcement on the compressive face and half on the tensile face [44], however, the increases compared to reinforcing only on the tensile face may not be significant enough to justify the additional material and labour costs. Increasing the modular ratio will result in greater increase in the flexural stiffness. An efficient method for investigating different reinforcing configurations is finite element modelling incorporating nonlinear material behaviour for timber [43],[72].

4.2 Shear strengthening

4.2.1 Shear strengthening by means of external reinforcement

The design of shear reinforcement is different for intact cross-sections without cracks and for damaged members. In the former case, both the timber and the FRP elements carry the forces whereas in the latter case only the FRP element carries the load in the regions of the cracked member. As a consequence the design procedures are different for both cases.

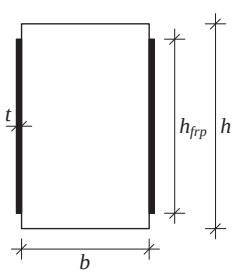


Fig. 13 Intact timber cross-section

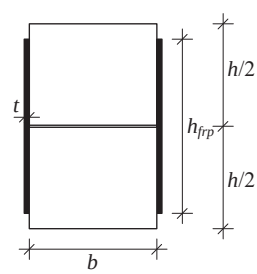


Fig. 14 Cracked timber cross-section

TRIANTAFILLOU [54] summarized the design procedure for the case where both timber and FRP elements carry loads proportionally to their stiffness in direction of the beams axis. In addition to Eq. (12), the area fraction of the FRP α_{frp} can be used for simplification.

$$\alpha_{frp} = \frac{A_{frp}}{A_{timber}} = \frac{2 \cdot t \cdot h_{frp}}{b \cdot h} \tag{13}$$

where the geometric quantities are defined in Fig. 13 and Fig. 14.

The maximum shear stress in the timber, due to the shear force V, can be calculated by choosing the timber as the reference material:

$$\tau_{timber,\max} = \frac{V\, S_{comp,\max}}{I_{comp}\, b_{comp}} \tag{14}$$

where the effective geometrical properties of the composite cross-section are denoted by the subscript *comp*. The effective width of the composite cross-section is:

$$b_{comp} = b + 2\, tn = b_{timber}\left(1 + n\,\frac{h}{h_{frp}}\,\alpha_{frp}\right) \tag{15}$$

The effective moment of inertia of the composite cross-section is:

$$I_{comp} = \frac{b\, h^3}{12} + \frac{2\, n\, t\, h_{frp}^3}{12} = I_{timber}\left[1 + n\left(\frac{h_{frp}}{h}\right)^2 \alpha_{frp}\right] \tag{16}$$

and the effective static moment of area in the centre of the composite cross-section is:

$$S_{comp,\max} = b\frac{h}{2}\frac{h}{4} + 2nt\frac{h_{frp}}{2}\frac{h_{frp}}{4} = S_{timber,\max}\left(1 + n\frac{h_{frp}}{h}\alpha_{frp}\right) \tag{17}$$

Hence, the stresses in the timber according to Eq. (14) can be calculated as follows:

$$\tau_{timber,\max} = \frac{V\, S_{timber,\max}}{b_{timber}\, I_{timber}} \cdot \frac{1 + n\frac{h_{frp}}{h}\alpha_{frp}}{1 + n\frac{h}{h_{frp}}\alpha_{frp}} \cdot \frac{1}{1 + n\left(\frac{h_{frp}}{h}\right)^2 \alpha_{frp}} \tag{18}$$

And consequently the maximum shear stresses in the FRP are:

$$\tau_{frp,\max} = n\, \tau_{timber,\max} \tag{19}$$

Using Eq. (12) and (13) the amount (area fraction α_{frp}) and properties (n) of the FRP can be adjusted in order to reduce the shear stresses in the timber and satisfy the shear strength of the FRP. For a good reinforcing effect FRP roving should be applied with an angle smaller than 90° with respect to the grain direction of the timber. WIDMANN et *al.* [61] reported good performance with an angle of 45° between FRP fibre and grain of the timber. In addition, the bondline stresses have to be checked.

In situation of a damaged beam by shear cracking in the cross-section centre (Fig. 14), shear is transferred only by the FRP. For a perfect reinforcement the full shear stress has to be carried by the FRP. However, in reality the reinforcing effect connecting the upper and lower beam parts will not be

perfectly stiff and, hence, a reduction of the transferred shear forces will be the result. Reinforcement with low stiffness, like FRP roving perpendicular to the grain, will not be able to transfer loads between the timber members and will not contribute to the stiffness of the beam. The two beam parts will act separately as individual members. Hence, an inclined application as described by WIDMANN et *al*. [61] should be preferred. An estimation of the maximum stresses acting in the FRP can be made assuming perfectly stiff reinforcement, where shear stresses at the crack location have to be carried in full by the FRP.

Stress peaks occur in the bond line between FRP and timber in vicinity of the shear crack. Securing an intact bond line is crucial for achieving the optimal reinforcing effect. Special consideration has to be paid to the long term behaviour of the bond line under the influence of moisture variations and changes in ambient conditions. The slenderness of the FRP sheets together with the high stiffness can cause high tension stresses in the laminate in the case of swelling of the timber or buckling due to compression in the case of shrinkage of the timber.

4.2.2 Shear strengthening by means of internal reinforcement

Internal reinforcement with FRP rods act locally along the beam axis. In the undamaged cross-section, the distribution of shear stresses is affected only in the reinforced area. For design of undamaged cross-sections, the internal FRP reinforcement should be neglected. Similar considerations as for reinforcement by means of self-tapping screws [73] can also be made for FRP rods. Due to the low relative portion of FRP compared to the timber cross-section the impact of the reinforcement on stiffness can be considered to be low. Nevertheless, FRP rods can be an adequate measure for reinforcing damaged beams with shear cracks. The FRP rods should be designed to carry the forces acting between the upper and lower timber parts. In the literature, most tests have been made with FRP rods installed perpendicular to the grain, however, an inclination should be chosen in order to benefit from the high tensile strength of the reinforcing elements. SVECOVA and EDEN [58] observed a good reinforcing effect with a dowel distance equal to the beam depth in their tests.

4.3 Long-term deformations in timber and composites

4.3.1 Overview

Timber structures are subject to long-term deformations due to creep, which are exacerbated in the presence of moisture and especially moisture variation, known as mechano-sorptive creep. By reinforcing timber with material that has superior properties when it comes to short- and long-term stiffness, the long-term behaviour could be improved and deflection could be reduced. The first recordings of accelerated creep in wood due to varying humidity conditions were described in the 1960s [74]. These recordings were later verified by performing bending tests on small wood specimens in both cyclic and constant humidity [75]. Specimens subjected to constant humidity show an almost constant creep rate. For cyclic humidity the deflections varies with the drying and wetting cycles, but the total deflection increases for every cycle. For higher stress rates, the rate of deflection increases significantly in the presence of cyclic humidity.

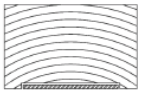
Generally, the fibres in composite materials are not expected to creep significantly. It is widely known that the creep for specimens loaded in the fibre direction is negligible [76]. On the other hand, the resins and adhesives exhibit marked rheological properties (viscous properties) experiencing significant creep, which are strongly influenced by temperature. To improve the long-term properties of two-component epoxy type adhesives, a study was made where dog-bone shape epoxy specimens were cast [77]. Half of the specimens were reinforced with 0.5% carbon fibres. These specimens had a creep rate 38% lower than the unreinforced specimens.

4.3.2 Experimental creep test on solid wood strengthened with composites

Creep testing of unreinforced and reinforced beam specimens was carried out at Chalmers University of Technology [78]. In the study, 24 specimens were used with the cross-section 45x70 mm and length of 1.1 m. In order to minimise the variability of the material properties, the specimens were well defined in terms of the origin of the raw material and sawing pattern [79]. The specimens were assigned to four different groups with approximately the same mean value and standard deviation in modulus of elasticity. The different reinforcement schemes are shown in Tab. 4. Two types of CFRP were used, namely, CFK 150/2000 (CFRP 165) by S&P and Carbodur H514 (CFRP 300) by Sika. No creep was expected to occur in the CFRP reinforcement or in the steel as it was loaded in the direction of the fibres. The adhesive used was in all cases S&P Resin 220 Epoxy. In order to allow moisture

exchange on three parts of the cross-section, the tension surface of unstrengthened timber specimens were sealed with epoxy.

Tab. 4 Creep test specimens and reference beam, sealed with epoxy on the tension side.

Beam types				
	Timber	CFRP 165	CFRP 300	Steel
Strengthening schemes	Sealed with epoxy	CFK 150/2000*	CarboDur H514**	Steel
Reinforcement area [mm²]	-	70	70	100
E-Modulus [GPa]	-	165	300	210
Tensile strength [MPa]	-	2310	1350	235

* Sto S&P flexural strengthening product (plates)

** Sika Chemicals flexural strengthening product (plates)

The creep tests were conducted in four-point bending at a stress level of 8 MPa. Directly after loading of the specimens the climate was set to 90% RH thereafter the climate was changed between 30% RH and 90% RH in two week cycles, while the room temperature was held constant at 23°C. The mid-point curvature was measured using a LVDT gauge. The results showed that the reinforced specimens had a lower initial deflection than the un-reinforced specimens. This result was expected due to the higher stiffness of the reinforced specimens. The creep results were presented as relative creep, i.e. the deflection divided with initial deflection (after 60 seconds). The mean values of the relative creep from all series are shown in Fig. 15.

The variation in relative creep between the wet and dry periods is very high, much higher for the reinforced specimens than for the unreinforced specimens. For the reinforced specimens the variation between 90% and 30% RH is of the same magnitude as the initial deflection. This is caused by the longitudinal shrinkage of the timber material while the reinforcement material does not shrink. This effect is much larger when one-sided reinforcement was used. The conclusion is that the reinforcement material should be distributed on both the tension and compressive side of the beams. The trend lines in Fig. 16 were calculated by taking the sum of the initial value and the final value for each two week period. The trend showed clearly that the reinforcement prevents the mechano-sorptive creep rather well. After the first four weeks the increase of the relative creep for the steel and CFRP-300 reinforced specimens is only a fourth of the increase for timber with epoxy.

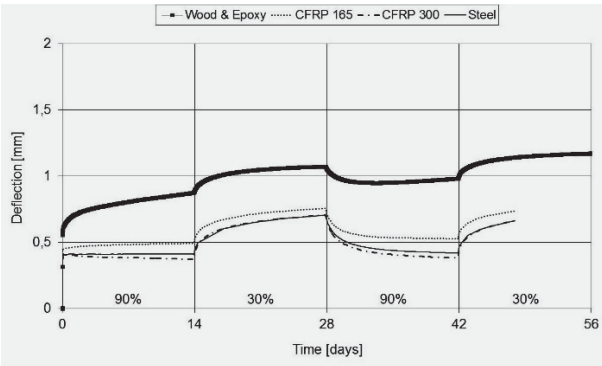


Fig. 15 Mean relative creep for the four different reinforcement schemes

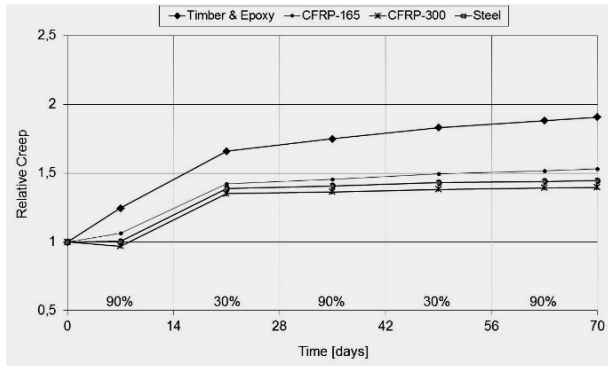


Fig. 16 Mean relative creep - trend lines for the four different reinforcement schemes

4.3.3 Design aspects

In order to minimise the mechano-sorptive creep, it is beneficial to strengthen both the tension and compression sides of a timber beam to avoid large moisture-induced movements. EC5 does not provide k_{def} factors for FRP strengthened beams. However, the main benefit is due to increased short-term stiffness of the strengthened beams, but the long-term creep deformations are the same as for unstrengthened beams. More research is needed to investigate various climate classes and load durations for certain standardised strengthening system. Calculation of deflection for a reinforced beam (with about 2% reinforcement) in a residential house showed that it is possible to increase the span length by as much as 20% or reduce the size of the cross-section compared to an un-reinforced beam.

5. Quality Control for Bonding on Site

The success of the reinforcement intervention depends on the implementation of adequate quality control measures at each stage of the process. All work must be carried by appropriately experienced personnel, using suitable materials, procedures and equipment in accordance with the Quality Plan. A procedure for quality control on site was developed as part of the EU CRAFT Project LICONS [80]. This was subsequently adopted by COST Action E34 [19]. Draft standards for on-site acceptance testing of mixing and application of adhesives have been prepared by CEN TC 193/SC1/WG11 [8]-[11]. These groups recommended that the Quality Plan should include the following checks and should specify the frequency of each check / test.

1. Reception of Materials: A record should be kept of all materials delivered to site. The materials should be checked to ensure that they match the specification and the expiry date should be checked.
2. Inspection and tests
   a) Elements to be repaired: the timber moisture content should be checked and recorded, the condition of the gluing surface should be checked and the dimensions and locations of holes, slots etc. should be checked and recorded.
   b) Mixing and application of adhesives: the following tests should be carried out in accordance with the draft standards [8]-[11]
      - On-site sampling and measurement of the cure schedule for adhesives
      - On-site sampling and subsequent laboratory measurement of the shear strength of adhesive joints
      - On-site sampling and proof loading of the strength of adhesive joints
   c) Visual inspection of final repair including dimensional checks.

Based on these tests, the actual properties obtained on site can be compared to the values used in the design.

6. Outlook and recommendations

Using bonded fibre-reinforced polymer laminates for the strengthening and repair of wooden structural members has been shown to be an effective and economical method. The high strength and stiffness, light weight and good durability properties of FRP composites, together with advantages offered by adhesive bonding, have made it a suitable alternative for traditional strengthening and repair techniques. FRP materials and adhesives and their properties suitable for various methods are discussed. It is pointed out that careful surface preparation is essential in order to achieve good bond strength and durability.

Two-part, cold-cure epoxy adhesives have generally been found to be most suitable for on-site bonding, as they have good gap-filling properties and low curing shrinkage. It is pointed out that careful surface preparation is essential in order to achieve good bond strength and durability. Research

on the bond behaviour of FRP to timber is still in its infancy. More research is needed, focusing in particular on various factors affecting the bond between FRP and wood, including combined properties and the local bond-slip relationship for FRP to timber, which can be used directly for finite element analysis.

A design approach, which accounts for the fracture and delamination in FRP strengthening, including appropriate material models, is presented. Failure-mode-based strength criteria are discussed and the development made by PUCK is shown in terms of the mathematical formulation and strength indices for the PUCK-KNAUST failure criterion. The applications presented are related to beam-end reinforcements, improving tension strength perpendicular to the grain, shear reinforcements and the pre-stressing of FRP on the tension side of a beam in flexure.

Beam-end reinforcement is a very important repair method for the decayed part of a structure and specifically the restoration of historically significant structures. An increase in structural performance of 140% for timber-on-timber supplements has been reported. Studies of the slack reinforcements of beams in flexure and shear reinforcement are briefly summarised, as studies of this kind have been conducted for many years and a large amount of research is available. The pre-stressing of FRP materials on the tension side of a beam in flexure offers the most effective utilisation of these materials; increasing the load-bearing capacity and pre-cambering of existing beams and, by doing so, improving the serviceability limit state which often governs the design.

It is recognised that one major challenge when it comes to the design of reinforced timber members in the ultimate limit state is the lack of appropriate properties of timber and of older timber structures in particular. The maximum increase in flexural stiffness (in the serviceability limit state) can be achieved by placing half the reinforcement on the compressive face and half on the tensile face. There are no creep factors for long-term deflection as defined in design codes for timber beams strengthened in flexure. It is therefore not possible to take advantage of lower mechano-sorptive creep than that in unstrengthened beams. In general, more research is needed on long-term tests and durability performance to cover the effects of variations in moisture and ambient conditions. The need for appropriate quality control for bonding on site is recognised and a quality plan based on various standards is presented.

Acknowledgement

This paper was first published in 'Reinforcement of Timber Structures. A state-of-the-art report', Ed. A. Harte, P. Dietsch, Shaker Verlag, 2015.

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