Fibre-reinforced polymers in timber structures: description and evaluation of possible applications and overview of research and development work carried out until 2020

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Summary

The use of fibre-reinforced polymers (FRP) in structural engineering has increased significantly worldwide until the early 2000s. FRP composites offer some advantages over other types of reinforcement, namely very high load-carrying capacities, resistance to harsh environments, and high strength-to-weight ratios.
Structural timber elements can be strengthened during production (*hybrid timber-FRP composite members*) or reinforced later on site (*FRP-reinforced timber members*). The strengthening can be *passive* (i.e. applied with changing the stress state of the timber member) or *active* (e.g. pre- or post-tensioned, changing the stress state of the reinforced member). FRPs can also be used instead of steel parts (e.g. as glued-in rods GiRs, or as dowel-type fasteners). A sensible use of FRP in timber structures requires knowledge not only of the advantages of both materials, but also of their performance limits and weak points. An extensive summary of the characteristics and behaviour of both materials is presented in this report.

Many attempts to improve the behaviour of timber beams have been reported, with different production and cost requirements and varying levels of effectiveness. The reviewed literature covered several decades and showed that most FRP-based strengthening strategies had been tried by the late 2000s. The most commonly studied strategies were related to improving the bending behaviour of GLT beams. Improvements in the load-carrying capacity and stiffness of GLT beams were observed by gluing FRP composites directly to the tension face of the beams, by inserting them between timber laminations (therefore protecting them from direct exposure) or in longitudinal grooves, and also by embedding only the fibres in the timber adhesive (usually PRF). It was also often reported that longitudinal bending strengthening led to other types of unwanted brittle failures, such as shear failures. Studies on strengthening with pre-stressed FRP composites have repeatedly shown no significant improvement over passive (i.e. non pre-stressed) strengthening. An important aspect regarding the behaviour of timber beams reinforced with FRP composites is that, unlike in reinforced concrete, ductility comes from over-reinforcing in tension, to induce compressive parallel to the grain failures in timber. This is because FRP composites are brittle, unlike steel reinforcements. Most of the tests reported have been performed under displacement control usually inducing slowly progressing failures that allow for stress redistribution. The corresponding force-displacement curve is jagged, exhibiting abrupt load drops followed by an increase in the applied force. This gives a false impression of deformation capacity or ductility. In reality, most loading scenarios are not displacement-controlled, but force-controlled, i.e. it is the load applied to the element that increases gradually and not the displacement.

Regarding shear and tension perpendicular to the grain, research has been much more limited. Strengthening based on applying FRP composites on the side faces of timber members is an effective strategy, but aesthetically unappealing. Internal shear and tensile reinforcements with glued FRP composites does not seem to provide any significant advantage over the nowadays-common reinforcement with self-tapping screws or steel rods, except maybe if fire is a concern and exposed steel parts are to be avoided.

Research on the strengthening of timber columns with FRP composites has also been limited and tests at the structural scale have shown that steel reinforcements might performed better and that FRP-based reinforcement might only be advantageous in the case of relatively high cross sections.

The use of FRP composites in timber connections has shown that FRP composites can be successfully used to strengthen timber in the connection area or to replace some of the connection components that are usually made of steel. In the first case at the cost of additional production steps and facing the competition of high-performing timber-based panels, and in the second case at the cost of lower load carrying capacities. As GiRs, FRPs have a major difference to steel, which is that ductile failure modes are not possible.

The timber-FRP bond behaviour has been extensively studied and current knowledge allows the development of reliable systems, i.e. combinations of materials, adhesives, and surface treatments. The durability of timber-FRP bonded interfaces is highly dependent on the specific nature of the components that are used and on the bonding procedure, being extremely difficult to make even general statements about types of adhesives or FRPs. Nevertheless, specific combinations have shown that they can fulfil the same performance requirements that are set for timber-timber bond interfaces. Given the wide variety of timber species, the inherent anisotropy and high variability of timber, the development of reliable general bond-slip models is complex and has mostly started in the last few years.

The long-term behaviour, namely creep, is still an open issue, namely under carrying climate and loading conditions (as it still is for unreinforced timber members!). There are only a few available studies, but most of them seem to show no reduction of creep deformations in FRP-strengthened beams.

The fire behaviour of FRP-strengthened timber elements is highly dependent on how protected the FRP composite is within the timber cross section. Since the effect of the strengthening is significantly reduced when the FRP composite is not located very close to the zone of maximum stresses, the need to protect it against direct exposure to fire usually involves a compromise between increasing the load-carrying capacity at normal temperature and fire resistance. Nevertheless, if exposure to fire is a concern, some types of strengthening with FRP might be more interesting than the equivalent reinforcement with steel parts, since FRPs might not conduct so much heat into the cross section. This is mainly relevant for reinforcement or components that are embedded and go through the entire cross section (or most of it), such as internal reinforcements against shear and tension perpendicular-to-the-grain failures and also for dowel-type fasteners.

Nowadays, FRP-based strengthening of structural timber faces well-established competitors. For longitudinal strengthening (i.e. bending, compression), the main alternative is the use of high-performing timber-based products (e.g. spruce or beech LVL), which can be easily integrated in existing production processes. For transversal strengthening (i.e. shear and tension perpendicular-to-the-grain), the main alternatives are self-tapping screws and glued-in steel rods. Steel reinforcements have a significant advantage of being easily machined and connected to other structural elements and easily allowing for designing for ductile failures.

Finally, and ever more relevant, the ecological aspects related to reusing, recycling, and disposing FRPs is still far from being solved and might pose ever more difficulties to the use of FRP composites. Another aspect that might hinder the adoption of FRP-based strengthening to timber members is that the widespread public perception of timber as an "eco-friendly" material clashes with the image of "plastics", often associated with the omnipresent "plastic pollution".

Zusammenfassung

Der Einsatz von faserverstärkten Polymeren (FRP) im Hochbau hat bis Anfang der 2000er Jahre weltweit deutlich zugenommen. FRP-Verbundwerkstoffe bieten gegenüber anderen Verstärkungsarten einige Vorteile, nämlich sehr hohe Tragwiderstände, Beständigkeit gegen raue Umgebungen und ein hohes Festigkeits-Gewichts-Verhältnis.

Bauteile aus Holz können während der Produktion (hybride Holz-FRP-Verbundbauteile) oder später auf der Baustelle verstärkt werden (FRP-verstärkte Holzbauteile). Die Verstärkung kann passiv (d. h. mit Änderung des Spannungszustandes des Holzbauteils) oder aktiv (z. B. vor- oder nachgespannt und damit den Spannungszustand des verstärkten Bauteils ändernd) erfolgen. FRPs können auch anstelle von Stahlteilen verwendet werden (z.B. anstelle von eingeklebten Stahlstäbe GiRs oder stiftförmigen Verbindungsmitteln). Ein sinnvoller Einsatz von FRP in Holzkonstruktionen erfordert nicht nur Kenntnisse über die Vorteile beider Baustoffe, sondern auch über deren Leistungsgrenzen und Schwachstellen. Dieser Bericht enthält eine ausführliche Beschreibung der Eigenschaften und des Verhaltens beider Materialien.

Es liegen zahlreiche Studien vor, die zum Ziel hatten, das Verhalten von Biegeträgern aus Holz zu verbessern, mit jeweils unterschiedlichen Anforderungen bezüglich Herstellung und Kosten und mit unterschiedlichem Wirkungsgrad. Die gesichtete Literatur deckte mehrere Jahrzehnte ab und zeigt, dass die meisten auf FRP basierenden Verstärkungsmethoden bereits Ende der 2000er Jahre erprobt worden waren. Die am häufigsten untersuchten Methoden bezogen sich auf die Verbesserung des Biegeverhaltens von Brettschichtholz-Trägern. Die Tragfähigkeit und Steifigkeit von BSH-Trägern wurde verbessert, indem FRP-Verbundwerkstoffe direkt auf die Biegezugseite der Träger geklebt wurden, indem sie zwischen die Holzlamellen (und damit vor direkter Exposition geschützt) oder in Längsnuten (z.B. FRO-Stäbe) eingefügt wurden, und auch indem Kunststoff-Fasern in den Klebstoff (normalerweise PRF) eingebettet wurden. Es wurde auch oft berichtet, dass die Erhöhung des Biegewiderstands durch eine Verstärkung zu anderen Arten von unerwünschten Sprödbrüchen, wie z.B. Schubbrüchen, führte. Studien zur Verstärkung mit vorgespannten FRP-Verbundwerkstoffen haben wiederholt keine signifikante Verbesserung gegenüber passiver (d.h. nicht vorgespannter) Verstärkung gezeigt. Ein wichtiger Aspekt in Bezug auf das Verhalten von mit FRP Verbundwerkstoffen verstärkten Holzträgern ist, dass die Duktilität im Gegensatz z. B. zu Stahlbeton nur aus einer Überbewehrung auf Zug resultieren kann, wodurch ein Druckversagen parallel zur den Faserrichtung auf der Biegedruckseite induziert wird. FRP-Verbundwerkstoffe zeigen, im Gegensatz zu Stahlverstärkungen, ein sprödes Bruchverhalten. Die meisten Versuche, über die berichtet wurde, wurden verformungsgesteuert durchgeführt, was in der Regel zu langsam fortschreitendem Versagen führt und eine Spannungsumverteilung ermöglicht. Die entsprechende Kraft-Verformungs-Kurve ist gezackt und zeigt bei abrupten Lastabfällen einen sofortigen Wieder-Anstieg der aufgebrachten Kraft. Dies vermittelt einen falschen Eindruck von Verformungsfähigkeit oder Duktilität. In Wirklichkeit sind die meisten Belastungsszenarien nicht verformungs-, sondern kraftgesteuert, d.h. es ist die auf das Element ausgeübte Last, die allmählich zunimmt, und nicht die Verformung.

Was die Verstärkung von Holz-Bauteilen mit Beanspruchung auf Schub und Zug rechtwinklig zur Faserrichtung betrifft, sind die durchgeführten Forschungsarbeiten deutlich weniger zahlreich. Verstärkungen mit FRP-Verbundwerkstoffen aufgeklebt auf die Seitenflächen von Holz-Bauteilen, sind eine effektive Strategie, aber ästhetisch unattraktiv. Im Bauteilinneren platzierte interne Verstärkungen auf Schub und Zug rechtwinklig zur Faserrichtung mit FRP-Verbundwerkstoffen scheinen keine signifikanten Vorteile gegenüber der heutzutage üblichen Verstärkung mit selbstbohrenden Schrauben zu bieten, ausser vielleicht, wenn es um Feuer geht und freiliegende Stahlteile vermieden werden sollen.

Forschungsarbeiten zur Verstärkung von Stützen aus Holz mit FRP-Verbundwerkstoffen sind ebenfalls wenige durchgeführt worden. Versuche an Prüfkörpern mit praxisgerechten Abmessungen haben gezeigt, dass Stahlverstärkungen möglicherweise besser funktionieren und dass eine Verstärkung auf FRP-Basis nur bei relativ grossen Querschnitten vorteilhaft sein könnte.

Der Einsatz von FRP-Verbundwerkstoffen in Holzverbindungen hat gezeigt, dass FRP-Verbundwerkstoffe erfolgreich eingesetzt werden können, um das Holz im Anschlussbereich zu verstärken oder einige der Verbindungsteile, die üblicherweise aus Stahl bestehen, zu ersetzen. Im ersten Fall auf Kosten zusätzlicher Produktionsschritte und im Wettbewerb mit leistungsfähigen Holzwerkstoffplatten, im zweiten Fall mit dem Resultat einer geringeren Tragfähigkeiten. Der wesentliche Unterschied zwischen FRPs und Stahl, der darin besteht, dass duktile Versagensmodi mit FRPs nicht möglich sind, manifestiert sich auch hier.

Das Verbundverhalten zwischen FRPs und Holz ist umfassend untersucht worden, und der aktuelle Wissensstand erlaubt die Entwicklung zuverlässiger Systeme, in der Form von Kombinationen von verschiedenen Hölzern bzw. Holzwerkstoffen HWS, Klebstoffen und entsprechender Oberflächenbehandlung. Die Dauerhaftigkeit von Holz-FRP-Verbunden hängt in hohem Masse von der spezifischen Beschaffenheit der verwendeten Komponenten und vom Klebeverfahren ab. Es ist jedoch äusserst schwierig, allgemein gültige Aussagen zum Verbundverhalten zwischen Holz bzw. HWS und FRPs zu machen. Dennoch hat die Untersuchung spezifischer Kombinationen von Holz, Klebstoffen und FRPs gezeigt, dass solche Verbunde die gleichen Leistungsanforderungen erfüllen können, die an Holz-Holz-Verklebungen gestellt werden. Angesichts der grossen Vielfalt der Holzarten, der inhärenten Anisotropie und der hohen Variabilität von Holz ist die Entwicklung zuverlässiger allgemeiner Verbund-Modelle komplex und hat erst in den letzten Jahren begonnen.

Das Langzeitverhalten (Kriechverhalten), ist nach wie vor ein offenes Thema, sowohl unter wechselnden Klima- wie auch Belastungsbedingungen (wie dies im Übrigen auch bei unverstärkten Holzbauteilen der Fall ist!). Es sind nur wenige Studien verfügbar. Die meisten Untersuchungen führten zum Ergebnis, dass scheinbar keine Verbesserung des Kriechverhaltens von mit FRP-verstärkten Balken auftritt.

Das Brandverhalten von FRP-verstärkten Holzbauteilen ist stark davon abhängig, wie geschützt der FRP-Verbundwerkstoff innerhalb des Holzquerschnitts ist. Da die Wirkung der Verstärkung deutlich reduziert wird, wenn sich der FRP-Verbundwerkstoff nicht sehr nahe an der Zone der maximalen Beanspruchung befindet, bedeutet die Notwendigkeit, ihn vor direkter Feuereinwirkung zu schützen, in der Regel einen Kompromiss zwischen der Erhöhung des Tragwiderstands bei Normaltemperatur und dem Feuerwiderstand. Wenn die Brandexposition tatsächlich ein Problem darstellt, könnten einige Arten der Verstärkung mit FRP interessanter sein als die entsprechende Verstärkung mit Stahlteilen, da die Wärmeleitfähigkeit von FRPs deutlich geringer ist als diejenige von Stahl. Dies ist vor allem für Verstärkungselemente relevant, die eingebettet sind und den gesamten Querschnitt (oder den grössten Teil davon) durchlaufen, wie z. B. interne Verstärkungen zur Vermeidung von Schub- und Zugversagen rechtwinklig zur Faserrichtung und auch für stiftförmige Verbindungsmittel.

Die auf FRP basierende Verstärkung von Holzbauteilen steht heutzutage in Konkurrenz zu bereits etablierten Mitbewerbern. Für die Verstärkung in Längsrichtung der Holz-Bauteile (d.h. auf Biegung und axialen Druck) ist die Hauptalternative die Verwendung von hochleistungsfähigen Holzwerkstoffen (z.B. Furnierschichtholz LVL aus Fichte oder Buche), welche auf einfache Weise in die bestehenden Produktionsprozesse im Herstellerwerk integriert werden können. Für die Querverstärkung (d. h. bei auf Schub oder auf Zug rechtwinklig zur Faserrichtung beanspruchten Bauteilen) sind die Hauptalternativen selbstbohrende Schrauben und eingeklebte Stahlstäbe. Stahlverstärkungen haben den grossen Vorteil, dass sie leicht bearbeitet und mit anderen Elementen verbunden werden können und dass die Versagensmodi duktil ausgelegt werden können.

Schliesslich, und dies wird immer wichtiger, sind die ökologischen Aspekte im Zusammenhang mit der Wiederverwendung, dem Recycling und der Entsorgung von FRPs noch weit davon entfernt, gelöst zu sein, und könnten dem Einsatz von FRP-Verbundwerkstoffen zunehmend Schwierigkeiten bereiten. Ein weiterer Aspekt, der die Anwendung von FRP-basierten Verstärkungen im Holzbau behindern könnte, ist, dass die weit verbreitete öffentliche Wahrnehmung von Holz als "umweltfreundlicher" Baustoff mit dem Image von "Kunststoffen" kollidiert, das oft mit der allgegenwärtigen "Umweltverschmutzung durch Kunststoffe" in Verbindung gebracht wird.

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1 Introduction

1.1 Background

According to Hollaway (2010), the use of FRP composites in civil engineering falls into the following main categories:

- hybrid systems (FRPs incorporated during production);
- in situ repair / reinforcement of existing structures;
- all-FRP structural members and all-FRP structures.

The use of fibre-reinforced polymers (FRP) in structural engineering has increased significantly worldwide until the early 2000s, in part also because of the applied research and development work carried out at Empa. This research was mainly related to reinforced concrete structures, but was strongly focused on developing practical applications for which FRPs were particularly suited. An important aspect was the cooperation with companies from the private sector, who acquired the knowledge to further develop their products to meet the requirements for structural applications in the construction industry, increasing the available high-performing FRP products for structural design. FRP-based products for structural applications are now available at lower prices and, for certain applications, compete with other high-performing materials such as steel. The main advantages of FRPs are the:

- high strength and stiffness properties in the fibre direction;
- high dimensional stability in the fibre direction;
- corrosion resistance;
- good fatigue behaviour;
- low self-weight.

Experience has shown that the use of FRP composites as reinforcement has some advantages over steel. For example, handling steel profiles in a construction site is considerably more difficult, given their weight, than equivalent FRP profiles. The ease of handling FRP elements might compensate for their higher costs. Another advantage is that FRP has very good corrosion and chemical resistance, which makes it particularly suited for applications in harsh environments. Compared to timber and FRP, steel has high coefficients of thermal expansion, which can lead to additional stresses.

Structural timber elements are usually made from solid timber, glued-laminated timber (GLT), cross-laminated timber (CLT), or wood-based materials with comparable mechanical properties, such as laminated veneer lumber (LVL). Structural timber elements can be easily strengthened with high-strength timber-based products (e.g. adding a softwood or a hardwood-LVL lamination to a GLT beam), but also with steel (e.g. self-tapping screws as perpendicular-to-the-grain reinforcement) or FRPs (e.g. FRP laminate as tension reinforcement between timber laminations in a GLT beam). Timber-based reinforcements have the advantage of being prepared and glued with the same tools and products that are used to assemble the timber elements. Steel reinforcements have the advantage, in addition to high strength and ductility, of being easily processed and adapted (e.g. cut, welded, or threaded) and therefore easy to connected to other structural members. Nowadays, most connection types and reinforcement schemes in timber structures are based on the use of steel parts. However, as research has shown, FRPs could also be used as reinforcement or connection systems.

The most common FRPs use carbon, glass, or aramid fibres. Glass fibres, compared to other fibres, are equally suitable for compressive and tensile loading and their price is significantly lower. However, they exhibit corrosion problems and their strength and stiffness is significantly lower than that of carbon fibres. Aramid fibres are useful when weight reduction, abrasion resistance or resistance to impact is important. Compared to the other two, carbon fibres have higher strength and a significantly higher stiffness. They are also corrosion resistant. Nevertheless, they are significantly more expensive.

Structural timber elements can be strengthened during production (*hybrid timber-FRP composite members*) or reinforced later on site (*FRP-reinforced timber members*). The reinforcement can be *passive* (i.e. applied with changing the stress state of the timber member) or *active* (e.g. pre- or post-tensioned, changing the stress state of the reinforced member). FRPs can also be used instead of steel parts (e.g. as glued-in rods, or as dowel-type fasteners). A sensible use of FRP in timber structures requires knowledge of the performance limits and weak points of both timber and FRPs. This report addresses the properties of both materials and presents an extensive and critical review of the research conducted on the use of FRPs in timber structures since the 1960s until 2020. Other state-of-the-art reviews on the strengthening of structural timber members and on the development of composite and hybrid members have been published, namely by Bulleit (1984), Steiger (2001), Ansell and Smedley (2007), Tlustochowicz et al. (2010), Kasal (2012), Steiger (2014), Steiger et al. (2015), (Franke et al. 2015), Schober et al. (2015), and Schober and Tannert (2016).

1.2 Objectives and overview

The main objectives of this report are to:

- describe the material properties of timber and FRPs (advantages and disadvantages of each material) and compare the properties of FRP with alternative non-FRP strengthening materials;
- review the most relevant research and development studies that have been performed and briefly summarise their results;
- present examples of practical applications.

Therefore, the report is organised as follows:

- Section 2 presents an overview of the properties and behaviour of wood and of modern structural timber members;
- Section 3 gives an overview of the properties and behaviour of FRPs for civil engineering applications;
- Section 4 presents research conducted on the bond behaviour between timber and FRPs;
- Section 5 presents research conducted on the development of hybrid timber-FRP structural elements;
- Section 6 presents research related to on-site reinforcement timber structural elements with FRPs;
- Section 7 addresses other relevant aspects of timber-FRP composites (creep, durability, fire, ecologic aspects)
- Section 8 lists available guidelines for the use of FRPs in structural engineering;
- Section 9 presents some examples of the applications of FRP in timber structures.

1.3 Acronyms and definitions

1.3.1 Acronyms and abbreviations

AFRP – aramid-fibre-reinforced polymer

BFRP – basalt-fibre-reinforced polymer

CA – cellulose acetate

CFRP – carbon-fibre-reinforced polymer

CLT – cross-laminated timber

DVW – densified veneer wood

EBR – externally-bonded reinforcement

EB – externally-bonded

EP – epoxy

FRP – fibre-reinforced polymer

GFRP – glass-fibre-reinforced polymer

GiR – glued-in rod

GLT – glued-laminated timber

IMR – internally-mounted reinforcement

LVL – laminated veneer lumber

MF – melamine

MOE – modulus of elasticity

NSMR – near-surface mounted reinforcement

NSM – near-surface mounted

OSB – oriented strand board

PA – polyamide

PC – polycarbonate

PE – polyethylene

PEI – polyetherimide

PEEK – polyetheretherketone

PES – polyethersulfone

PF – phenolic

PFA – perfluoroalkoxy alkane

PI – polymide resins

PMMA – polymethyl methacrylate)

POM – polyacetal or polyoxymethylene

PP – polypropylene

PRF – phenol resorcinol formaldehyde

PS – polystyrene

PSU – polysulfones

PVC – polyvinyl chloride

PU – polyurethane

SMR – surface mounted reinforcement

SM – surface mounted

TCC – timber-concrete composite

UF – urea formaldehyde

UP – unsaturated polyester

VE – vinyl ester

2 Wood, structural timber, and engineered wood products (EWP)

Wood is a highly anisotropic material, with duration of load-, moisture- and temperature-dependent properties, that exhibits a high variability in its physical and mechanical properties. This high variability is observed not only between different wood species, but also within the same species and even within single timber members. The variability of the physical and mechanical properties within a timber member is mainly due to non-homogeneities, namely structural defects (e.g. knots, grain deviation), which strongly influence its performance (Thelandersson and Larsen 2003). Modern structural timber products are mostly produced from boards and veneers that come from the processing of logs at sawmills. To overcome the abovementioned issues, the source material must the graded to ensure that the processed timber and wood-based products meet specific performance requirements, usually regarding strength and stiffness.

2.1 Structural timber products

The most common structural timber products (Figure 2.1) are:

- **glued-laminated timber (glulam, or GLT)** – structural timber member composed by at least two essentially parallel laminations which may comprise of one or two boards side by side having finished thicknesses from 6 mm up to 45 mm (inclusive) (EN 14080:2013).
- **glued solid timber** – structural timber member with overall cross-sectional sizes not exceeding 280 mm comprising two to five essentially parallel laminations bonded, having the same strength class or manufacturer specific strength class and a finished lamination thickness greater than 45 up to 85 mm (EN 14080:2013).
- **laminated veneer lumber (LVL)** – wood based composite consisting of veneers, glued together predominantly parallel to the direction of the grain in adjacent layers that may have cross band veneers (EN 14374:2004; prEN 14374:2016).
- **cross-laminated timber (CLT)** – structural timber consisting of at least three face-bonded layers which comprise solid timber laminations and may comprise wood-based panels, at least one layer orthogonally oriented to the two adjacent layers (prEN 16351:2018).

2.2 Inhomogeneity

As a naturally grown building material, wood sometimes shows considerable variations in its properties. Depending on the type of wood, and even within the same type of wood, different values result. However, the use of wood as a building material requires a reliable material quality and the design of timber supporting structures must be based on assured material parameters (strength, modulus of elasticity). The natural scattering of material properties can essentially be limited by two different measures: grading and homogenisation.

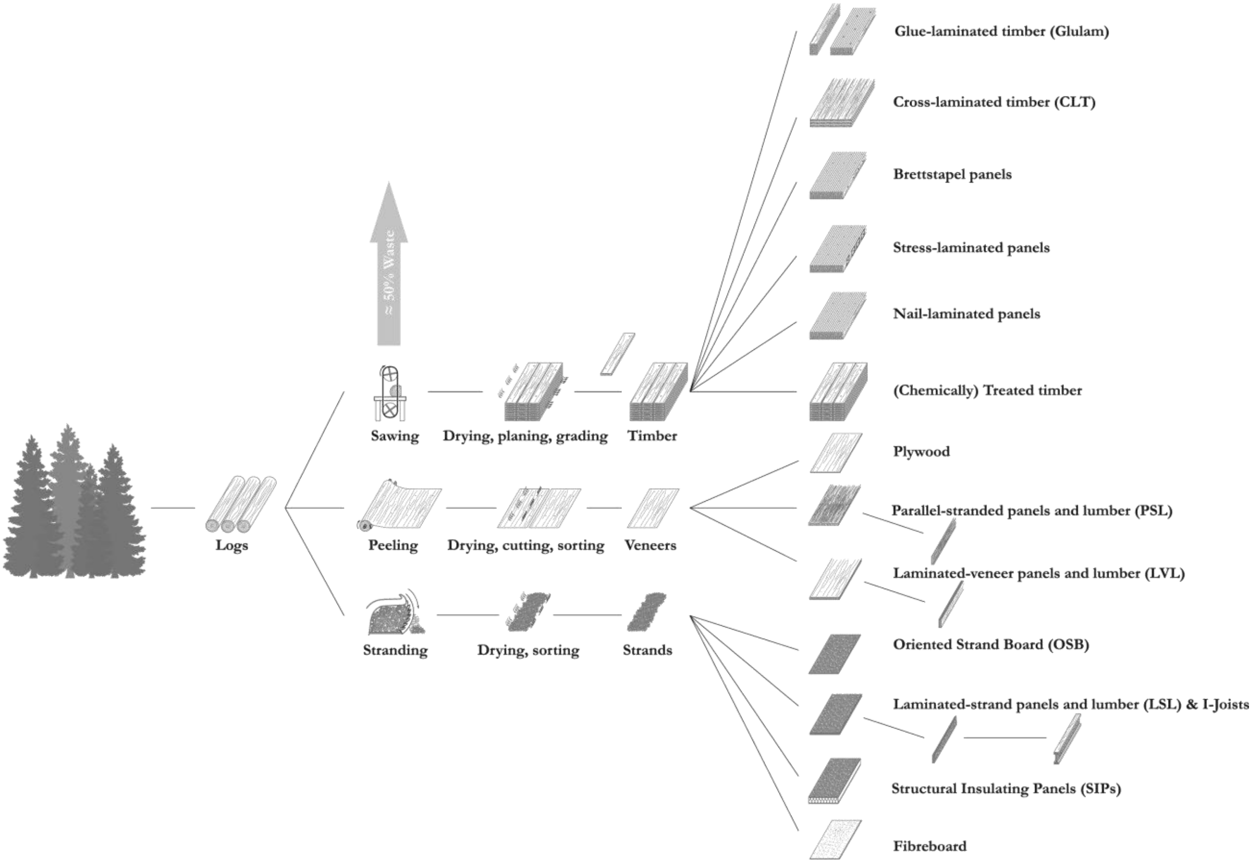


Figure 2.1. The processing chain of engineered wood products. Figure adapted from Ramage et al. (2017).

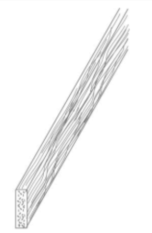
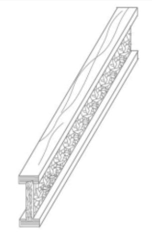
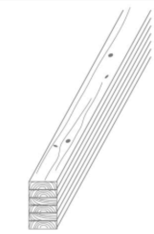
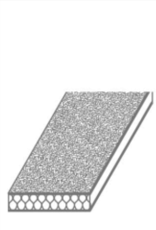
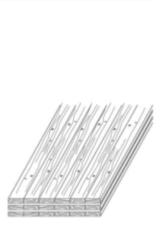
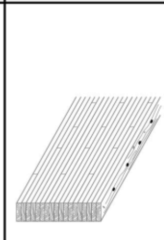
Engineered Timber Product	Parallel Strand Lumber (PSL)	Laminated Veneer Lumber (LVL)	I-Joist	Glulam	Structural Insulating Panel (SIP)	Cross Laminated Timber (CLT)	Brettstappel
Typical Detail							
Application	• Beams • Columns	• Beam • Columns • Cord	• Joist • Beam	• Beam (Long span) • High Loading	• Roof • Wall • Floor	• Roof • Wall • Floor	• Roof • Wall • Floor
Usage	Interior	Interior	Interior	Interior / Exterior	Interior	Interior/ Exterior	Interior/ Exterior

Figure 2.2. Common structural timber products in Europe. Figure adapted from Ramage et al. (2017).

2.2.1 Grading

A limitation of the scatter of mechanical properties can be achieved first and foremost by grading. Besides the conventional and widespread form of visual assessment of the wood, there are also methods of grading by means of non-destructive testing. The *visual grading* based on the visually recognizable and detectable structural features that significantly influence the strength and stiffness of the wood. The classification of the wood is based on the extent of existing defects (knottiness, grain deviation, etc.). *Machine grading* is based on the correlation of physical parameters (modulus of elasticity in bending, ultrasonic time of flight, etc.) that can be measured by a machine with the mechanical properties of timber. While machine grading methods allow for a precise and reliable assignment of timber into different strength classes, visual grading offers only insufficient selectivity and has the additional disadvantage that the correct verification of all grading criteria is connected with an effort that is hard to implement in practice in an industrial scale.

2.2.2 Homogenization

A homogenization effect can be achieved by splitting the wood into components of different sizes (boards, veneers, chips, etc.) and subsequently joining them to form engineering-wood products (laminated veneer lumber, veneer strip lumber, etc.) or semi-finished products (laminated glulam, squared lumber, etc.). Although the maximum values of the raw material round wood can no longer be reached after cutting, the scattering, which is reduced to a considerable extent by homogenization, leads to a stabilization of the average value and to a significant increase in the lower fractiles of the distribution. This is of central importance because the design is based on these lower fractile values (usually the 5% fractiles). If the components are additionally graded by strength before they are added together, the effect is intensified. Figure 2.3 shows the effect of homogenization just described using the example of squared lumber being cut into boards and subsequently glued together to form glulam with and without sorting of the lamellae.

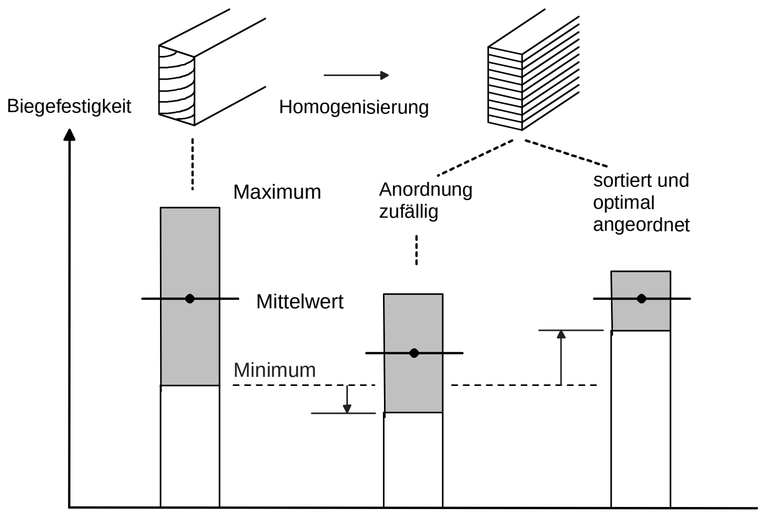


Figure 2.3. Homogenization through disassembly and optimal assembly.

2.2.3 Anisotropy

Due to its anatomical structure, wood is a distinctly anisotropic building material with good strength and stiffness properties in the longitudinal direction of the fibres and lower strength and stiffness across the fibres. For the frequently used wood species Norway spruce (*picea abies*), the transverse compressive strength is approx. 10% of the longitudinal compressive strength and the transverse tensile strength is approx. 3% of the longitudinal tensile strength. The ratio between the modulus of elasticity parallel to the grain and that perpendicular to the grain is approximately 30:1. As an example, the design approaches (permissible stresses) for "Normal" GLT of strength class B from the Swiss timber design standard SIA 164 (1981/92) are shown in Figure 2.4, illustrating the strong dependence of the strength properties on the angle of force to the grain.

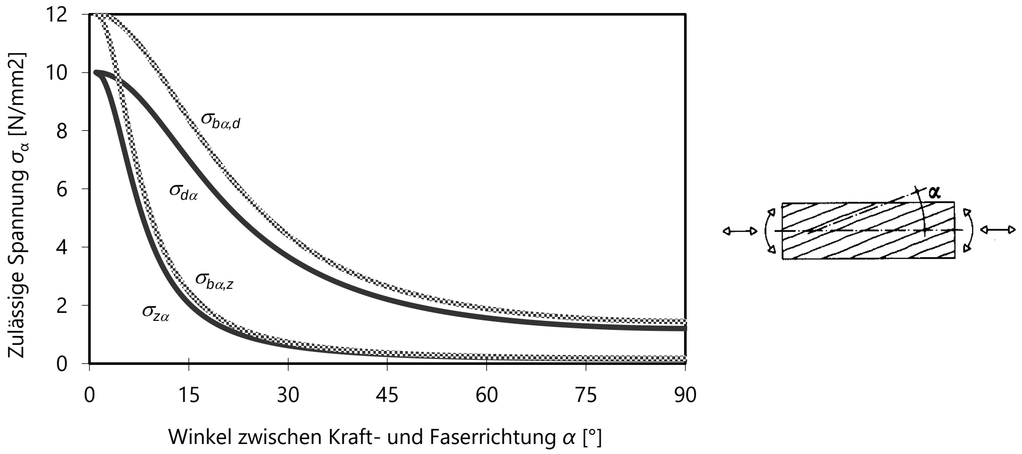


Figure 2.4. Permissible tension at an angle to the grain for glued laminated timber of strength class B according to SIA 164 (1981/92): σ_z = tension, σ_d = compression, $\sigma_{b,z}$ = bending tension, $\sigma_{b,d}$ = bending compression.

If the wood is predominantly stressed in the longitudinal direction of the fibres, usually parallel to the longitudinal direction of the structural elements, the effects of anisotropy become less important. Nevertheless, in zones of force application (connections, joints, supports), stresses occur transverse to the axis of the beam and thus transverse to the grain direction of the wood. The correct dimensioning of such problem zones and the well thought-out design solution is of great importance, especially in timber engineering, where large forces often have to be transmitted.

2.3 Mechanical properties and fracture behaviour

Apart from a few special cases, wood is a material with linear-elastic behaviour until it fails. For short-term loads, as long as the stresses remain small, one can assume the validity of Hooke's law and then talk about elastic deformations, which are linearly linked to the stresses via the modulus of elasticity.

Tables 2.1 and 2.2 give an overview of the mechanical properties (characteristic values) of structural timber and glulam (GLT) for a strength classification according to EN 338:2016 and EN 14080:2013.

Table 2.1. Characteristic properties of sawn timber from spruce (*picea abies*) / fir (*Abies alba*) (EN 338:2016).

f, E, G: [N·mm^{-2}] ρ: [kg·m^{-3}]		C14	C16	C18	C20	C22	C24	C27	C30	C35	C40
Bending strength	$f_{m,k}$	14	16	18	20	22	24	27	30	35	40
Compressive strength	$f_{c,0,k}$	16	17	18	19	20	21	22	23	25	26
	$f_{c,90,k}$	2.0	2.2	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9
Tensile strength	$f_{t,0,k}$	7.2	8.5	10	11.5	13	14.5	16.5	19	22.5	26
	$f_{t,90,k}$	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
Shear strength	$f_{v,k}$	3.0	3.2	3.4	3.6	3.8	4.0	4.0	4.0	4.0	4.0
MOE	$E_{m,0,mean}$	7'000	8'000	9'000	9'500	10'000	11'000	12'000	12'000	13'000	14'000
	$E_{m,0,k}$	4'700	5'400	6'000	6'400	7'400	7'700	8'000	8'700	9'400	10'700
	$E_{m,90,k}$	230	270	300	320	330	370	380	400	430	470
Shear modulus	G_{mean}	440	500	560	590	630	690	720	750	810	880
Density	ρ_{k}	290	310	320	330	340	350	370	380	400	420
	ρ_{mean}	350	370	380	390	410	420	450	460	480	500

Table 2.2. Characteristic properties of GLT made of spruce (*picea abies*) / fir (*Abies alba*) (EN 14080:2013).

f, E, G: [N·mm^{-2}] ρ: [kg·m^{-3}]		GL 24h	GL 28h	GL 32h	GL 24c	GL 28c	GL 32c
Bending strength	$f_{m,g,k}$	24	28	32	24	28	32
Compressive strength	$f_{c,0,\ g,k}$	16	19.5	22.5	14	16.5	19.5
	$f_{c,90,\ g,k}$	0.4	0.45	0.5	0.35	0.4	0.45
Tensile strength	$f_{t,0,\ g,k}$	20	26.5	29	21	24	26.5
	$f_{t,90,\ g,k}$	2.7	3.0	3.3	2.4	2.7	3.0
Shear strength	$f_{v,\ g,k}$	2.7	3.2	3.8	2.2	2.7	3.2
MOE	$E_{m,mean}$	11'500	12'600	14'200	11'000	12'500	13'500
	$E_{m,k}$	9'600	10'500	11'800	9'100	10'400	11'200
Shear modulus	$G_{g,mean}$	650					
	$G_{g,k}$	540					
Density	$\rho_{g,k}$	385	425	440	365	390	400
	$\rho_{g,mean}$	420	460	490	400	420	440

Table 2.3. Comparison of the specific strengths of different materials.

Material		Limit tensile stresses [N·mm^{-2}]	Density γ [kg·m^{-3}]	Specific strength $f_{t,y}/\gamma$ [m]
Steel Fe E 235	Yield strength f_y	235	78.5	3'000
Steel Fe E 355		355	78.5	4'500
Steel S 500		460	78.5	5'850
High-strength steel		1'600	78.5	20'380
Aluminium		200	27	7'400
GFRP	Tensile strength f_t	400	17.5	22'860
CFRP		2'000	18	111'100
Solid timber C 24		14	3.5	4'000
GLT GL 28		19.5	4.1	4'750
GLT GL 36		26	4.5	5'780

Obviously, wood has an extraordinarily high strength in relation to its density. The specific strength parallel to the grain, i.e. the strength divided by the density, reaches values that are not inferior to those of high-quality steel. The same applies to the modulus of elasticity. Table 2.3 shows a comparison of the specific tensile strengths of some important materials used in building construction. The specific tensile strength, calculated from the ratio of the tensile limit stress and the density of the material, can be thought of as the length at which a tensile bar made of the corresponding material would break under its own weight.

The stress-strain behaviour of wood is completely different depending on the type of stress. For small specimens without structural defects, the behaviour under tensile and compressive stress parallel to the grain is shown in Figure 2.5 (Dubas et al. 1981). For structural timber of normal and poorer quality, the tensile strength parallel to the grain is significantly lower than the compressive strength. High-quality structural timber has practically identical tensile and compressive strengths parallel to the grain. Corresponding characteristic values are listed in Tables 2.1 and 2.2. The elongation at failure of structural timber for a tensile or compressive stress parallel to the grain is in the order of 0.2 to 0.25% at the 5% fractile level, which is decisive for structural design.

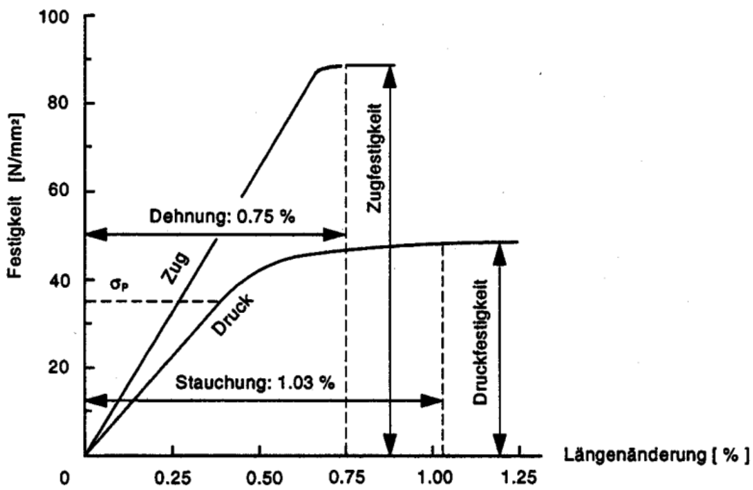


Figure 2.5. Stress-strain behaviour of wood under tensile and compressive stress (Dubas et al. 1981).

2.3.1 Tension

Under tensile stress, the behaviour is practically linear elastic until a brittle failure occurs. Inclined fibres considerably reduce the tensile strength (cf. Figure 2.4). Perpendicular to the fibre, the tensile strength is close to zero. The tensile strength is greatly reduced by structural defects (e.g. knots), and this is due to the combined effect of several influences: Inclined fibre in the area of the disturbance itself, as well as stress concentrations at the edge of the disturbances, the so-called notch effect.

2.3.2 Compression

The failure of components under axial compressive load is given by exceeding the buckling load or the compressive strength parallel to the grain, depending on the slenderness. In the case of slender elements, the load-carrying capacity is limited by lateral buckling, far below the compressive strength. In the case of stocky elements under compressive axial load, the fibres deflect locally to the side when the compressive strength is reached. The behaviour in the failed state is quasi-plastic. At low compressive stresses, below the proportional limit, the stress/strain relationship is linear elastic.

The influence of the knots and the oblique fibres on the compressive strength is not as pronounced as on the tensile strength (Figure 2.4). The decrease in strength under compression parallel to the grain is mainly due to the fibre deviations in the areas around knots, which promote premature local buckling of the fibres. The actual compressive strength in the direction parallel to the grain can hardly be defined, since even at low stress levels plastic deformations occur. A limitation of the transverse compressive stresses is therefore usually achieved by introducing a deformation criterion.

When subjected to compressive stresses perpendicular to the grain plastic deformations develop already at very low stress levels (of 1 MPa for Norway spruce for instance). The modulus of elasticity perpendicular to the grain is significantly lower than parallel to the grain.

2.3.3 Bending

The behaviour in bending is a combination of the behaviour in (axial) tension and in (axial) compression. The most common design approach for wood components in bending is based on a linear distribution of stresses over the cross-section (Figure 2.6, left) (Zakic 1973). This assumption is only correct if failure occurs by exceeding the tensile strength, before the stresses on the compression side have exceeded the proportional limit. The bending behaviour of normal structural timber is quite accurately represented by the linear model, since the structural defects reduce the tensile strength significantly more than the compressive strength. With high-quality structural timber and also with GLT, on the other hand, compression failure can occur on the compression side, which causes the neutral axis to shift downwards (Figure 2.6, middle). Failure in such cases will also occur on the tensile side. The effective load-carrying capacity must then be calculated taking into account the non-linear stress-strain behaviour in the compression zone (Figure 2.6, right).

There are different models for the non-linear behaviour of wood in bending. Figure 2.6 shows the model of Zakic (1973), which assumes a 2nd order parabola in the compression zone and a linear model in the tension zone. Zakic (1973) uses a uniform modulus of elasticity for the tensile and the compression zone and shows that the degree of plastification depends on the ratio of tensile and compression strength, i.e. directly on the strength class or the material quality.

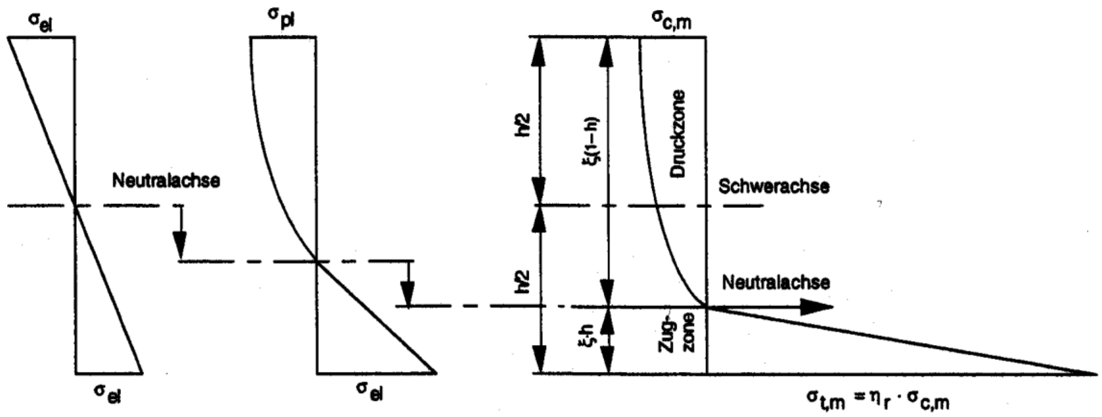


Figure 2.6. Stress-strain behaviour for wood in bending according to Zakic (1973).

Glos (1978) developped an empirical function for the stress-strain behaviour in the compression zone (Figure 2.7), based on more than 900 compression tests on Norway spruce boards. This curve is determined by the flexural compressive strength $f_{c,m}$, the modulus of elasticity $E_{c,0}$, the strain at maximum compressive strength $\varepsilon_{c,m,u}$, and the asymptotic final strength $f_{c,m,A}$.

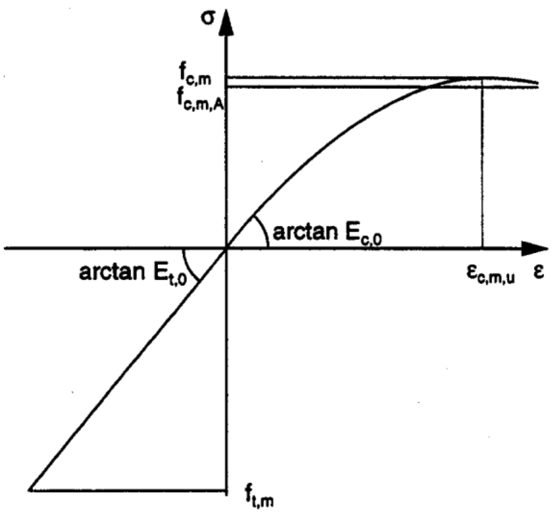


Figure 2.7. Stress-strain behaviour of wood in bending according to Glos (1978).

2.3.4 Shear

The shear strength of wood parallel to the fibre is quite low and is about 5 to 8% of the compressive strength. Perpendicular-to-the-grain shear fractures are hardly possible if the applied force acts as a compressive force; the wood is crushed in the load introduction zones rather than being subjected to shear failures. However, if a combination of transverse tensile stress and shear force is applied, the wood will fail even at very low forces. Furthermore, the shear strength is strongly reduced in case of cracks.

2.4 Rheological properties

Under long-term loads, time-dependent plastic deformations occur that must be added to the immediate elastic deformations. This increase in deformation is called creep and can be observed in all building materials, in wood and concrete already at normal temperatures and in steel only at elevated temperatures. This behaviour can be characterized schematically as shown in Figure 2.8 (Dubas et al. 1981).

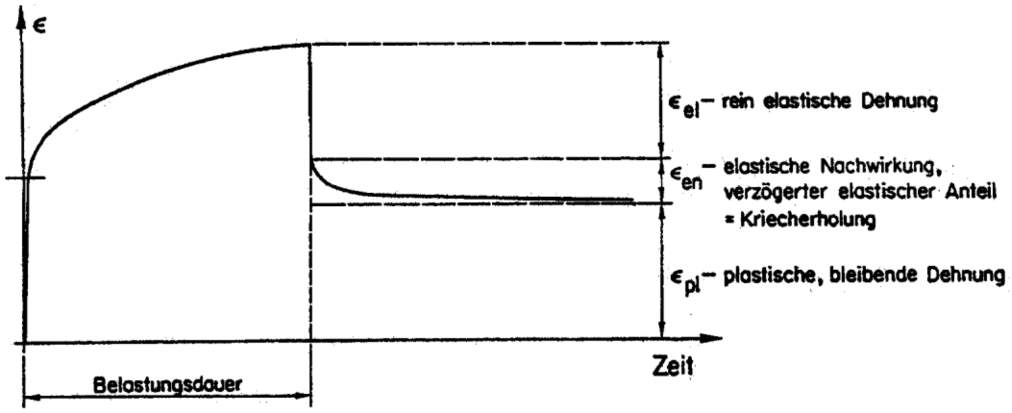


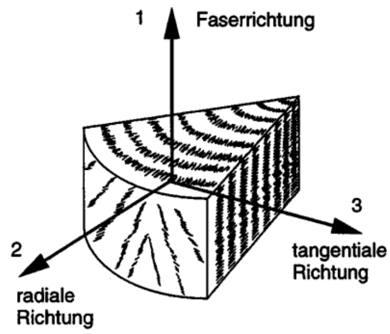
Figure 2.8. Schematic representation of the creep behaviour of wood (Dubas et al. 1981).

When the load is relieved, the elastic deformation ε_{el} recovers, whereas the creep deformation ε_k is only recovers to a practically negligible extent (ε_{en}, creep recovery, delayed elasticity). A permanent plastic deformation ε_{pl} remains even in the unloaded state.

The amount of creep deformation depends on the level of stress (in relation to the strength under short-term load), the type of stress (tension, compression, bending, shear) and the wood moisture content.

2.4.1 Influence of moisture

When subjected to changing moisture content, the volume of wood does not remain constant. The woods in addition tends to crack, especially when exposed to weathering. The shrinkage and swelling coefficients are different in the different directions due to anisotropy (see Figure 2.9). Shrinkage cracks reduce, in particular, the shear strength.



$\alpha_t : \alpha_r : \alpha_\ell = 2 : 1 : 1/20$

Figure 2.9. Definition of a principal axis system according to the wood anatomy and axis-specific shrinkage and swelling coefficients.

2.4.2 Thermal behaviour

The temperature expansion in the longitudinal direction is about 3 to $6\times10^{-6}\cdot C^{-1}$. This is about one third of the thermal expansion of steel. Wood is a good heat insulator and is affected by short-term temperature changes to a lesser extent. Thermal expansion is therefore of little importance, all the more so as it is far less than the effects of shrinkage or swelling. In addition, it must be taken into account that a long-term increase in temperature leads to evaporation and thus to shrinkage, so that both influences partially compensate each other. The ratio between the temperature expansion along the fibres and perpendicularly to the fibres is 1:10 to 1:15. The heat conduction of wood depends on the density, the water content and the fibre direction. Wood has good thermal insulation properties, a small heat storage capacity and a high surface temperature. This is particularly beneficial in the event of a fire.

2.4.3 Durability

Assuming optimal structural design and proper maintenance, the service life of timber structures is not less than that of steel or reinforced concrete structures and is in the order of 100 years. Compared to the effective service life of buildings, this is usually sufficient. Buildings are often demolished much earlier because their shape, size, structure, etc. no longer meet the changed requirements for their use.

2.4.4 Other aspects

2.4.4.1 Construction, processing, transport and assembly

The available cross-sections of construction timber and its length are limited. To get larger structural elements, either composite cross-sections (e.g. by gluing) or joints are required, with additional material and processing costs and inevitable weakening of the element.

Wood is easy to process and, thanks to modern wood-glued construction, there are hardly any limits to the design. Wooden buildings are characterized by an objectively appealing aesthetic.

Because of its high specific strength and rigidity, timber construction uses components that are light, extremely strong and at the same time rigid. Timber construction is therefore a typical assembly construction method, in which essential parts of the production are carried out in workshop and are thus protected from unfavourable weather conditions. Once available or "set" in the workshop, erection is a short phase of a few days, even for larger structures.

2.4.4.2 Chemical resistance

Wood is resistant to corrosive environments and chemical influences.

2.4.4.3 Fire behaviour

Wood is combustible and contributes to the spread of fire in buildings. With larger cross-sections, as used in particular in timber engineering, charring occurs slowly during fire exposure. Beneath the charred layer, due to the low thermal conductivity of wood, timber remains intact and with its load-carrying capacity unaffected.

2.5 Connections

Given the strong prefabrication component in modern timber construction, connections are a key element of the global structural behaviour. Since most connections tend to be weaker and much less stiff than the members being connected, selecting a connection type requires considering several aspects: load-carrying capacity, stiffness, fire resistance, cost efficiency, production process, erection method, and the preferences of the structural engineer or architects.

The main connection types in modern timber structures are connections with dowel-type fasteners, connections with glued-in rods, and screwed connections. Because of the development of CNC production techniques, carpenter connections experienced a recent comeback, but their ability to transfer significant loads or withstand load cycles is quite limited.

2.5.1 Connections with dowel-type fasteners

The most common types of structural connections are connections with metal dowel-type fasteners inserted through the connected members (Figure 2.10). Dowel-type fasteners are slender, cylindrical steel elements with smooth, grooved, or threaded surfaces and include nails, staples, bolts, screws, dowels and threaded rods. Load transfer occurs along the shank of the fasteners: the connected members take embedment stresses; the fasteners are mostly loaded in bending and tension (shear becomes important for high-strength timber elements such as densified veneer wood (DVW) or hardwood LVL).

Dowels are slender, cylindrical fasteners with smooth or even slightly grooved surfaces, with a diameter between 6 and 30 mm (tolerance of -0.1/+0.5 mm). Holes for dowels in timber members must be predrilled with the nominal diameter; holes in steel plates for steel-to-timber joints may be 1 mm larger than the dowel diameter. Dowelled connections excel when it comes to transferring large forces and are economic and easy to set up. **Bolts** are dowel-type fasteners with threaded ends (or a fully threaded shank) into which nuts and washers are fixed. They are inserted into predrilled holes and then tightened to ensure that the wood or steel parts are secured in close proximity. Bolts might need to be retightened in case shrinkage of the timber members occurs. The predrilled holes may be up to 1 mm larger than the diameter of the bolt. Bolted connections are less rigid than dowelled connections, due to the oversized holes, and should be avoided when high structural rigidity is required. Structural design of connections with dowel-type fasteners is covered by EN 1995-1-1:2004.

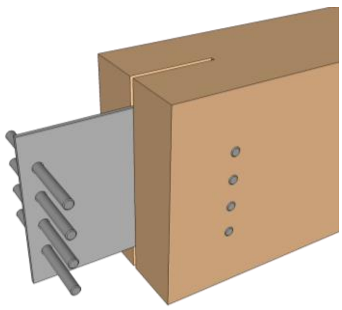


Figure 2.10. Connections with dowel-type fasteners.

2.5.2 Glued-in rods

Connections with glued-in rods are mainly used to transfer high parallel-to-the-grain forces between GLT or LVL members and to prevent cracks in curved and notched beams, due to the presence of tensile stresses perpendicular to the grain. Glued-in steel rods are also used for rehabilitation and repair, during which they may be subject to lateral, axial, or combined loading. Design of connections with glued-in steel rods is not included in EN 1995-1-1:2004, but attempts are currently undertaken to cover this gap. The advantages of using glued-in rods are the ability to transfer very significant loads in the direction parallel to the grain, very high connection rigidity (for axial loads) inherent fire resistance due to the absence of exposed steel parts.

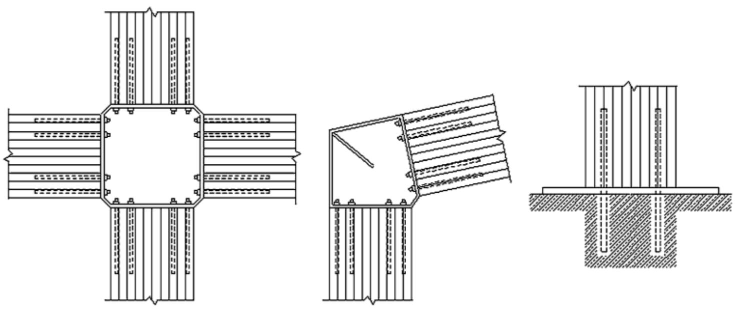
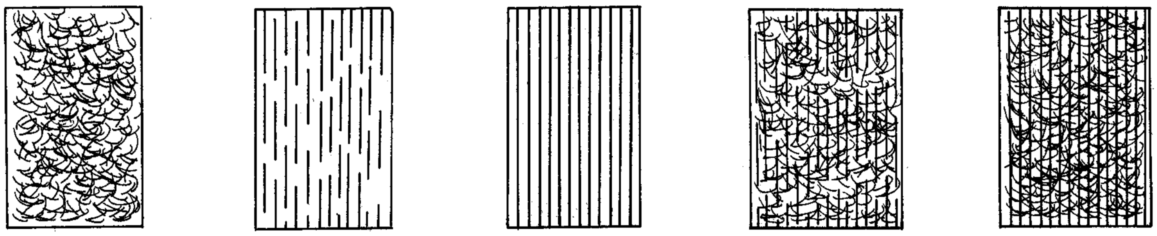


Figure 2.11. Connections with glued-in rods (Tlustochowicz et al. 2010).

3 Fibre-reinforced polymer (FRP) composites in civil engineering structures

3.1 Fibre-reinforced polymer (FRP)

By embedding high-modulus reinforcing materials in the form of fibres in a binder (matrix), a so-called fibre-reinforced composite is created. The mechanical and physical properties obtained in this process are significantly higher than those of the pure matrix. The matrix can be formed by a plastic or by another material, e.g. concrete, metal alloy ceramics or carbon-based. If plastics are used as binding agents, the term resin is also used. The length and orientation of the reinforcing fibres is decisive for the performance of the fibre-reinforced material (Figure 3.1). Moulded parts are produced using short fibres. However, only the use of long and continuous fibres brings significant increases in strength.



non-directional fibres | Unidirectional long fibres | unidirectional continuous filaments | Combination of non-directional with long fibres | Combination of non-directional with continuous fibres

Figure 3.1. Types of fibre composites (Michaeli and Wegener 1989).

Fibre-reinforced composites with polymer-based matrices, so-called fibre-reinforced polymers (FRP), have established themselves in many industrial sectors (transportation, construction, etc.). The most interesting property of FRPs is the ability to adjust their production to fulfil a wide range of requirements. They can have very low specific weight, high strength in the direction of the fibres, and a stiffness that can be adjusted within wide limits. In addition to their general resistance to chemicals, FRPs can also exhibit good damping properties and high fatigue resistance. An important aspect is the good workability both in production and in post-processing. During assembly, the low specific weight of the FRP is also an advantage.

Resistance to heat and thermal shocks, required by components that are exposed to high operating or ambient temperatures, must be met by composite materials with ceramic or carbon matrices and correspondingly resilient fibres.

The properties of FRPs are primarily determined by the reinforcing fibres and are therefore variable within wide limits. An essential advantage of FRPs is therefore the possibility to adjust the degree of orthotropy or anisotropy by orienting the fibres and having them is layers with different orientations.

Some advantages and disadvantages of FRPs are summarised in Table 3.1.

Table 3.1. Advantages and disadvantages of FRPs (Michaeli and Wegener 1989).

Advantages of FRPs	Disadvantages of FRPs
High mass-specific stiffness and very high strength in the fibre direction	Poor behaviour under stresses perpendicular to the fibre direction
Low bulk density (easy to handle)	Sensitive to irregularities
Endless delivery length, no joints	Special knowledge required
Good fatigue properties	Brittle failures
Good chemical resistance	Increased efforts in quality control / management
Low thermal expansion	Low temperature resistance
Low energy consumption for production	Relatively little experience available
	Very difficult to recycle or reuse.

3.2 Fibres

The main tasks of the fibres are to provide the composite material with the necessary mechanical properties. The reinforcing fibres are available in different forms as textiles or rovings. Which form to be used depends mainly on the chosen manufacturing method. The most common fibres used in the building industry are glass, aramid and carbon fibres. In order for the fibre to effectively reinforce the plastic, it must have a higher modulus of elasticity and a higher strength than the matrix material. In addition, there must be good compatibility with the binder (adhesion) and good processability.

3.2.1 Glass fibres (G-fibres)

For production, threads with diameters of 7 to 13 µm in various qualities are drawn from molten glass. Unlike carbon and aramid fibres, glass fibres are isotropic (same properties perpendicular and parallel to the fibre). They do not conduct electric current. They have a temperature resistance up to 250 °C. Glass fibres degenerate under the influence of moisture and UV light and suffer a reduction in strength in a strongly alkaline environment.

For many purposes and therefore widely used is the E-glass fibre. "E" stands for "electrical", as this fibre was developed for electrical applications. In addition, there are also R- and S-glass fibres. Despite the different names they mean the same ("R" stands for "resistance" in French and "S" for strength in English). The C-glass fibre has particularly good chemical resistance.

3.2.2 Aramid fibres (A-fibres)

Aramid fibres are organic, synthetic chemical fibres. The elementary fibre, like the glass fibre, is round and not structured on the surface. The fibre diameter is about 12 µm. Aramid fibres are considerably less sensitive to mechanical stress during processing than glass and carbon fibres. However, they easily absorb moisture. UV light and heat can damage the fibres. Aramid fibres are flame resistant, self-extinguishing and resistant to many chemicals. They have a negative coefficient of thermal expansion, which is higher than that of carbon fibres.

Due to their high strength and stiffness, aramid fibres are well suited for use in the building industry. The good impact strength combined with low weight makes them particularly suited for components that have to be very light and are exposed to dynamic, impact-like forces.

3.2.3 Carbon fibres (C-fibres)

Carbon fibres are inorganic fibres, which are today produced industrially mainly by the decomposition of organic fibres. They consist of over 90% carbon and have a diameter of 5 to 10 µm. Modulus of elasticity and strength can vary in a wide range (see below). The actual values depend on the degree of orientation of the carbon layers involved in the structure and the defects that form in the fibre during production. In contrast to plastics, C-fibres have a progressive stress-strain behaviour, i.e. the modulus of elasticity increases with increasing load. Carbon fibres are electrically conductive. In addition, the C-fibres are almost fatigue resistant, but very sensitive to kinking. C-fibres are chemically inert and have a negative coefficient of thermal expansion in the fibre direction. Together with matrix materials with positive coefficients of expansion, composites can be produced, extremely dimensionally stable over a wide range of temperatures.

Various types of carbon fibres are available on the market:

- HT-fibres: high tenacity / high strength
- IM-fibres: intermediate modulus / medium stiffness
- HM-fibres: high modulus / high stiffness
- UHM-fibres: ultra-high modulus / very high stiffness
- HST-fibres: high strain and tenacity / high elongation and high strength

HM fibres have the disadvantage of low elongation at failure. A further disadvantage is the associated brittleness, which makes the fibres very sensitive to impact forces. Such impact forces are best tolerated by materials that have a high energy-dissipation capacity. The requirement for high elongation at high stresses is met by HST fibres. The group of IM-fibres is a compromise between the HST and HM fibres in that both strength and stiffness have good values.

C-fibres are particularly suitable for applications where high strength and stiffness are required.

3.2.4 Comparison of fibre properties

Table 3.2 summarises the most important mechanical and physical properties for the reinforcing fibres mainly used in construction. The listed strengths only apply if the fibres are undamaged. With careless post-processing, the strength can drop to approximately 2/3 or even only 1/3 of the original value.

Table 3.2. Comparison of the most important fibre characteristics (Michaeli and Wegener 1989).

Key aspects		Glass fibres E	Aramid fibres HM	Carbon fibres HT	Glass fibres IM	Aramid fibres HM	Carbon fibres UHM
Tensile strength	[$kN \cdot mm^{-2}$]	2.5	2.9	3.8	5.6	2.7	2.6
MOE	[$kN \cdot mm^{-2}$]	72	120	230	290	400	630
Failure strain	[%]	3.3-4.8	2.3	1.8	1.8	0.7	0.4
Density	[$g \cdot cm^{-3}$]	2.6	1.4	1.8	1.8	1.82	2.1
Thermal expansion coefficient	[K^{-1}]	5×10^{-6}	-3.5×10^{-6}	0.3×10^{-6}	0.1×10^{-6}	-0.7×10^{-6}	-1.1×10^{-6}

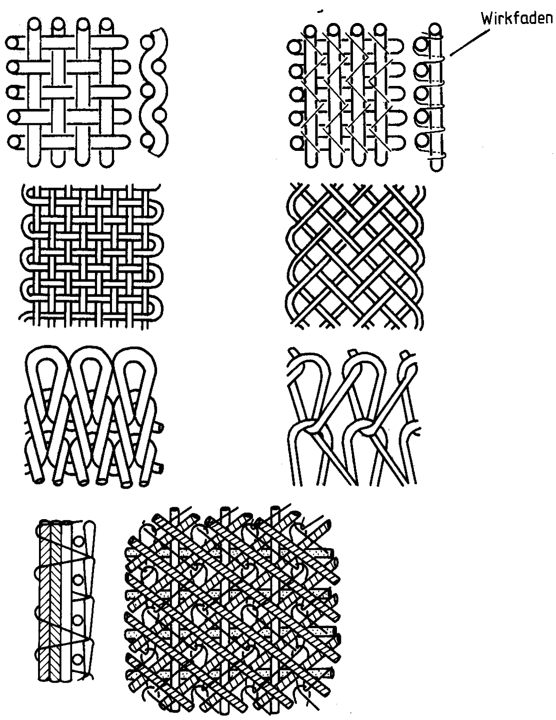
3.2.5 Types of reinforcement

Depending on the application purpose, the fibre reinforcement is arranged uni-directionally, bi-directionally or even multi-directionally. Bi-directional reinforcing elements usually consist of two orthogonal layers. Flat arrangements of fibres are called textile fabrics / semi-finished products. They can be divided into three main groups:

- Non wovens (are hardly used for the production of FRP)
- non mesh-forming systems (woven fabrics, scrims, braids) and
- stitch-forming systems (knitted fabrics).

The general advantage of semi-finished textile products is their economical processing, as many fibres can be processed simultaneously. If such semi-finished products are impregnated directly in the matrix, they are called pre-pregs.

Multiaxial reinforcement is useful where multi-axial stress states occur.



Woven plain fabric

Woven braided fabric

Warp-knitted fabric

Multiaxial warp-knitted fabric

Figure 3.2. Different types of semi-finished textile products (Michaeli and Wegener 1989).

3.3 Adhesives

The matrix, also called resin in the case of plastics, does not have high strength. Nevertheless, it must perform tasks in conjunction with the fibres. The most important tasks of the resin are:

- to transmit forces into the fibres or to pass them on to the next fibre;
- to ensure the geometric position of the fibres;
- to support the fibres when subjected to compressive stresses (buckling);
- ensure that the external shape of the component is maintained;
- to protect the fibre from environmental influences.

Plastics are macromolecular compounds that are created synthetically or by converting natural products. Plastics consisting of linear or branched molecular threads result in thermoplastics. At temperatures above the service temperature, they change into a plastic, soft state that can be formed under pressure and can be melted and dissolved again.

It is possible to connect molecule chains to a greater or lesser extent by means of *cross links*. The degree of cross-linking (Figure 3.3) has a significant influence on the temperature-dependent behaviour and on the hardness and brittleness of the plastic. If there are only a few cross links between individual molecules, i.e. if there is wide-meshed cross-linking, such plastics cannot be re-melted. They are also not soluble, but are swellable. Such materials are called elastomers (rubber or rubber-elastic materials) if they have such a large chain mobility that they are in a rubber-elastic state.

With increasing cross-linking, the material becomes harder and more brittle and is consequently no longer meltable. It is also neither swellable nor soluble. Such strongly cross-linked plastics are called thermosets (synonyms: thermodure, duromers). Thermosetting plastics behave above the service temperature with very limited deformability, but are not malleable.

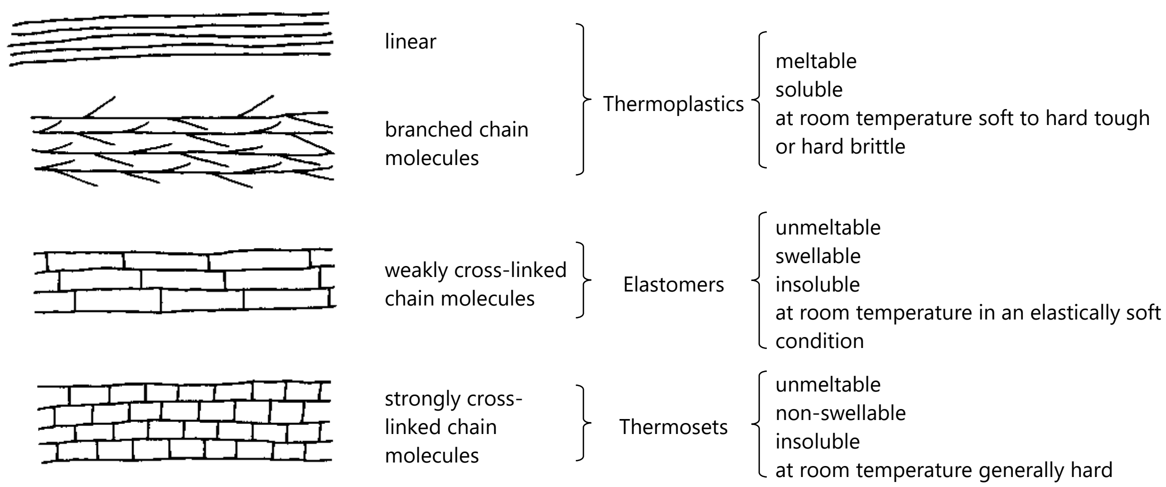


Figure 3.3. Schematic representation of the arrangement of the chain molecules in plastics and their properties (Menges et al. 2011).

The choice of suitable matrix materials is mainly dictated by the application conditions of the later composite material. Fibre-reinforced thermosets are currently still in first place, both in terms of their economic importance and in terms of quality and technology. For some years now, however, fibre-reinforced thermoplastics have also been gaining in importance for demanding technical tasks. Due to their rubber-like behaviour, elastomers are not usually used as binders for FRPs. In the following, therefore, only thermoplastics and thermosets will be discussed in more detail.

3.3.1 Thermosets

Thermosets cure both without pressure and in cold conditions as well as under pressure and heat. The molecule chains are closely cross-linked (Figure 3.4) and generally cannot be dissolved in organic solvents. They also do not melt under the influence of temperature, but decompose or carbonize. At normal temperature, thermosets are hard and brittle. Depending on the degree of cross-linking, they can become visco-plastic when heated. They are not plastically deformable, cannot be melted or welded, are insoluble, but have limited swelling properties.

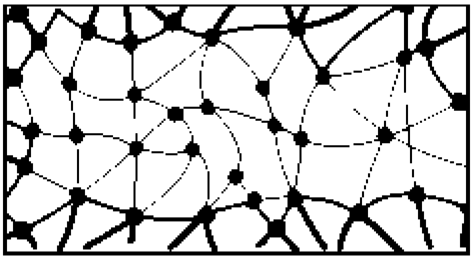


Figure 3.4. Network structure of thermosets (Michaeli and Wegener 1989).

Depending on the degree of cross-linking, the brittleness and heat resistance of the reaction product can be adjusted, whereby at a high degree of cross-linking both heat resistance and brittleness increase. With the increase in brittleness, the strain at failure, the energy dissipation, and the resistance to chemicals are reduced at the same time. A compromise must therefore always be made between the required heat resistance and ductility during the curing reaction.

The most important thermosets are (for FRPs, the relevant resins are in bold):

- **unsaturated polyester (UP) resins**
- **vinyl ester (VE) resins**
- **epoxy (EP) resins**
- **phenolic (PF) resins**
- polyurethane (PU)
- melamine formaldehyde (MF) resins
- urea (UF) resins
- casein-formaldehyde

3.3.1.1 Unsaturated polyester (UP) resins

UP resins have been known since 1936 and have since then found wide application in FRP production, mainly because of their ease of handling. However, they are rarely used for modern high performance FRP. UP are dissolved in styrene and have a low glass transition temperature. The use of the solvent styrene requires the vapours to be extracted and targeted ventilation at the workplace in order to prevent damage to health. Compared with other resins, UP have only moderate mechanical properties and low chemical resistance. They are also brittle and have the disadvantage that they shrink during the extremely exothermic curing process. The use of UP resins is recommended when their limited mechanical properties can be reconciled with correspondingly low requirements. This means that UP resins are usually reinforced with long or short glass fibres and only rarely with high-quality aramid or carbon fibres.

3.3.1.2 Vinyl ester (VE) resins

VE resins have better mechanical properties than UP resins, and their heat and chemical resistance can be specifically optimised. Shrinkage during curing is much lower than with UP resins.

3.3.1.3 Epoxy (EP) resins

EP resins have been manufactured since 1938 and are by far the most widely used matrix resins for modern high-performance composites. It is a 2-component system consisting of so-called epoxies, which cure when mixed with hardeners. Depending on the choice of epoxies and hardeners, properties such as heat resistance (up to 250°C), plasticity and the curing process of the resin can be changed. Therefore, many adaptable epoxy formulations are commercially available. Compared with UP and VE resins, EP matrix systems have better mechanical properties and are more heat resistant. However, they have a significantly higher price. A disadvantage of EP resins is that they absorb moisture and during absorption lose their mechanical properties.

3.3.1.4 Phenol formaldehyde (PF) resins or phenolic resins

Phenolic resins are the oldest resin systems and are produced by a condensation reaction of phenols with formaldehyde, resulting in water as a release product. The water thus formed in the resin evaporates during the curing reaction and sometimes causes considerable processing problems, as cracks and pores can form in the FRP laminate. By applying high pressures during the curing reaction, the defects can be reduced to an acceptable level. Phenolic resins have an excellent fire behaviour. They also have high thermal stability, good chemical resistance and low water absorption. PF resins are among the cheapest of the temperature-resistant resins. These resins contain formaldehyde, which can have a negative impact on human health, but in this case it is trapped within the hardened polymer matrix from which it is not expected to be released (Canada.ca 2019).

A comparison between the most important thermoset resin systems EP and UP is shown in Table 3.3. Table 3.4 shows the mechanical properties of the two resin types.

Table 3.3. Comparison of the properties of the most common matrix systems UP and EP (*Tragkonstruktionen aus Faserverbundkunststoffen im Bauwesen* 2002).

UP - Unsaturated polyester resins	EP - Epoxy resins
Proven, inexpensive and durable casting resins	More expensive than UP, but excellent adhesion and bonding properties
Great possibilities in processing	Relatively long curing time
Large shrinkage coefficient (up to 8%)	Low shrinkage coefficient (3%) and therefore low residual stresses and high dimensional accuracy
Post-curing is important for complete curing	Small tolerances in the mixing ratio of components
Environmental and health aspects due to styrene	

Table 3.4. Mechanical and physical properties of UP and EP.

Resin	Density [$kg \cdot m^{-3}$]	Tensile strength [$N \cdot mm^{-2}$]	MOE [$kN \cdot mm^{-2}$]
UP	1.2	60-70	3.5
EP	1.2	70-90	3.5

When processing thermosets with fibres, resin and hardener are usually poured in a still liquid state into a mould in which the fibres have already been laid. The optimum wetting of the fibres and the absence of bubbles in the resin is achieved by pressure or mechanical rolling (hand lamination). The curing process of the resin must be coordinated with the processing method so that the resin only cures after it has been applied. The curing process of thermosets is a chemical process that can be thermally controlled.

3.3.2 Thermoplastics

The fact that a compromise always has to be found between the required heat resistance and the toughness of thermoset matrices ultimately led to the development of matrices that are both highly plastic and exhibit high heat resistance. The thermoplastic binders polyether ether ketone (PEEK), polyether ether sulfone (PES), etc. have advantages over thermosets due to their different material behaviour:

- high impact strength
- high ultimate strain
- good compression, crushing and buckling behaviour
- good chemical resistance
- low moisture absorption
- short processing cycles, no curing reaction during processing
- weldability
- recyclability of waste
- unlimited storage time at room temperature.

However, there are also some disadvantages of thermoplastic matrices:

- tendency to creep at elevated temperatures
- high temperatures and pressures required during processing
- difficult impregnation of the fibre due to the high viscosity during processing.

Thermoplastics have the largest share of all plastics and are characterized by the fact that they soften reversibly under the influence of temperature and can then be plastically formed. At normal temperature, thermoplastics are brittle or tough-elastic. They are fusible, weldable, swellable and for the most part soluble in organic solvents. The chemical solubility is based on the special (linear) arrangement of the molecular chains (Figure 3.3).

The thermoplastics are bonded to the fibres mainly in the molten state, more rarely also in the dissolved state. Due to the high viscosity, machines operating at high pressures are required in the melting process to ensure good wetting of the barrels.

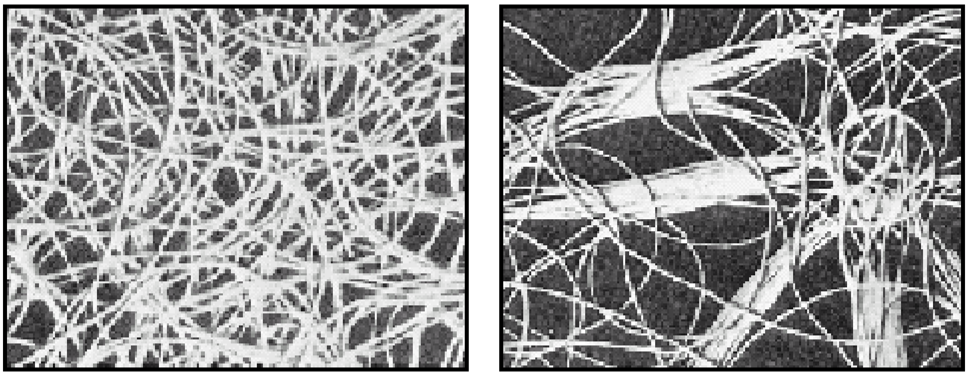


Figure 3.5. Structure of thermoplastics (left: amorphous, right: semi-crystalline) (Michaeli and Wegener 1989).

The most common thermoplastics are:

- Polypropylene (PP)
- Polyethylene (PE)
- Polystyrene (PS)
- Cellulose acetate (CA)
- Polyvinyl chloride (PVC)
- Polymethyl methacrylate (PMMA)
- Polyamide (PA)
- Polycarbonate (PC)
- Fluoroplastics
- Polyacetal or polyoxymethylene (POM)

In modern high performance FRPs, the applicability is limited to a relatively small group of high-temperature thermoplastics due to the high demands on thermal stability. These are polyetherimides (PEI), polysulfones (PSU), polyethersulfones (PES) and polyetheretherketones (PEEK). What all these thermoplastics have in common is that the processing problems resulting from the high melting temperatures and their high viscosities have not yet been completely solved.

3.3.3 Blends between thermosets and thermoplastics

Due to the ever-increasing demands on the thermal and oxidative resistance of resins for special applications, resin systems made of bismaleimides and polymides have been developed. In terms of chemical structure, most polymide resins (PI) consist of blends with duromic and thermoplastic components. Besides their temperature resistance, polymide resins are characterized by good mechanical properties, high resistance to organic solvents and low moisture absorption. The advantages of bismaleimides include their adaptability to a wide range of desired properties, good strength and stiffness even at high temperatures, high resistance to solvents, resistance to ageing and non-toxic combustion.

3.4 Properties of FRPs

The physical and mechanical properties cannot be explained by the properties of the individual components (fibre and matrix) alone. The coordination of the components with each other is decisive, primarily with regard to adhesion (Subsection 3.4.1) and mechanical compatibility between fibre and matrix (Subsection 3.4.2).

3.4.1 Adhesion between fibre and matrix

The adhesion in the boundary layer between fibre and matrix is especially important for the stress transfer from the matrix to the fibre. Without good adhesion of the two components, no reinforcement of the composite can be achieved. In principle, there are the following adhesion mechanisms, which can all occur simultaneously and in different strengths and characteristics in the boundary layer:

- chemically covalent bonds;
- secondary chemical bonds;
- adsorption and wetting;
- interdiffusion;
- electrostatic attraction;
- mechanical adhesion

In order to improve the adhesion between fibre and matrix, fibre treatment is a targeted attempt to install certain adhesion mechanisms or to improve the effect of existing ones. Possible fibre treatments are:

- coating of the fibre with adhesion promoters;
- chemical or physical surface treatment.

3.4.2 Compatibility of fibre and matrix

The chemical and mechanical compatibility of the two components, fibre and matrix, is a fundamental prerequisite for an optimal FRP. The chemical compatibility has already been discussed in the previous Subsection. It occurs manila through covalent and secondary bonds.

Good mechanical compatibility between fibre and matrix is said to exist if the interactions between the components and the technical application of the composite material do not lead to any unfavourable changes in properties, i.e. if the mechanical properties of both components are well matched.

The compatibility of fibres and matrix is shown in Table 3.5.

Table 3.5. Compatibility of fibres and matrix (Michaeli and Wegener 1989).

	Matrix								
Fibre	Thermosets					Thermoplastics			
	UP	EP	PF	VE	PI	PP	PA	PEEK	PES
Aramid	0	+	0	+	+ , 0	-	+		0
Glass C	-	-	-	-	-	-	-	-	-
Glass E	0	+	+	+	-	0	0	0	0
Glass R or S	0	0	0	0	-	-	-	-	-
Carbon HT	+	+	+	0	+	0	0	+	+
Carbon HST		+	+	-	-	-	-	-	-
Carbon HM	0	+ , 0	+	-	-	-	-	-	-

+ compatible combination, adequately studied
0 compatible combination, hardly examined
- incompatible combination

3.4.3 Mechanical properties

A qualitative comparison between the most important properties of FRP laminates is presented in **Fehler! Verweisquelle konnte nicht gefunden werden.**. It can be seen that the CFRP laminates are the most widely used in the construction industry, particularly because of its good resistance, high modulus of elasticity and excellent fatigue behaviour.

Due to the large number of fibres, the scattering of tensile strength in uni-directional direction is low. If one fibre breaks within a laminate, the failure will not propagate as in a solid, but the remaining fibres will remain intact. In addition, the embedding of the fibres in the matrix means that even a torn fibre a few millimetres from the point of separation can take over the full load again. Since the strength and the modulus of elasticity of the fibres in longitudinal direction are many times higher than those of the epoxy resin matrix, the laminate's properties in longitudinal direction are mainly determined by the properties of the fibres. In the transverse direction of the laminate's, however, the mechanical properties are primarily determined by the matrix properties due to the extremely anisotropic behaviour of the fibres, so the strength of the laminate in the transverse direction and the shear strength is only a fraction of the strength in the longitudinal direction. If the strengths in the transverse direction are exceeded, cracks are formed transversely to the fibres, but the tensile strength in the longitudinal direction of the fibres is hardly affected. However, the relatively low inter-laminar shear strength can be critical for the introduction of high forces. In the load introduction area, for example at the end of a lamella, a shear failure could occur due to the relatively low inter-laminar shear strength. However, since the strengths are still significantly greater than those of wood, inter-laminar shear failures will not normally occur there.

Table 3.6. Qualitative comparison between glass, aramid and carbon fibres (Michaeli and Wegener 1989).

Property	FRP laminations made from:		
	Carbon fibres	Aramid fibres	Glas fibres
Tensile strength	very good	very good	very good
Compressive strength	very good	insufficient	good
MOE	very good	good	sufficient
Creep behaviour	very good	good	sufficient
Fatigue behaviour	very good	good	sufficient
Density	good	very good	sufficient
Chem. resistance	very good	good	sufficient
Cost	good	good	very good

In certain static systems, areas that are usually stressed in tension may also be under compressive stress, depending on the load arrangement. Aramid fibre reinforced plastic lamellas would also fail due to the very low compressive strength of this material. Carbon lamellae, on the other hand, are very well suited to this task. In order to achieve a higher load-bearing resistance or a higher flexural stiffness, high degrees of reinforcement must be selected for glass fibre reinforced lamellae because of their lower stiffness compared with CFRP and AFRP.

3.4.4 Creep and relaxation

Creep and relaxation can be neglected for unidirectional CFRP laminates in longitudinal direction. The longitudinal stiffness is dominated by that of the carbon fibres, which hardly creep or relax at all. In the transverse direction, which is usually unimportant for reinforcement purposes, CFRP laminates are as sensitive as the other FRPs, since the behaviour is governed by the epoxy resin matrix.

3.4.5 Fatigue behaviour

There is hardly any other material that shows such excellent fatigue behaviour as carbon fibres. While steel, for example, is sensitive to fretting corrosion, CFRP laminates do not appear to be subject to this damage mechanism. This is important for the bridging of cracks under oscillating loads. When reinforcing with CFRP plates, it is almost unthinkable that these will be the decisive factor in terms of fatigue. In most cases, the fatigue behaviour of the other material (timber, concrete, aluminium or steel) or the adhesive is likely to be critical. AFRPs have a metal-like fatigue behaviour. GFRPs do not have any fatigue strength. The creep strength for a period of 100 years is about 60% of the short-term strength.

3.4.6 Chemical resistance

Based on the available knowledge, it can be assumed that CFRP laminates are resistant to the chemical environments normally occurring in reinforcing tasks in civil engineering over the long term. In case of exceptional requirements, this resistance would have to be checked in the relevant technical literature or discussed with experts. Normally, however, the epoxy resin matrix, which is also known to be very stable, should become critical before the carbon fibres. GFRP are characterised by a high resistance to salts, acids and aromas. Despite the protective effect of the synthetic resin, glass fibres can be attacked by alkalis (e.g. concrete). AFRPs are also resistant to solvents, fuels, lubricants, salt water etc., but can be attacked by some strong acids and alkalis.

3.4.7 Temperature effects

The coefficient of thermal expansion along the fibre is almost zero for CFRP laminates, so that length changes at temperature extremes are very small. Perpendicular to the fibres, however, it is about three times the coefficient of thermal expansion of steel. GFRPs have similarly low coefficients of thermal expansion as wood. Aramid fibres have negative coefficients of thermal expansion in the longitudinal direction of the fibre. The fibre shortens in heat, while the matrix resin expands. Due to these opposing forces, AFRPs exhibit high dimensional stability at elevated temperatures. If the temperature rises above 50 °C, a significant decrease in strength of the adhesive can be expected. The low glass transition point of fibre-reinforced lamellas therefore often leads to the need of fire protection cladding in the form of high-quality fire protection panels.

3.4.8 Electrical conductivity

AFRP and GFRP do not conduct electricity. As the carbon fibres do not act as electrical insulators in the longitudinal direction, but are also not good conductors, there is a risk of damage to the blades if they were struck by lightning. The fibres would be heated up considerably due to their relatively high electrical resistance compared with metallic materials. The temperature could exceed 400 °C and burn the epoxy resin matrix in which the fibres are embedded. The fibres would generally remain intact, but could no longer perform their load-bearing function, since the stresses should be introduced via the damaged epoxy resin matrix. In most applications, there is no danger of lightning strike, as the CFRP laminations are used inside buildings and bridges (boxes) or under bridges. If CFRP laminates are applied to the surface of masts or facades, lightning protection measures must be taken.

4 Bond behaviour between timber and FRP composites

Most applications of FRP composites in timber structures rely on the use of an adhesive to bond these two materials and transfer stresses between them. The bond behaviour is therefore critical to the performance of timber-FRP composites. The bond behaviour depends not only on the type of materials, for which there is a wide variety available (Sections 2 and 3), but also on the specific application (Sections 5 and 6), the application conditions (e.g. regarding curing of the adhesive), and the conditions to which it will be exposed during its service life (e.g. moisture, level of stresses). It is therefore clear that there is no single adhesive, or family of adhesives, that outperforms all others under all circumstances.

The study of the bond behaviour between timber and FRP composites has mostly followed a pragmatic approach, whereas in steel and concrete structures it seems to have followed a more systematic approach. In the pragmatic approach, the strengthened members are tested under conditions similar to those in which they are to be used (e.g. a GLT beam with a FRP composite lamination applied as bending strengthening is tested as such, Figure 4.1 (Blaß and Romani 2001)). In the systematic approach, the bond behaviour is assessed based on extensive smaller scale tests (e.g. pull-out tests, which include single-lap shear tests, Figure 4.2) from which bond-slip models are derived and finally used to estimate the behaviour of the reinforcement in a specific situation. This is mostly related to the complex challenges posed by the anisotropic nature of wood, its noticeable hygroscopicity and moisture-dependent properties, and the high variability exhibited by its physical, chemical, and mechanical properties. Since most structural timber products are produced by gluing together timber boards or wood veneers, fibres can be embedded in these gluelines and "sandwiched" between the boards or veneers during production. In this case, bond performance is usually assessed by simple shear-block tests (Figures 4.4, 4.5, and 4.8). These tests have also been used to evaluate the bond between timber and "thick" FRP composites (Figure 4.6). The bond behaviour between timber and FRP composites has also been assessed by performing pull-out tests on glued FRP composite elements(e.g. on NSMR as in Figures 4.1 and 4.6, or GiRs as in Figure 4.3 (Tlustochowicz et al. 2010)).

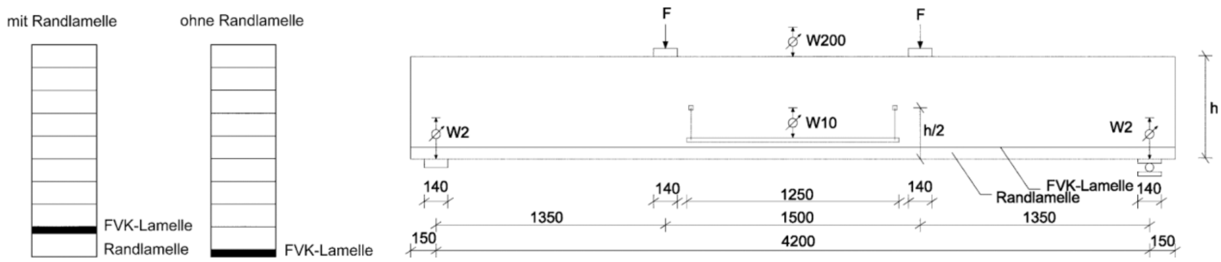


Figure 4.1. Bending test on GLT beam strengthened with a FRP lamination in the tension zone (Blaß and Romani 2001).

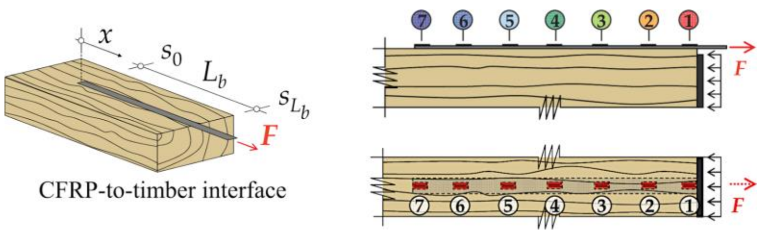


Figure 4.2. Single-lap shear test, with strain gauges 1-7 installed on the CFRP laminate. Figure adapted from Biscaia et al. (2016).

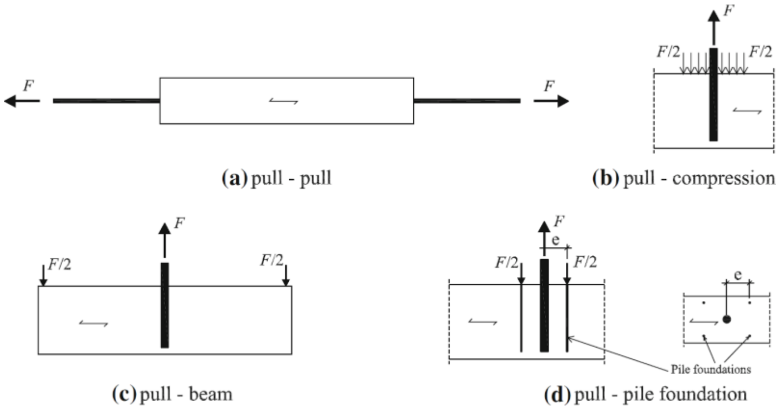


Figure 4.3. Loading configurations when performing pull-out tests of glued-in rods (Tlustochowicz et al. 2010).

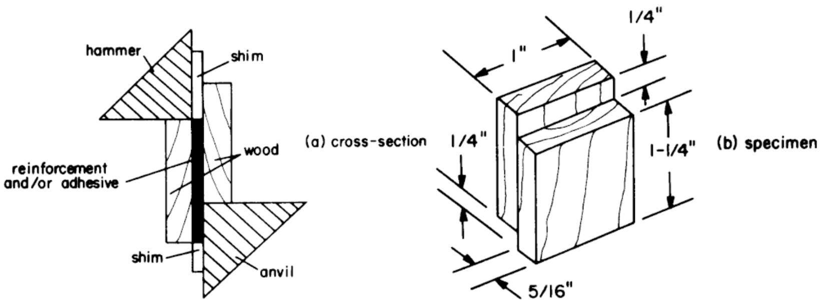


Figure 4.4. Shear-block test specimens (Rowlands et al. 1986).

4.1 Shear-block tests

In the U.S.A, Rowlands et al. (1986) studied the shear strength and the durability of timber-to-timber gluelines reinforced with different types of embedded fibres and adhesives (including epoxy, RF, and PRF) and performed shear-block tests (Figure 4.4) to assess the shear strength of the many fibre-adhesive-wood combinations. Under normal dry conditions, the epoxy, RF, and PRF adhesives performed well, unlike isocyanate and PF adhesives. The epoxies are reported to have deteriorated significantly under a severe moisture cycle, but Rowlands et al. (1986) question "the practicality of the environment used". The authors also note that "the relatively inexpensive" PRF adhesive performed essentially as well as the RF adhesive and point out the difficulty of ensuring adequate wetting of the fibres, particularly when textile multidirectional fibre reinforcement are used.

About a decade later, still in the U.S.A, Gardner et al. (1994) studied the shear behaviour of timber-timber gluelines reinforced with thin FRP composites (Figure 4.6b), instead of only adding fibres to the timber adhesive. The wood species was yellow poplar (hardwood), the tested adhesives included RF, epoxy and isocyanate, and vinylester and polyester pultruded GFRP composites. The specimens were tested in normal dry conditions, in wet saturated conditions, and after an accelerated-aging test comprising a 5-cycle vacuum-pressure-soak-dry procedure. The results showed that all adhesives produced "adequate" strength values under dry conditions. However, only the RF adhesive showed promising results under the wet conditions. Gardner et al. (1994) also report that the specimen with RF adhesive did not exhibit any delamination after the accelerated-aging tests. The authors concluded that FRP composites could be successfully bonded to wood. For interior applications, both RF and epoxy adhesives were assessed as suitable for gluing vinylester FRP to wood, for exterior applications the RF adhesive showed good ability to glue both polyester and vinylester FRP composites to wood.

Still in the U.S.A., Jordan (1998) studied the behaviour of hybrid GLT-timber beams produced using the "wetpreg" process, which is a more controlled version of hand layup. (details of Jordan's (1998) research on beams are presented in Subsection 5.1.1.1). To select the best adhesive/fibre combination, Jordan (1998) performed shear-block tests (Figure 4.6b) to compare the different combinations. Under dry conditions, the shear strength of gluelines reinforced with pre-impregnated stitched fabrics was reported to be equivalent to that of the timber-timber bond, even using conventional structural timber PRF adhesives, However, the shear strength tended to decrease with the number of reinforcement layers. Gluelines reinforced with a unidirectional weave also showed no significant difference to the timber-timber bond, but the shear resistance of the produced FRP (i.e. the "FRP-FRP bond") increased with the clamping pressure, probably because of better impregnation of the fibres. After exposure to moisture cycles to simulate the effects of exterior exposure, the results of the shear tests on gluelines showed that the PRF adhesive performed well with stitched fabrics and unidirectional weave reinforcement. The fibre-adhesive combination finally selected to produce the strengthened GLT beams was with GFRP unidirectional weaves and a structural timber PRF adhesive.

Also in the U.S.A, Davalos et al. (2000a; b) studied the performance and fracture of timber-FRP bonded interfaces with the objective of developing test methods to evaluate the service performance of these bonds. Two types of timber-FRP interfaces were analysed: phenolic GFRP-timber and epoxy GFRP-timber bonds (Figure 4.5). The wood was red maple. The phenolic GFRP composites were glued using RF and PRF adhesives (the RF adhesive was the same that was used to produce the red maple GLT); the epoxy FRP composites were directly glued by the epoxy resin used in the filament winding process (red maple GLT was used as the mandrel to apply the epoxy FRP around the wood by filament winding). The bond performance was evaluated in terms of delamination (after wet-dry cycles) and in terms of shear strength under normal and wet conditions (after a vacuum-pressure-soak cycle). Regarding delamination, the results of the phenolic FRP-timber gluelines were within the limit of 8% established for hardwood timber-timber gluelines and delamination decreased with increasing clamping pressure during curing. The results of the epoxy FRP-timber were well within the delamination limits when a HMR coupling agent was used, but far exceeded the limits when a RF coupling agent was used. Regarding the shear strength evaluated with block-shear tests (Figure 4.5), the low interlaminar shear strength of the phenolic GFRP composite led to very low shear resistances (much lower than the average timber-timber glueline shear strength), dominated by cohesive failures in the matrix. As mentioned, the epoxy FRP-timber interface was produced by the filament-winding process, and two primers (HMR and RF) were applied on the wood surface before wrapping the epoxy FRP around the wood core. The HMR primer led to failures occurring mainly in the timber layer, for both dry and wet conditions. The specimens prepared with the RF primer exhibited lower shear strengths, much higher variability, and much less wood failures. The RF primer had already shown to result in severe delamination problems after the wet-dry cycles. This showed that the block shear tests under wet conditions might not be enough to capture the severe delamination that might occur and that both tests are needed to evaluate the long-term performance of timber-FRP gluelines. The overall results showed that the delamination tests could be used to study the effect of several bonding parameters (e.g. primers to promote bonding, clamping pressure) and then, for the best combination of bonding parameters, the average interface shear strength can be obtained by block-shear tests under dry and wet conditions. Davalos et al. (2000b) also developed a contoured double-cantilever beam (CDCB) specimen to evaluate Mode I fracture of bonded interfaces, and obtained the interface fracture toughness for dry and wet conditions, for the various FRP-timber combinations discussed before.

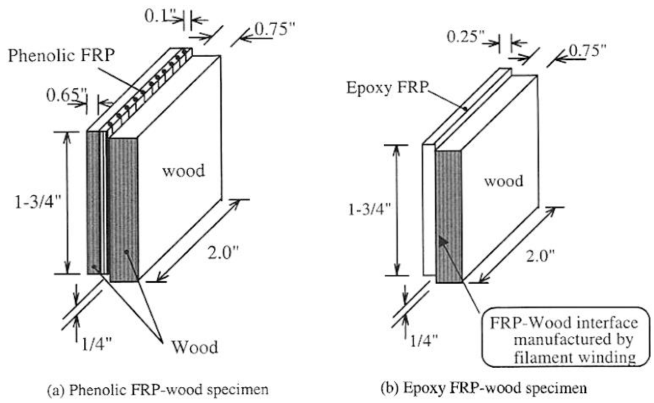


Figure 4.5. Modified ASTM D 905 specimens for block-shear tests (Davalos et al. 2000a).

In the U.S.A, Lopez-Anido et al. (2005) performed delamination and shear-block tests (Figure 4.6) on timber-FRP gluelines, using four FRP composite material systems under test conditions that included the presence of moisture and temperature fluctuations, and the effect of wood preservatives (approach similar to Davalos et al. (2000a; b)). The FRP composites included a broad spectrum of fibre reinforcement (glass and carbon fibres), matrix (vinyl ester, urethane, and epoxy), adhesives (urethane, and epoxy), fabrication processes (resin infusion, pultrusion, and continuous lamination), and gluing process (textile fibre reinforced simultaneously bound and glued by resin infusion, and FRP sheet glued with epoxy or urethane adhesives). The delamination and shear-block tests revealed that the durability of the timber-FRP bonds were seriously affected by chemical treatments of timber and that none of the tested FRP composite/preservative combination tested would be acceptable. However, Lopez-Anido et al. (2005) noted that long-term field monitoring studies were needed to validate the accelerated test protocol and the delamination limits.

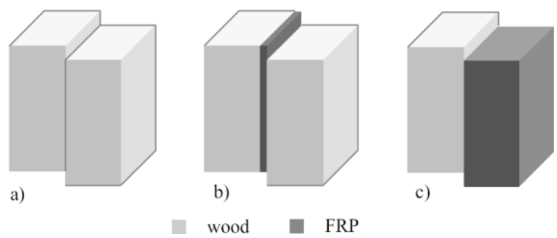


Figure 4.6. Block-shear specimens: a) conventional timber-timber specimen; b) modified timber-FRP specimen for thin reinforcements; c) modified timber-FRP specimen for thick reinforcements (Lopez-Anido et al. 2005).

4.2 Pull-out and single-lap shear tests

In the U.K., Davis (1997) published a review of the performance of adhesive systems for structural timber. He recognised that timber provides a good substrate for adhesive bonding, but requires adequate surface preparation. From his perspective in the late 1990s he predicted that the then already common formaldehyde-based adhesives would continue to be used in the production of manufactured timber products, but that their requirement of pressure to be applied during curing and their limited gap-filling qualities would lead to the use of other adhesives (e.g. epoxy or polyurethane-based) in the field of repair of timber structures. However, at the time research on the strengthening of structural timber members was mostly focused on the use of steel GiRs.

In Austria, Luggin (2000) performed about 400 "pull-out" tests (Figure 4.2) with different adhesives, different bond lengths and width of the CFRP laminations, and different "widths of the longitudinal openings".

In Italy, Lorenzis et al. (2005) performed pull-out tests on CFRP GiRs in prismatic blocks of spruce GLT (pull-pile foundation set-up, Figure 4.3). The tests were performed to assess the influence of the surface preparation of the CFRP rods (sanded or non-sanded); the bonded length (50-300 mm) and of the angle between the bar and the timber fibres. The GLT blocks for the tests with the GiR parallel-to-the-grain had 160×160 mm^2 cross sections with the rod in the centre, and the blocks for the perpendicular-to-the-grain tests had 120×230 mm^2 cross sections. The CFRP rods had a nominal diameter of 12 mm and the embedment lengths ranged from 4 to 24 times the nominal diameter. An epoxy adhesive was used and the glue-line thickness was 2 mm. The tests were performed under displacement control. In the direction parallel-to-the-grain, the test results showed that the GiRs with bond lengths of 300 mm exhibited shear failures parallel to the grain in timber, with a wooden plug 1-6 mm thick around the glueline being pulled out. In the specimens with smaller bonding lengths, splitting failures occurred in the GLT blocks, often without even intercepting the glue line, effectively splitting the block in two parts. This is most likely related to the test set-up and size of the test GLT block, and it might not happen in other types of tests (e.g. pull-pull) or if larger blocks are used. In tests with perpendicular-to-the-grain GiRs, failure was mostly in the timber-adhesive interface, with the CFRP being pulled out with the adhesive layer still attached and some wood splinters. These tests reached higher load-carrying forces than the equivalent tests in the direction parallel to the grain. All test specimens exhibited a linear-elastic behaviour up to the maximum load. The type of surface of the GiR had a negligible influence on the load-carrying capacity, since failure occurred either in timber or in the timber-adhesive interface. The load-carrying capacity increased approximately linearly with the bond length, even though in the specimens loaded in direction parallel to the grain the failures for bond lengths smaller than 300 mm were due to splitting of the GLT block, as already mentioned. The load-slip behaviour of the GiRs loaded in direction parallel to the grain was linear elastic until failure. In direction parallel to grain, the sanded CFRP rods exhibited a higher initial bond-slip stiffness that (Lorenzis et al. 2005) attribute to improved adhesion but also to the higher MOE of these GiRs. The tests with the rod in direction perpendicular to the grain exhibited a softening phase after the maximum load, but this was probably due to the fact that the tests were performed under displacement control. The bond strength was mostly independent of the bond length, even though, as already mentioned, in direction parallel to the grain the failures for bond lengths smaller than 300 mm were due to splitting of the GLT block.

Figure 4.7. Test set-up for investigating the impact of the anchoring length (Johnsson et al. 2007).

In Switzerland, Richter and Steiger (2005) performed tests to evaluate the temperature-dependent creep of epoxy adhesives in timber-FRP glueline. Through tensile tests performed on timber-FRP splices made from spruce (*Picea abies*), a carbon-fibre laminate, and an epoxy adhesive, Richter and Steiger (2005) observed that adhesives with high viscosity performed better than those with low viscosity and that up to temperatures of 50 °C no significant losses in strength and stiffness were observed. However, for temperatures above 50 °C, the various adhesives exhibited very different behaviours, some of them exhibiting shear strengths comparable to that of timber, but most much lower strengths. Richter and Steiger (2005) concluded that adhesives must be tested before being considered suitable for structural use.

In Sweden, Johnsson et al. (2007) performed pull-out tests to evaluate the effective anchoring length of NSMR CFRP bars on GLT. The effective anchoring length is the bond length above which the load transfer no longer increases and is an important parameter for load-carrying applications. The CFRP rods had rectangular 10×10 mm^2 cross sections and were glued with an epoxy resin in a 12×14 mm^2 groove (2-4 mm glueline thickness) made in the timber blocks, made of spruce GLT of strength class GL 32 and with dimensions 400×70×70 mm^3 (Figure 4.6). The tested bond lengths were 100, 150, 200 and 250 mm. The tests were conducted under displacement control. The results showed that the most common failure mode was a plug shear failure in timber, except for 100 mm bond lengths, in which failure occurred in the adhesive. The results showed an effective anchoring length of 150 mm. This length was later also verified by strain gauges installed on a NSMR CFRP rod in a GLT beam tested in four-point bending. Details of Johnsson's et al. (2007) research on beams are presented in Subsection 5.1.1.1.

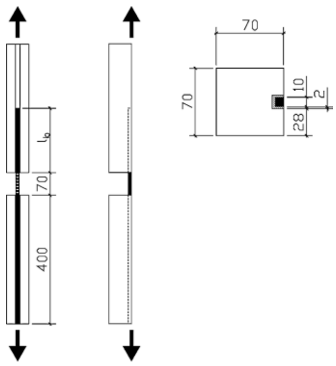


Figure 4.8. Test set-up for inestigating the impact of the anchoring length (Johnsson et al. 2007).

In Ireland, Raftery et al. (2008) performed shear block tests on timber-timber bond interfaces, using a range of conventional structural timber adhesives (PRF, MUF, polyurethane (PU), isocyanate (EPI), and a polyvinyl acetate) to evaluate the influence of several parameters (proportions of juvenile wood, moisture cycling, and grain orientation) on the bond behaviour. The timber used in this study was Irish-grown Sitka spruce. The results showed that the integrity of the bond was highly dependent on the adhesive. The presence of juvenile wood did not significantly influence the results and no significant difference in performance was observed when bonding to tangential grained or radial grained wood. In a following study, Raftery et al. (2009a; b) performed similar tests on the same type of timber, but inserting a FRP composite in the glueline (Figure 4.9). Raftery et al. (2009a) examined the hygrothermal compliance of five commercial structural timber adhesives when bonding commercially available FRP composites to wood. The tested FRP composites were polyurethane Fulcrum and a vinylester GFRP composite with uni-directionally aligned fibres. Some specimens were exposed to five vacuum-pressure-soak-drying cycles before being tested in shear. The test results showed that many specimens failed during the moisture-cycling phase. The combinations that exhibited the poorest performance, i.e. that were most susceptible to failure from the hygrothermal stresses introduced during the moisture cycling, were MUF-bonded GFRP when no adhesive promoter was applied and PRF-bonded Fulcrum also when no adhesive promoter was applied. PRF-bonded GFRP with adhesive promoter behaved quite well. Regarding shear strength, considerable variability in performance was found depending on the FRP-adhesive combination. Some combinations exhibited almost no strength reduction, compared to the timber-timber bond with the same adhesive, even after the moisture cycles, whereas for other combinations the presence of the FRP composite severely reduced the shear strength. The performance of a common PRF adhesive was reported to be very "encouraging" and the GFRP composite also behaved well with a number of adhesive types. An important conclusion was that the integrity of the bond depended not only on the type of adhesive, but also on the FRP type under consideration. Raftery et al. (2009b) performed similar tests, but this time using an epoxy adhesive. In this case, the bond lines were much thinner (approximately 0.5 mm), but the results showed that some combinations of epoxy and FRP were able to reach the same shear strength as the timber-timber bonds and show no significant strength reduction even after the moisture cycles.

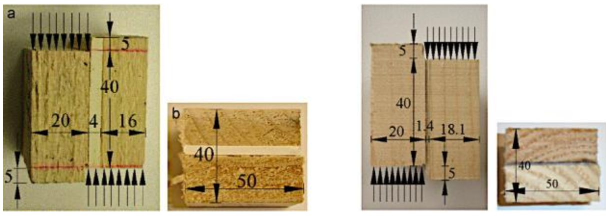


Figure 4.9. Test specimens subjected to shear testing by Raftery et al. (2009a; b), Figure adapted from Raftery et al. (2009a; b).

In Italy, Benedetti and Colla (2010) examined the bond behaviour of EBR CFRP laminations and NSMR CFRP rods on solid timber elements cut from 200-year old beams. The timber elements were divided into two quality classes based only on visual inspection. The bond behaviour of the EBR CFRP laminations (50 mm wide) was evaluated through single-lap shear tests and the behaviour of the NSMR pultruded CFRP rods (12 mm diameter) through pull-out tests (Figure 4.10, left). "Double-cut" shear tests were also performed (Figure 4.10, right), but the results are not reported. The FRP composites were glued to the timber block using an epoxy adhesive. The single-lap shear and pull-out tests were performed under displacement control and the "double-cut" shear tests under force control. The tests results show that the EBR CFRP laminates exhibited a linear-elastic brittle behaviour, with about 60% of timber failures (longitudinal shear in timber, with a timber wedge or a very thin timber layer being taken out still attached to the adhesive layer), 15% cohesive failures in the adhesive, and 25% mixed adhesive-timber failures. The NSMR CFRP rods reached higher load-carrying capacities, but also failed mostly in timber, with 85% of timber failures (longitudinal shear in timber, with a timber block or a very thin timber layer being taken out still attached to the adhesive layer) and 15% cohesive failures in the adhesive. Regarding the effective anchoring length, the results are not conclusive, but seem to indicate values about 200-300 mm (less for the EBR CFRP lamination than for the NSMR CFRP rod). No influence of timber quality was observed on load-carrying capacity of the NSMR CFRP rods, but it was clear in the EBR CFRP laminations.

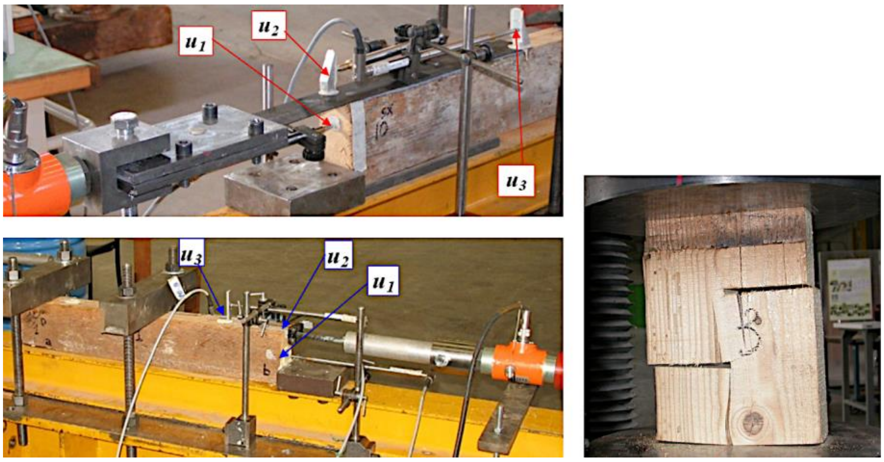


Figure 4.10. Test set-ups applied by Benedetti and Colla (2010): single-lap shear test on EBR CFRP laminate (top left); pull-out test on NSMR CFRP rods (bottom left); "double cut shear test" (right). Figure adapted from Benedetti and Colla (2010).

In Portugal, Sena-Cruz et al. (2012) studied the bond behaviour of NSMR GFRP rods using different pull-out tests: "direct pull-out" and "beam pull-out" (Figure 4.11). The timber blocks were made from spruce GLT, strength class GL 24h. Two types of GFRP rods were tested, both with a diameter of 10 mm and produced in the same way, the only difference being that one had a rougher external surface. The GFRP rods were glued with an epoxy adhesive. Both tests were performed under displacement control, using the same displacement transducer that measured the slip at the loaded end. The test results showed that the direct pull out tests exhibited a linear force-slip behaviour, whereas the equivalent beam pull-out tests exhibited a much more non-linear behaviour. Nevertheless, both tests led to very high load-carrying capacities for the same configurations, maybe slightly higher for the beam pull-out tests. The highest anchorage capacity was reached by the configuration with the GFRP rod with the roughened surface and placed deeper into the groove. Many failures occurred in the timber-adhesive and FRP adhesive interface, but more timber shear failures were observed when the rod was placed deeper in the groove. No clear effective bond length can be determined from the results, but a value between 120 and 180 mm seems to be plausible.

Also in Portugal, Juvandes and Barbosa (2012) performed pull-out tests on EBR CFRP laminates and NSMR CFRP laminates. The timber GLT block was produced with timber boards of strength class C 30 and the CFRP reinforcement was made from uni-directional carbon fibres cured in situ (the amount of reinforcement fibres was the same for all reinforcement schemes) (Figure 4.10). The anchorage lengths were 20, 40 and 60 mm and the tests were performed under displacement control. The results showed that the main failure mode in the in EBR tests was longitudinal shear failure in timber The reported effective anchorage length is only 40 mm, which is significantly smaller than the values found in previous studies. The NSMRs tests showed a much greater proportion of mixed failures, including many cohesive and FRP failures, but the effective anchorage lengths are reported to be around 60 mm and for the "vertical" NSMR and between 40 and 60 mm for the "horizontal" NSMR (Figure 4.10), which is quite small.

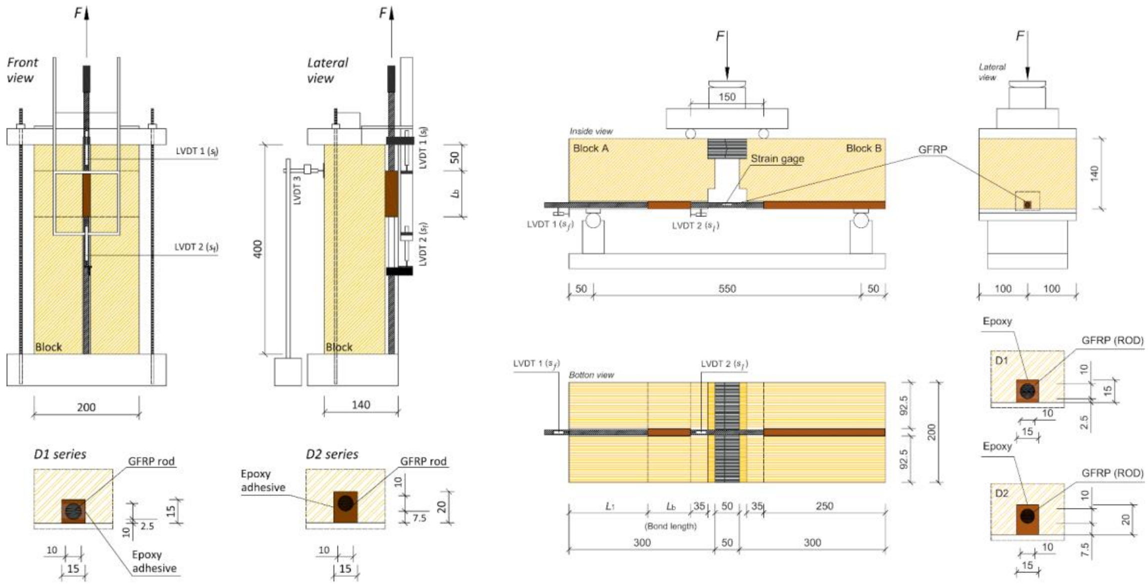


Figure 4.11. Reinfocement schemes (NSMR) and test set-ups studied by Sena-Cruz et al. (2012): direct pull-out tests (left); beam pull-out tests. Figure adapted from Sena-Cruz et al. (2012).

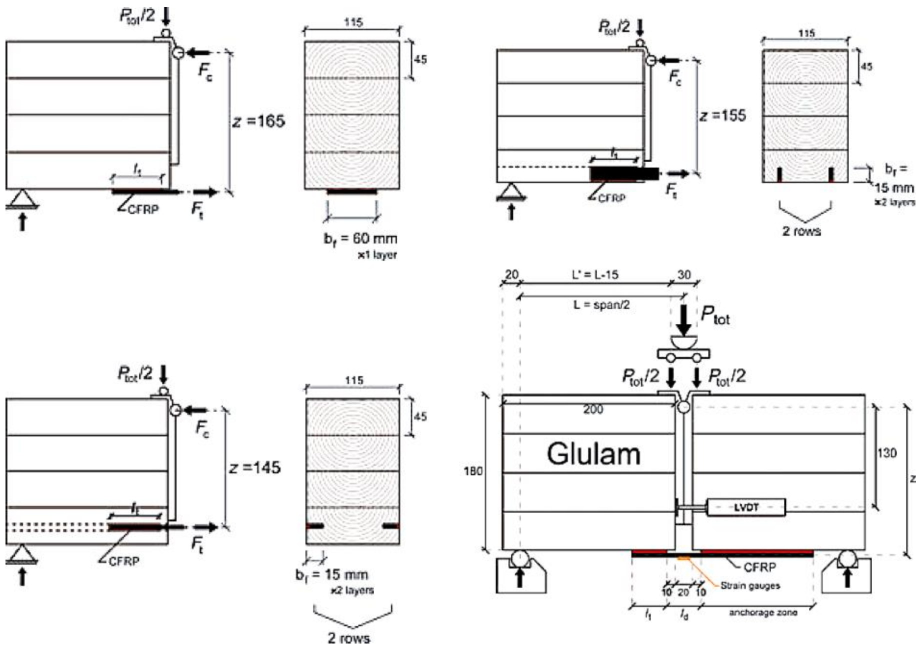


Figure 4.12. Reinfocement schemes: EBR (top left) studied by Juvandes and Barbosa (2012); "vertical" NSMR (top right); "horizontal" NSMRT (bottom left); and test set-up (bottom right). Figure adapted from Juvandes and Barbosa (2012).

In China, (Wan et al. 2011, 2014) and Wan (2014) studied the influence of adhesive type, CFRP plate type, and timber species on the bond behaviour of timber-FRP bonds, based on single-lap shear tests (Figure 4.13). The tested configurations included softwood and hardwood timber (pine (*pinus*) and camphor (*cinnamomum camphora*), respectively), pultruded and wet lay-up CFRP plates, and six different epoxy adhesives. The tests were performed under displacement control. The results showed a great variation between the epoxy adhesive products, reinforcing the idea that the type of adhesive is not enough to characterise its performance and that different formulations can have extremely different behaviours. The most common failure modes were longitudinal shear failures in timber and failures in the timber-adhesive interface, in which a thin layer of wood comes out attached to the adhesive. The combination that led to higher pull-out forces and more longitudinal shear failures was the pultruded plate in hardwood. In softwood, the CFRP plate also exhibited higher pull-out resistance than the wet lay-up CFRP. Interestingly, one of the adhesives led simultaneously to the highest (in CFRP plate and hardwood) and one of the lowest (in wet lay-up and softwood) average pull-out forces of all tests. The CFRP plates were less efficient than the wet lay-up CFRP, i.e. their tensile strength was much higher than their level of tensile stresses when pull out occurred. All joints exhibited a linear-elastic brittle force-slip behaviour.

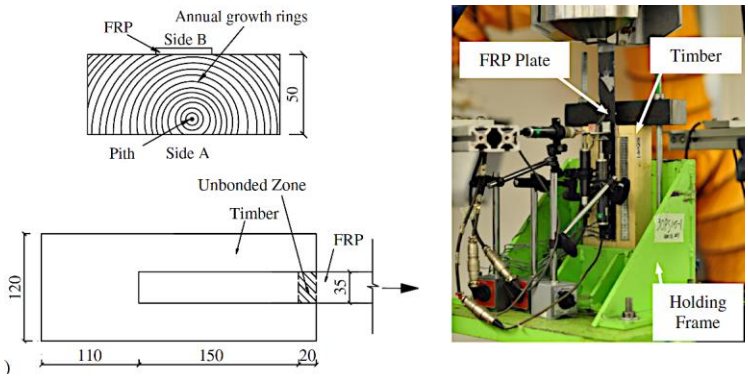


Figure 4.13. Single-lap shear tests performed by Wan et al. (2014). Figure adapted from Wan et al. (2014).

In Italy, Corradi et al. (2015) evaluated the bond behaviour of NSM CFRP bars in timber under direct pull-out. The solid timber blocks were made of fir (*Abies alba*) and chestnut (*Castanea sativa*), of strength classes C 24 and D 24, and cross sections 200×200 mm^2 and 220×220 mm^2 , respectively. The longitudinal, parallel-to-the-grain notches into which the CFRP were installed were 14×15 mm^2. The CFRP rods had a diameter of 7.5 mm and a sandblasted helically-wound deformed surface. The rods were glued with an epoxy adhesive. Bonded lengths of 150, 200, 250 and 300 mm were tested. The test set-up comprised an actuator positioned between the two timber blocks (Figure 4.14), controlled using a hand pump with which the oil pressure was increased at an approximately constant rate (therefore the tests were performed under force control). It should be noted that, by testing two NSM rods simultaneously, this test method is bound to give lower load-carrying capacities than if single rods were tested, since the weaker of the two NSM rods will govern the reached maximum force. The results of the tests on chestnut timber specimens showed that for bonded lengths of 150 and 200 mm the most common failure mode was in the CFRP-adhesive interface, and for 250 and 300 mm bonded lengths the most common failure mode was tensile failure of the rod. The tests on fir timber specimens showed more failures in the FRP-adhesive interface for bonded lengths of 150 m, more longitudinal shear failures in timber for bonded lengths between 200 and 250 mm, and more rod tensile failures for bonded lengths of 300 mm. The effective anchorage length would seem to be around

250 mm, but since the threshold was governed by the tensile failures in the rod, the definition of effective anchorage length is not applicable.

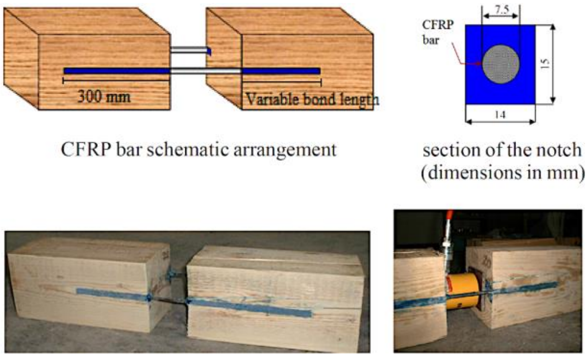


Figure 4.14. Pull-out test specimens with two CFRP rods performed by Corradi et al. (2015). Figure adapted from Corradi et al. (2015).

In Portugal, Biscaia et al. (2016) compared the bonding between CFRP laminates and different structural materials, namely timber, steel, and concrete (Figure 4.15). The tests were performed using same CFRP laminates and adhesive agent. The comparison was based on single-lap shear tests. The CFRP composite had a 1.4×10 mm^2 cross section. The timber specimens were made from Redwood (*Pinus Sylvestris* L.), salvaged from a 19th century building and the properties were assessed based on small-scale tests, so the obtained mechanical properties are not representative of the material at a structural scale. It is not reported if the tests were conducted under displacement or force control. The test results show an effective anchorage length of 80 mm for timber, compared to 50 mm for steel and 160 mm for concrete, and a low efficiency of this EBR technique because only 50% of the strength of the CFRP laminate was used. Regarding failure modes, only the concrete specimens exhibited cohesive failures in the substrate (i.e. in concrete). In some of the timber specimens partial cohesive failures were observed, but most failures were in the CFRP-adhesive interface. The CFRP-to-timber interfaces showed the highest pull-out strength. Biscaia et al. (2016) also report that the local nonlinear bond-slip curve of CFRP-to-timber or steel interfaces can be approximated by trilinear and bilinear bond-slip relations, respectively, and CFRP-to-concrete bond-slip by exponential curves (Figure 4.15). It is also reported that the CFRP-to-timber interfaces exhibited the highest fracture energy. Using the same timber, CFRP laminates, and adhesive, Biscaia et al. (2017) performed single-lap shear tests on EBR CFRP laminates and NSMR CFRP laminates. Some of the EBR specimens had an additional mechanical anchor at the unloaded CFRP end. Almost no experimental details are provided, namely regarding the NSMR and the mechanical anchor for the EBR, but the results showed that the NSMR exhibited a better performance compared to the EBR. The NSMR led to a higher pull-out capacity and a shorter effective bond length (about 150 mm) than the EBR (about 210 mm).

Still in Portugal, Biscaia and Diogo (2020) evaluated different anchorage solutions for laminated CFRP adhesively bonded to timber (Figure 4.16). The comparative study was based on single-lap shear tests. The timber element was made from, judging by the provenance and the images, maritime pine (*Pinus pinaster* Ait.). The cross section of the timber blocks was 100×70 mm^2. Small-scale tests were performed on samples taken from the timber elements, but the extremely high reported strength values results are representative of the values expected from structural-sized members. The CFRP laminate had a cross section of 10 ×1.4 mm^2. The CFRP laminate was glued with an epoxy adhesive and a bonded length of 250 mm was used in all the specimens, which was above the estimated effective bond length. The anchorage systems were installed after this bonded length. The system that reached the higher load-carrying capacity was the one with "two small superposed metallic L-shapes" (top right corner of Figure 4.16). It exhibited a 150% higher load-carrying capacity than the reference EBR specimens. It is not fully clear how this anchorage works, but it seems that metallic angles are glued to the CFRP laminate, epoxied into the notches in the timber element, and also glued to each other. This would indeed create a very stiff foundation for the anchorage, but also make its installation more complicated. The second highest pull-out capacity is reported to have been reached by the NSMR CFRP lamination, without any additional anchorage device. No geometric details are provided about this reinforcement scheme, but it could be that the CFRP lamination was simply vertically inserted and glued in a longitudinal slot. In any case, this seems to confirm the potential of the NSMR. The third highest load-carrying capacity was reached by embedding the free end of the CFRP lamination in the timber member (bottom right corner of Figure 4.16).The list of the remaining anchorage systems, ordered by decreasing pull-put capacity, is: "embedded rectangular hollowed section"; "steel plate"; "3 CFRP spike anchors"; and "2 CFRP spike anchors". With the exception of the CFRP spike anchors, all other anchorage systems increased the pull-out load-carrying capacity when compared to reference EBR specimens.

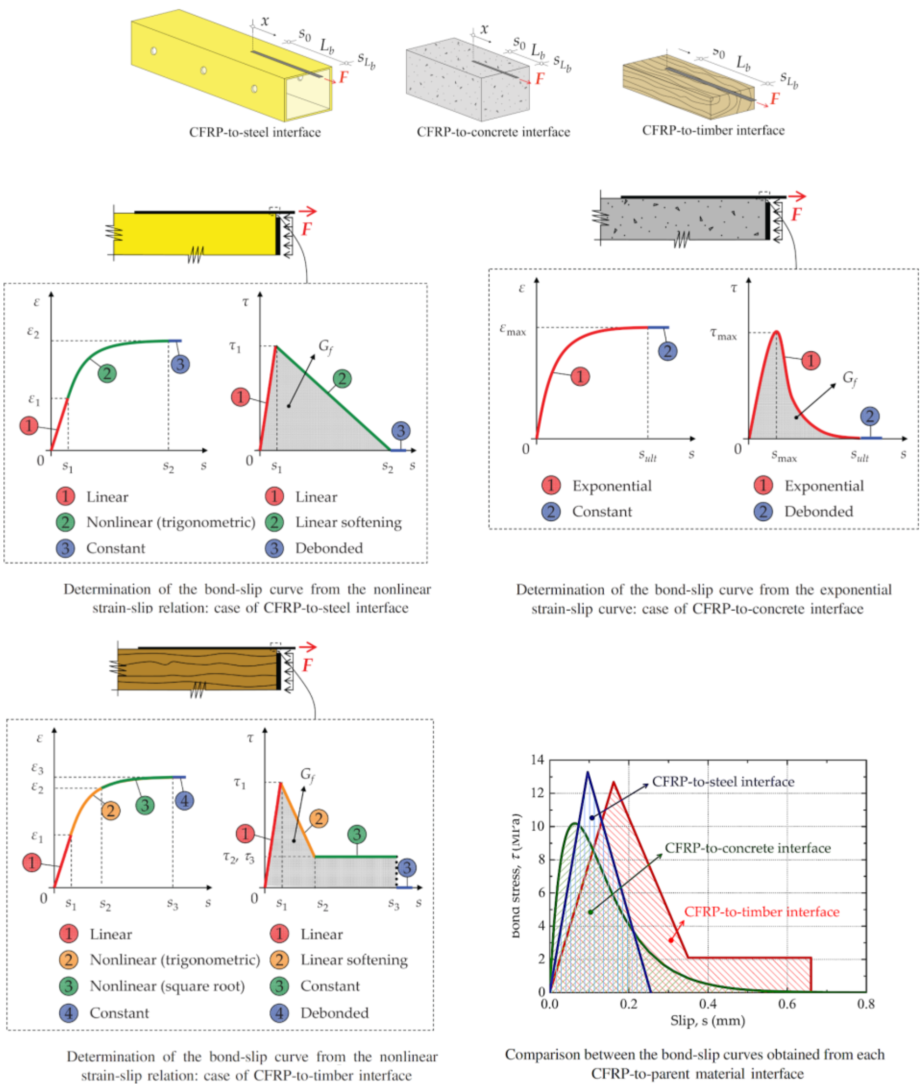


Figure 4.15. Single-lap shear tests and obtained bond-slip curves for steel, concrete and timber reported by Biscaia et al. (2016). Figure adapted from Biscaia et al. (2016).

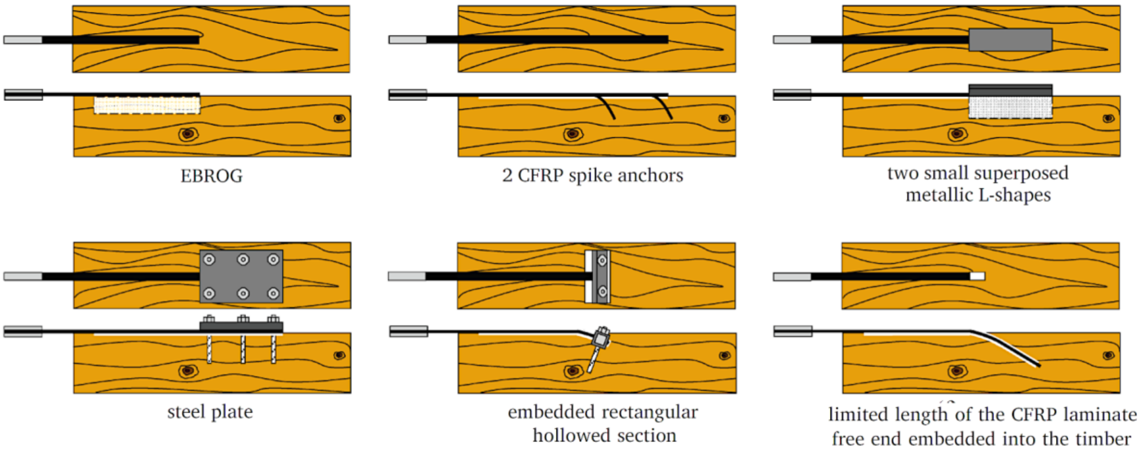


Figure 4.16. Anchorages analysed by Biscaia and Diogo (2020).

Subhani et al. (2017b) studied the effect of grain orientation on the CFRP-to-LVL bond. The LVL blocks were made from type e-beam+, produced by Hyne Timber, which does not seem to have no cross layers. The CFRP laminates had a cross section of 35×1.4 mm^2 and were glued with an epoxy adhesive. The study was based on pull-out tests on the EBR CFRP laminates, performed under displacement control. When the reinforcement was applied on the lamination face of the LVL (as if it was applied in the tension face of a LVL beam), the most common failure modes were in the CFRP-adhesive and timber-adhesive interfaces, independently of the grain orientation. When the reinforcement was applied on the outer veneer and in the direction parallel to the grain, the most common failure mode was cohesive failure of the adhesive. Applying the reinforcement on the outer veneer and in the direction perpendicular to grain was not tested, but it would anyway not be advisable due to the low resistance of the wood veneers to rolling shear. The load-slip curves were always linear elastic and brittle.

Figure 4.17. Test specimens, test set-up, and failed test specimens. Figure adapted from Subhani et al. (2017b).

In Australia, Vahedian et al. (2017) performed single-lap shear tests on timber-CFRP bonded interfaces, to evaluate the effective bond length. The tests were made on LVL and hardwood (no details about wood species are available) using uni-directional wet-lay up CFRP bonded with epoxy adhesive to the timber surface. CFRP reinforcements with different widths were tests, and bond lengths between 50 and 250 mm.

The results showed that most specimens exhibited timber failures (either longitudinal shear or a thin layer of timber came out attached to the adhesive). In LVL, the effective bond length was found to be between 150 and 200 mm, whereas for hardwood it seemed to be closer to 100 mm.

5 Hybrid timber-FRP structural elements

This Section deals with hybrid timber-FRP structural elements. In many of these elements, the FRP composites, with or without pre-stressing, are combined with the timber elements during their production, with the main objective of producing members with improved mechanical properties. Research more related to on-site reinforcement of timber members is presented in Section 6. Since there is a clear overlap between the two, because some strengthening techniques may be applied in a production facility or on site, only the studies conducted on existing structures or in elements taken from existing structures (damaged or not) are reviewed in Section 6 *Reinforcement of structural timber elements using FRP composites*.

Even though most structural timber products are produced by gluing together timber boards or wooden veneers (Subsection 2.1) and FRP composites by gluing together inorganic fibres (Section 3), only a few applications are based on embedding the fibres in the adhesive used to produce the structural timber members. It has been shown that this would work (Subsection 5.1.1.1) in terms of mechanical performance, however, it introduces additional complexities in the production process, namely regarding handling of the fibres. An additional disadvantage is that the resulting FRP is of lower quality than if it would have been produced separately, using a more appropriate polymeric matrix, instead of the timber adhesive, and ensuring a better alignment of the fibres and immersion of the fibres in the matrix. An advantage of this approach is the lower cost of structural timber adhesives (e.g. PRF) compared to the cost of the adhesives used to glue FRP composites to wood (e.g. epoxy) and their ability to withstand harsh service conditions, including high moisture and elevated temperatures, in which structural epoxy adhesives might not behave so well.

The vast majority of applications are based on gluing commercially available FRP composites to finished structural timber elements, to the tension side of timber beams, or during their production. If the FRP composites are introduced during production, structural timber adhesives may be used if adequate clamping pressure can be applied during curing of the adhesive, as in the case of FRP laminations inserted between timber laminations in GLT beams. Otherwise, if no clamping pressure can be applied on the FRP composite, as in the case of glued-in rods, other, usually more expensive, structural adhesives must be used. For these situations, epoxy-based adhesives are often used, namely due to their gap-filling properties, high strength, and good bonding properties to many materials.

5.1 Structural members

The development of hybrid timber-FRP elements started in the 1960s, but it was in the 1990s and early 2000s that most of the studies on the strengthening of timber using FRP composites took place and that the significant technological obstacles were overcome, namely though the development of appropriate adhesives. Since then, most studies are focused on optimising hybrid elements for specific applications and there were no significant breakthroughs.

An important aspect that must be kept in mind when analysing experimental results is related to how the tests were conducted, namely if under displacement or force control. When testing reinforced specimens, the way in which failure is induced (i.e. under displacement control or under force control) has a significant influence the observed behaviour and perceived performance of the reinforcement. Nevertheless, some authors fail to report this in the test procedure. Under displacement control, an increasing displacement is applied to the test specimen and the force required to apply this displacement is measured. Under load control, an increasing load is applied to the specimen and the corresponding displacement is measured.

Reinforced specimens are prone to consecutive failures, during which the stresses are transferred between different components of the system (e.g. from the timber member to the reinforcement after a crack appears). Therefore, performing tests under displacement control usually induces slowly progressing failures that allow for stress redistribution and the corresponding force-displacement curve is jagged, exhibiting abrupt load drops followed by increased in the applied force. This gives a false impression of deformation capacity or ductility. In reality, most loading scenarios are not displacement-controlled, but force-controlled, i.e. it's the load applied to the element that increases gradually and not the displacement. For reinforced specimens, force-controlled tests would very likely not allow for such a stress redistribution between the various elements and the final failure would occur at a much smaller displacement and maybe at a load level similar to that of the unreinforced specimen (due to the impact of the dynamic effects of the first failure on the reinforcement).

5.1.1 Beams or members in bending

Beams are likely the most common structural members in which structural timber products are used. Other timber-based members that work mostly in bending are composite timber-concrete floors and, more recently, CLT floors or CLT rib panels (CLT slab screwed-glued to GLT beam). Most strengthening studies were conducted on solid timber and GLT beams and the main objective was to prevent brittle timber failures in the tension side, to allow for ductile timber failure in the compression side.

5.1.1.1 Bending – passive strengthening

The first reported studies on the strengthening of timber beams using FRP composites were conducted by Wangaard (1964, 1965), Biblis (1965), and Theakston (1965). These works used fibreglass in different formats, namely rovings, woven rovings, cloths, and unidirectional nonwoven roving mats. Theakston (1965) reports problems with adhesives, specifically swelling issues with water-based adhesives and brittleness of epoxy adhesives. At the same time, studies on the production of timber beams with bonded steel elements (Granholm 1954; Sliker 1962; Lantos 1970), including pre-stressed steel sheets (Peterson 1965), do not report problems with adhesives. Lantos (1970) states that the most efficient and convenient way to bond steel to timber was to use conventional phenol resorcinol formaldehyde adhesive on steel dipped into a latex based primer before assembly. Despite the fact that Lantos (1970) conducted his tests on hybrid timber-steel beams, some of his conclusions are valid for strengthening with FRP composites, namely that the variability of the mechanical properties of the strengthened beams appeared to be substantially reduced and that it should be possible to reach the performance of a beam made from high-grade timber using lower-grade timber combined with adequate strengthening.

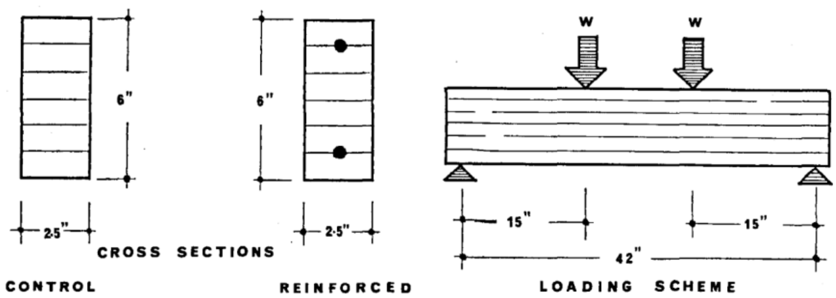


Figure 5.1. Details of loading scheme and cross sections of experimental beams (Lantos 1970).

In the mid 1980s, Bulleit (1984) published a state-of-art of the reinforcement of wood-based materials with steel or FRPs (which at the time meant only fibre-glass composites). The main conclusion was that, even though the reviewed methods were mostly successful in improving the mechanical performance of strengthened members, the possibility of using reinforced wood-based materials on a commercial basis seemed unlikely for materials such as reinforced laminated timber since it was not a cost-effective product. Shortly after, Rowlands et al. (1986) conducted an extensive experimental feasibility study on the production of internally reinforced laminated timber, testing various combinations of fibres (glass, graphite, and Kevlar fibres) and adhesives (epoxy, resorcinol formaldehyde, phenol resorcinol formaldehyde, and isocyanate). They concluded that glass-fibre reinforcement was particularly suitable and technically and economically superior for the studies applications. Epoxy, resorcinol formaldehyde, and phenol resorcinol formaldehyde adhesives proved to be adequate, but the epoxy adhesives deteriorated significantly under severe moisture cycles. The isocyanates and the phenol-formaldehyde adhesives proved not to be suitable.

By the early 1990s, research on the strengthening of timber members with FRP composites was being conducted in many countries. It was though that, like the development of reinforced and post-tensioned concrete, this research could unlock the full potential of timber as a structural material. In France, Moulin et al. (1990) tested small hybrid timber-FRP beams strengthened with unidirectional fibre-glass weave reinforced polymers using a phenolic adhesive. They concluded that the introduction of the FRP reinforcement increased stiffness and ductility, but not the load-carrying capacity. Like Lantos (1970), they saw some potential on combining FRP with low-grade timber. In The Netherlands, Van de Kuilen (1991) also conducted a research program on the strengthening of laminated timber beams using glass-fibre reinforced profiles (with approximately 60% glass fibres) and reports that the stiffness of the beams was raised by 17% with 4% reinforcement (i.e. 4% of the initial beam height) and by 55% with 16% reinforcement.

In the U.S.A. research also continued through the 1990s, mostly through the work of Plevris and Triantafillou (1992) at MIT, Sonti et al. (1995) and Hernandez et al. (1997) at the Forest Products Laboratory, and Dagher et al. (1996, 1998a; b) and Jordan (1998) at the University of Maine. These works were part of a wider effort to evaluate the potential for commercial production of hybrid timber-FRP beams by GLT producers, using adhesives compatible with existing equipment. Plevris and Triantafillou (1992) were some of the first to study the use of CFRP composites to strengthen timber beams. They performed three-point bending and eccentric compression tests on small hybrid timber-FRP beams, with CFRP sheets externally bonded on the tension zone using an epoxy resin. These authors report that ductile compression failures in timber, followed by rupture of the composite sheet and tension failure in timber, were observed for area fractions of fibre composite, i.e. ratios between the area of FRP and area of timber, between 0.33 and 4.03%. As expected, the bending load carrying capacity and stiffness increased with the area fraction of fibre composite. Sonti et al. (1995) and Hernandez et al. (1997) studied reinforcing GLT beams with pultruded GFRP laminates (E-glass rovings embedded in a vinyl-ester matrix) in tension and both in tension and compression. A resorcinol formaldehyde adhesive was used to glue the GFRP laminates to the timber surfaces. The results showed that the commercial production of glulam-GFRP beams was feasible and increases of 18% in stiffness and 26% in load-carrying capacity were reported, for a reinforcement ratio of 3% of GFRP by volume. The improved performance of top and bottom reinforcement was deemed not to be significant enough to offset the added material and handling costs of adding two layers of GFRP. Delamination of the reinforcement was also observed, which showed the bonding between timber and GFRP interfaces has to be improved. Dagher et al. (1996, 1998a; b) focused on using FRPs to improve the behaviour of GLT beams made with low-grade timber. Unreinforced GLT beams and GLT beams reinforced on the tension side with two different FRPs were tested in four-point bending. The GLT beams were of three different wood qualities, so that the relative benefit of the reinforcement could be assessed. The results show that with addition of 1-3% FRP reinforcement the load-carrying capacity increased up to 56% and the stiffness up to 37%. Due to the more efficient utilisation of the compressive strength of timber, ductile failure modes were observed. The largest increases in load-carrying capacity were obtained with the low-grade GLT. However, the authors also note that a major concern was still the long-term behaviour of the timber-FRP interface and the creep behaviour, namely regarding the influence of moisture, temperature and fatigue. Like others before, Dagher et al. (1996) also notes that the commercial success of hybrid GLT-timber beams "will depend upon the future savings of removing wood laminations being greater than the future expense of adding FRP reinforcement".

Jordan (1998) studied the behaviour of hybrid GLT-timber beams produced using the "wetpreg" process, which is a more controlled version of hand layup (the fabric is run through a resin bath, between rollers, which impregnate the fabric and, finally, the wet fabric is "layed-up" and pressure applied). The GLT timber beams were made from Eastern Hemlock (*Tsuga Canadensis*) wood (visual grade No. 2 and better) and had a length of 3.66 m and a cross section of 18×87 cm^2. The reinforcement comprised GFRP uni-directional weaves applied with a PRF matrix/adhesive adequate for GLT production to the tension side of the GLT beams (Figure 5.4), using the "wetpreg" process. Reinforcement ratios of 2, 3, and 4% were tested. The beams were tested in four-point bending, with a span of 3.35 m, under force control. The results showed that the mean load-carrying capacity of the GLT beams increased by 54% with a 2% reinforcement ratio, 71% with a 3% reinforcement ratio, and 88% with a 4% reinforcement ratio. The coefficient of variation of the tests on the hybrid beams (3-9%) was significantly smaller than that of the GLT beams (20%). The bending stiffness also increased with increasing reinforcement ratio (7-23%), but less than the load-carrying capacity.

The research on hybrid timber-FRP beams in the U.S.A. seems to have mostly stopped by the early 2000s. Stevens and Criner (2000) published an economic analysis and concluded that the ability to produce hybrid GLT-FRP beams with production cost advantages over normal GLT beams was limited to relatively high beams with high bending strength, for which steel and concrete might offer better solutions.

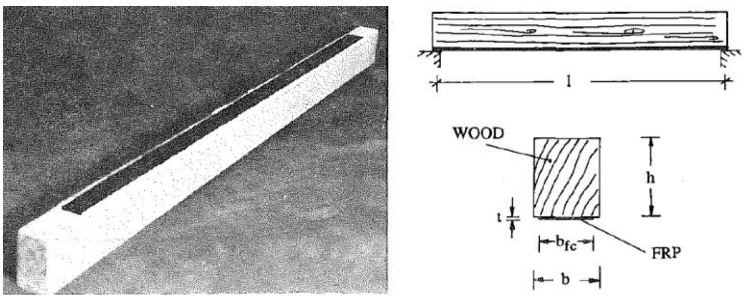


Figure 5.2. FRP-reinforced timber beam (Plevris and Triantafillou 1992).

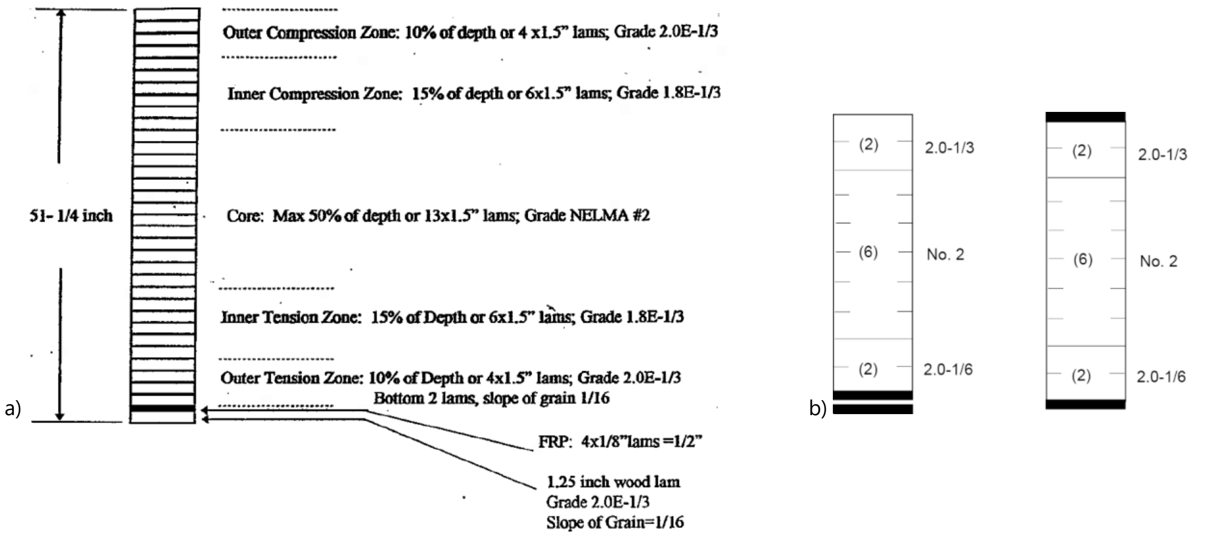


Figure 5.3. Lay-up of reinforced beams: a) GLT beam from Medway Bridge (Dagher et al. 1998c); b) beams with tension and tension and compression reinforcement (Hernandez et al. 1997).

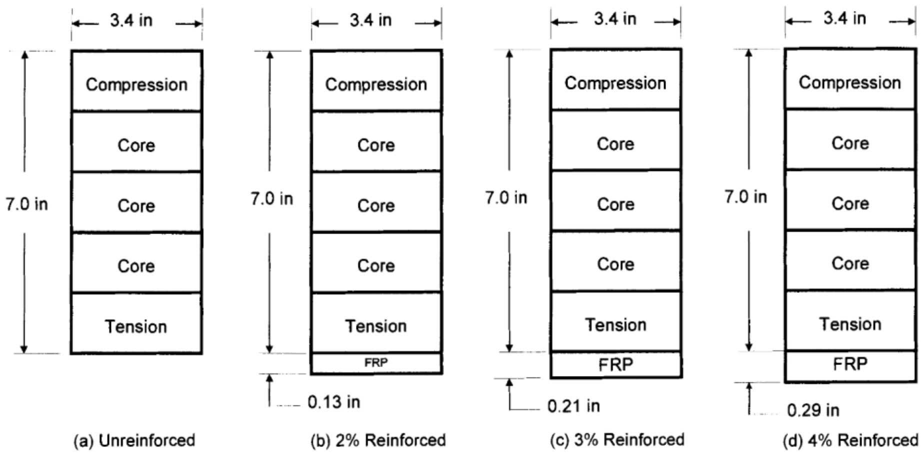
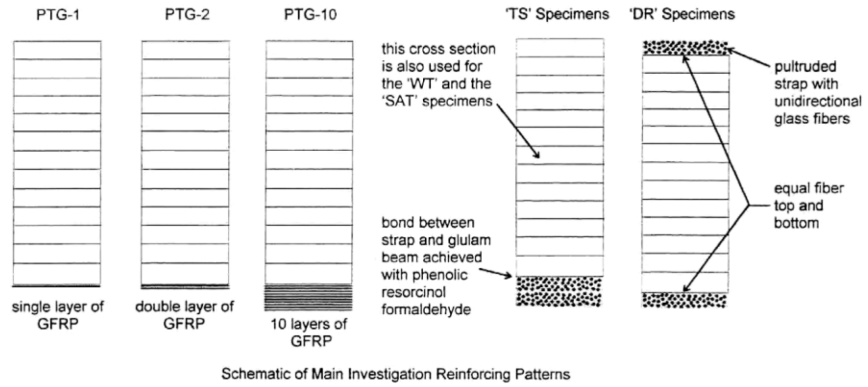


Figure 5.4. Cross sections of tested beams (Jordan 1998).

In Canada, Dorey and Cheng (1996) studied how a number of different parameters (fibre profile, fibre fraction, weathering effects, and beam size) affected the overall strength and stiffness of hybrid GFRP-GLT beams (Figure 5.5). The reinforced specimens showed a significant increase in stiffness and load-carrying capacity. Increases of 130 and 120% in load-carrying capacity and stiffness, respectively, were observed for beams reinforced only in tension, whereas increases of 150 and 200% were observed for beams reinforced in tension and in compression (fibre fraction of 6.7%). The weathering tests showed that there was no significant effect on the PRF bond between GFRP and timber. Beam size effects were inconclusive.



- **Figure 5.5. Cross sections of tested beams. Figure adapted from Dorey and Cheng (1996).**

In the yearly 2000s, the use of CFRP instead of GFRP to strengthen timber beams became increasingly popular. This was mostly due to improvements in production that lowered the cost of CFRP composites. In Japan, who is still nowadays the world's largest manufacturer of CFRP composites, Ogawa (2000), then working for Toho Rayon*, developed an industrial method to produce CFRP-reinforced GLT, based on the previous development of a new phenolic resin and a new CFRP sheet. The developed hybrid GLT-CFRP members showed, as in earlier studies, increased bending stiffness and strength, and a smaller coefficient of variation of the mechanical properties.

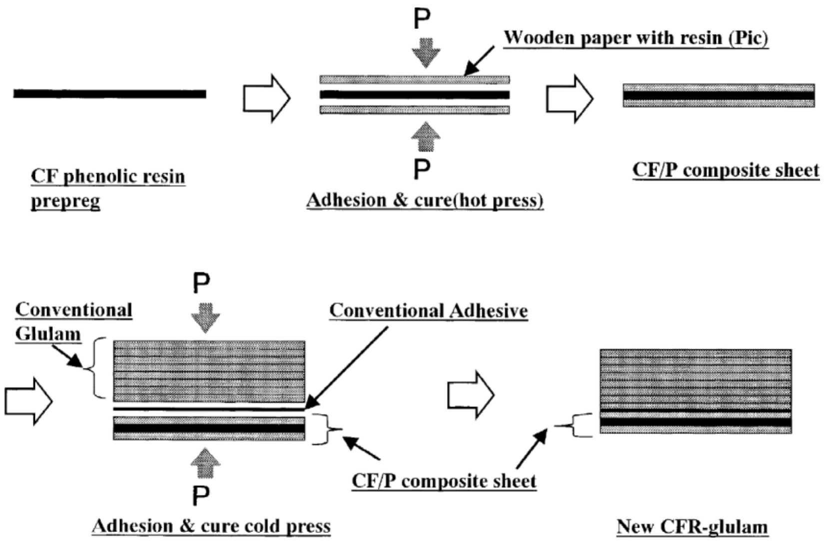


Figure 5.6. Production of CFRP-reinforced GLT beams (Ogawa 2000).

In Canada, Gentile (2000) and Gentile et al. (2002) studied the bending behaviour of creosote-treated Douglas Fir (*Pseudotsuga menziesii*) solid timber beams strengthened by GFRP rods. The solid timber beams had 100×300 mm^2 cross sections and the GFRP rods were installed in grooves cut on the side faces, 30 mm above the bottom (Figure 5.7). The GFRP rods were glued using an epoxy adhesive. The results showed no

* The company *Toho Rayon* is now *Mitsubishi Rayon*, who is one of the three global key players in the carbon fibre market, alongside with two other Japanese companies (*Toray* and *Teijin*).

influence of the reinforcement on the bending stiffness, but an increase in the load-carrying capacity and some additional plastic deformation, which was not observed in the strengthened beams. This small improvement in ductility might be a consequence of performing the tests under displacement control, even though it is stated that compressive failures were observed in timber in about 60% of the bending failures. The author mentions that 10% of the reinforced beams failed in shear and not in bending. For short-term quasi-static behaviour, there seemed to be an adequate bond between the creosote-treated timber and the epoxy resin. Simultaneously, Johns and Lacroix (2000) studied the use of CFRP strips to reinforce the tension face of solid timber beams and of GFRP strips to reinforced the tension and side faces (U-shaped strengthening). The strength increase for lower grade beams were of the order of 40-100%.

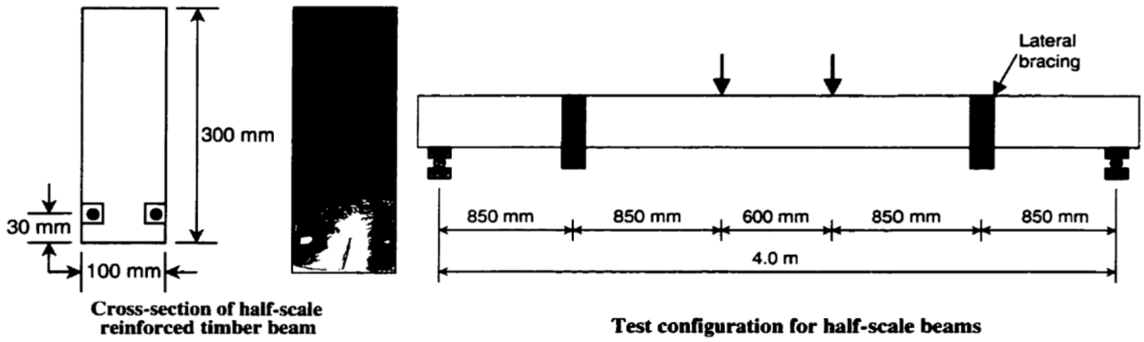


Figure 5.7. Creosote-treated Douglas Fir solid timber beams strengthened by GFRP bars (adapted from Gentile (2000)).

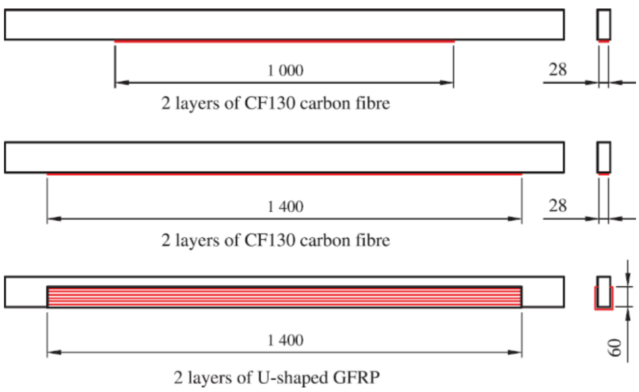


Figure 5.8. Reinforcement schemes (Johns and Lacroix 2000).

At the University of Karlsruhe in Germany, Blaß and Romani (2001) studied the load-carrying behaviour of hybrid FRP-GLT beams, namely regarding the influence of the amount and position of the FRP reinforcement and of different FRP laminates (CFRP and AFRP) and adhesives. As in previous studies, boards of a lower strength class were used in the production of the GLT beams, to show that the reinforcement could compensate the use of timber with lower structural value. The FRP laminates were glued in the tension side of the beam: some were externally-bonded under the outermost timber lamination; others between the outermost and the second timber laminations, to make it less exposed. The results showed an increase in bending stiffness, but an even more pronounced increase in the load-carrying capacity. In tests with an additional timber lamination below the FRP laminate, it was observed that the failure of this "protective" timber lamination was followed by debonding of the CFRP lamination, which reduced the load-carrying capacity in the debonded area. This debonding was not so pronounced in the tests on beams reinforced with AFRP laminates. It is also reported that in the test series with a high degree of reinforcement, compression wrinkles could be observed up to 1/3 of the cross-sectional height, showing that ductility can be mobilised through reinforcement. Regarding the adhesives, the authors concluded that hybrid GLT-FRP beams could be produced using adhesives and manufacturing processes commonly used by the timber industry.

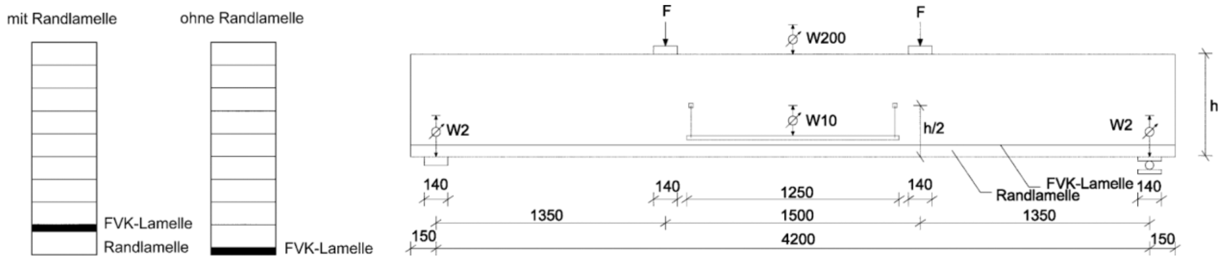
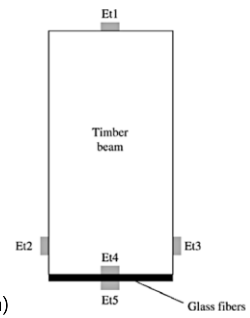
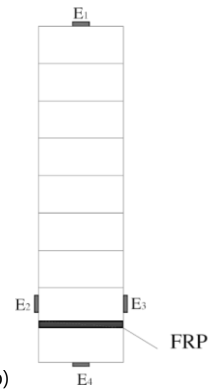


Figure 5.9. Cross sections and test set-up (Blaß and Romani 2001).

In Brazil, Fiorelli and Dias (2003) performed bending tests on solid timber reinforced with GFRP and with CFRP composites in the tension side. In this case, the fibres were laid on the surface of the beam and the epoxy adhesive was then applied on the fibres, creating the FRP composite and simultaneously gluing it to the timber surface (which had to be cleaned in advance). The volume of FRP relative to volume of timber was 1.0% for glass fibres and 0.4% for the carbon fibres. The authors report an increase in strength and stiffness of the reinforced timber beams, as well as a significant increase in ductility through the behaviour of timber in compression in the direction parallel to the grain. Some years later, Fiorelli and Dias (2011) tested structural-sized GLT beams reinforced with NSMR GFRP (a timber lamination was placed underneath the reinforcement). The reinforcement was an uni-directional GFRP fabric glued to the adjacent timber laminations with an epoxy adhesive. The percentage of reinforcement was 1.2 or 3.3% of the height of the beam, which was approximately 30 cm. Even though only two beams of each type were tested, the reinforced beams exhibited increased stiffness, approximately 20 and 33% for 1.2 and 3.3% of reinforcement, and increased load-carrying capacity, 54 and 100% for 1.2 and 3.3% of reinforcement. Failure of the reinforced beams is mentioned to occur in two stages: tensile failure of the outermost timber lamination on the tensile side (located underneath the GFRP reinforcement), followed by compression yielding of timber, followed by both debonding and tensile failure in timber. The last failure occurred at a load level approximately 19% higher than the initial failure, for the beams with 1.2% reinforcement, and at a level only 9% higher for the beams with 3.3% reinforcement.



a)



b)

Figure 5.10. Reinforced beams: a) solid timber (Fiorelli and Dias 2003); b) GLT (Fiorelli and Dias 2011).

In Northern Ireland, Gilfillan et al. (2003) tested GLT beam made with lower-grade C16 timber boards and reinforced with . Reinforced LVL beams, which exhibit a much lower variability of mechanical properties than GLT beams, were also tested. The reinforcement was in the form of pultruded strips of CFRP (with an epoxy matrix), three types of GFRP (with polyester, phenolic, and polyurethane matrixes), and a steel rod. Various reinforcement arrangements were tested: reinforcement only on the tension side; reinforcement on the tension and compression sides; reinforcement in longitudinal grooves. The results show that beams fabricated using low-grade GLT greatly benefited from the addition of reinforcement, exhibiting significantly enhanced strength and stiffness, ductile failure modes in compression parallel to the grain, and less variability. The authors report that the benefits of reinforcing LVL, which is comparably stronger and stiffer than solid timber, are less evident. The beams reinforced with a steel rebar exhibited a behaviour very similar to that of the beams reinforced with a CFRP strip only on the tension face.

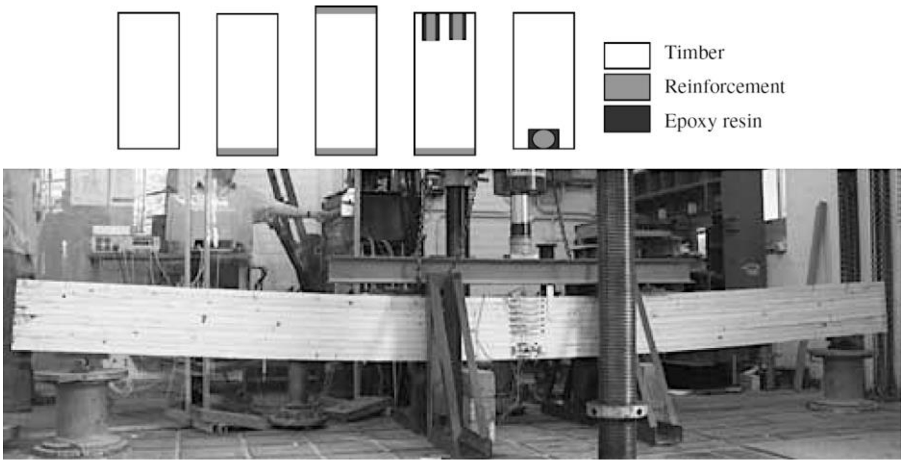


Figure 5.11. Reinforcement schemes and test set up applied in tests by (Gilfillan et al. 2003).

In Italy, Borri et al. (2005) evaluated different strengthening schemes on solid timber beams. The study included strengthening with CFRP laminations and rods (Figure 5.12) and was based on four-point bending tests performed under force-control. Regarding the load-carrying capacity, the results of the tests on beams with laminations on the tension side only, showed increases between 40 and 60%, compared to unreinforced beams, whereas the reinforcement with CFRP laminations on the edges of the beams showed an increase of

55%. Regarding the stiffness, the increase was moderate, around 20-30%. The behaviour was always linear elastic until brittle failure. The reinforcement with CFRP rods inserted in longitudinal grooves led to an increase of 30 (for a single rod) to 50% (for two rods) of the load-carrying capacity, compared to the unreinforced beams. Stiffness increase was approximately 22-25%.

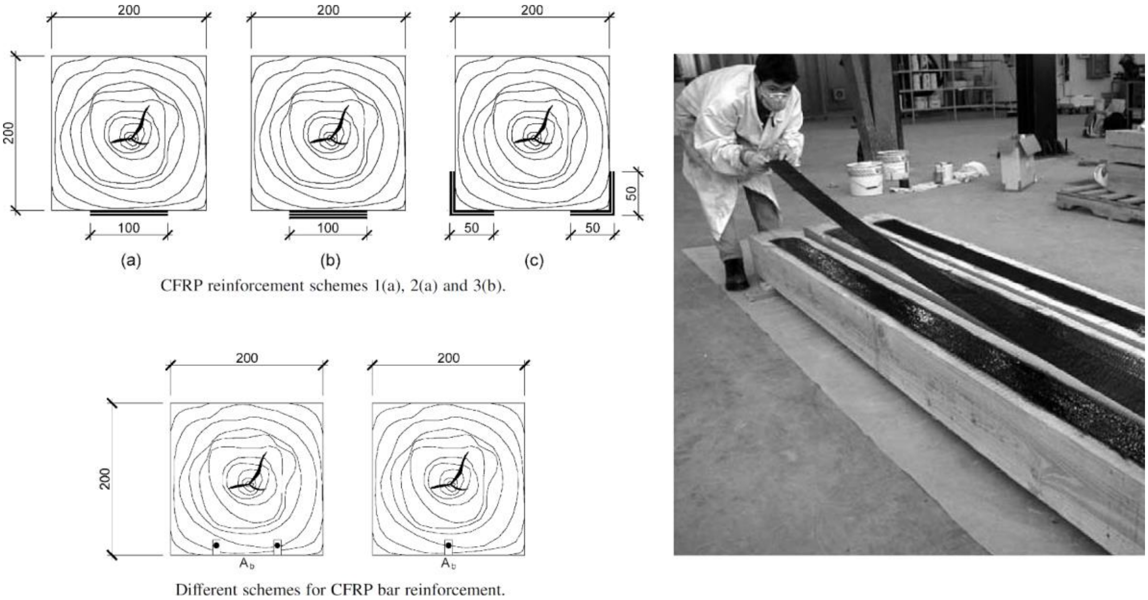
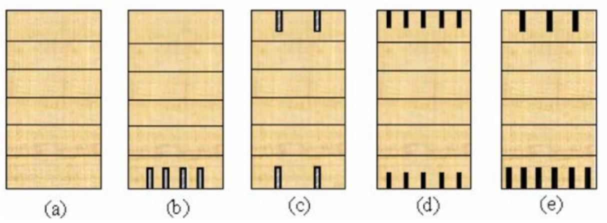


Figure 5.12. Reinforcement schemes and application of EBR CFRP sheets investigated by Borri et al. (2005). Figure adapted.

In Sweden, Kliger et al. (2007, 2008) studied the influence of reinforcement with steel plates and FRP laminates on the behaviour of GLT beams under four-point bending. In this case, the reinforcement was inserted in longitudinal vertical grooves (i.e. NSMR), to increase the shear area between the reinforcement and timber (at the cost of having less reinforcement close to the outermost edge of the cross section), and glued with an epoxy adhesive. The reinforcement was placed on the tension and compression sides of the beam, except for some tests in which the steel plates were only inserted in the tension side. The tests show that all reinforced specimens had approximately the same increased stiffness. The increase in bending capacity compared with the unreinforced GLT beam was between 57% and 96%. The authors state that the highest increase in load carrying capacity was for the beams reinforced with steel only in the tension zone (2% of gross cross section), but the plots show that the highest load-carrying capacity was reached by the GLT beams with reinforcement on both sides. All reinforcing schemes are reported to have a positive effect on ductility, which was due to yielding of the reinforcement or failure of timber in compression parallel to the grain. The reinforced beams also exhibited a reduced variability of the mechanical properties. The final failure is reported to be due to timber in tension, but an interesting aspect that was also observed was that the heavily reinforced beams failed in shear. This is a relevant aspect that had so far not been mentioned in previous studies. Still in Sweden, Johnsson et al. (2007) investigated the possibility of strengthening GLT beams using pultruded rectangular CFRP rods and studied the required anchoring lengths. In this case, the strength class of the GLT was GL 32h, which is quite high, and the CFRP rods were inserted in longitudinal grooves on the tension side and glued with an epoxy adhesive (i.e. NSMR). Regarding the results, the load-carrying capacity of the reinforced beams increased by 44-63%, the stiffness by 10%, and the failure mode changed from a brittle failure on the tension side of the beam to a ductile failure on the compression side.

Regardless of the positive results, the authors note that the long-term performance of such hybrid elements is mostly unknown and should be addressed.



Beam Type	Reinforcements				Adhesive System
	Type	Size	E-Modulus	% in c/s	
a	Blank	-	-	-	
b	Steel	4x(4mmx30mm)	210GPa	2%	SikaDur®-30
c	Steel	4x(4mmx30mm)	210GPa	2%	SikaDur®-330
d	Sika® CarboDur® H	10x(1.4mmx25mm)	300GPa	1.5%	SikaDur®-330
e	Sika® CarboDur® S	9x(1.4mmx30mm)	165GPa	2.5%	SikaDur®-330

Figure 5.13. Reinforcement schemes(Kliger et al. 2008).

In Taiwan, Li et al. (2009) studied the feasibility of reinforcing local wood species and tested small-scale solid timber elements reinforced with various layers of CFRP composite sheets on the tension side. As in the other studies, the results show that the increased strength of the reinforced specimens is related to the amount of reinforcement. It went from 39% for a single sheet of FRP to 61% for three sheets. In Malaysia, Yusof and Saleh (2010) also studied reinforcing solid timber beams made from local wood species with GFRP rods inserted in longitudinal groves (NSMR). The results showed that the load-carrying capacity of the strengthened beams was 20-30% higher than that of the beams without reinforcement and that the stiffness was also 24-60% higher. The authors concluded that bonding GFRP rods showed a good potential. In India, Nadir et al. (2016) studied the behaviour of small scale specimens of rubber wood reinforced with GFRP and CFRP sheets. For single- and double-layer GFRP sheets, the increase in stiffness was 26 and 46%, respectively, in load-carrying capacity it was 37 and 40%. For single- and double-layer CFRP sheets, the increase in stiffness was 36 and 64%, respectively, and in the load-carrying capacity it was 46 and 51%.

By the end of the 2000s, the major obstacles to the use of FRP to strengthen timber elements, e.g. selection of appropriate adhesives and production issues, were mostly overcome. Commercial success did not follow but research continued worldwide, however, focused on optimising detailing, trying new FRPs, timber species, and adhesives.

In the UK, Alam et al. (2013) studied the influence of the geometry, material properties, and position of the reinforcement on the bending behaviour of LVL beams (*Kerto S*), which exhibit lower variability of mechanical properties than other structural timber products. Four different materials were used to reinforce the LVL beams, namely mild steel, pultruded GFRP, pultruded CFRP and pultruded glass fibre reinforced polyurethane (FULCRUM), in the form of plates and rods. These were installed in longitudinal grooves with an epoxy adhesive. It was observed that the failure mode was dependent on the properties of the reinforcing material, the quality of the adhesive-reinforcement bond and the position of the reinforcement. Stiffer reinforcement materials (steel and CFRP) contributed to a greater increase of the bending stiffness, but regarding load-carrying capacity were not more effective than GFRP and FULCRUM. The positioning of reinforcements was found to be a key aspect, with lower volume fractions of rod and plate reinforcement located near the outermost tension and compression surfaces of the LVL beam being more effective than full-depth vertical laminates.

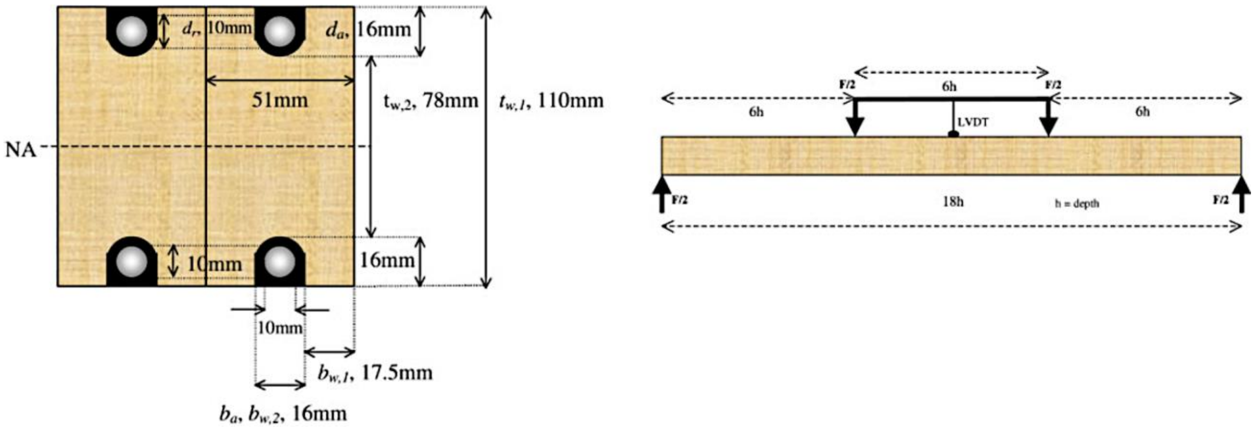


Figure 5.14. Reinforced cross section and test set up in a study by (Alam et al. 2013).

Also in the UK, Petkova et al. (2014) studied GLT beams reinforced with a U-shaped wrap of GFRP fabric, with a externally-bonded GFRP fabric on the tension side, and with a CFRP plate on the tension side. Even though only a single beam of each type was tested, the results show that the CFRP strengthening led to an increase of 167% in the load-carrying capacity and both GFRP strengthening led to an increase of 54%. The CFRP strengthening also exhibited higher elastic stiffness and reached a higher maximum displacement.

In the US, Gentry (2011) developed a method to strengthen the shear capacity of GLT beams using vertical epoxy-bonded GFRP glued-in rods, but also tested GLT beams reinforced with an additional longitudinal GFRP ply, inserted between the outermost and the second timber laminations in the tension zone, in addition to the rods. The idea was that the vertical glued-in rods would act as shear connectors between the FRP ply and the timber laminations, even if durability-related debonding occurred. During the tests, the initial failure was always in a lamination on the tension side of the beam, after which the tension stresses were mostly taken by the GFRP tension ply and load and displacement continued to increase. The bond between the longitudinal GRFP plies and the timber is reported to have been quite good, so the effects of the glued-in rods in the bending behaviour could not really be assessed.

In Ireland, Raftery and Harte (2009, 2011), Raftery and Whelan (2014), O'Ceallaigh et al. (2014), and Raftery and Kelly (2015) studied the use of FRP composites to strengthen low-grade GLT. Raftery and Harte (2009, 2011) tested beams reinforced with pultruded GFRP plates in the tension side, with and without an additional outermost timber lamination (which the authors called "sacrificial lamination"). The GFRP plates were made with an engineered thermoplastic polyurethane matrix, because it was thought that a ductile matrix would facilitate a better load sharing between the fibres than a brittle thermosetting resin, and glued to the timber using an epoxy adhesive. The reinforcement percentages were 1.26% and 1.12%. As in previous tests, the tension reinforcement caused timber to fail in compression parallel to the grain, which introduced significant ductility. Reinforced beams exhibited modest improvements in stiffness, but significant improvements in the load-carrying capacity and reduced variability. No premature delamination of the FRP plate or of the outermost timer lamination were observed, however, the authors conclude that failures of timber laminations in tension close to the FRP plate limit ductility and propose that these laminations should be of a better grade. Raftery and Whelan (2014) made a very similar study, but reinforcing low-grade GLT with GFRP rods glued-in longitudinal grooves in the tension side (near surface mounted reinforcement). Some beams were reinforced in the tension and compression sides. The rods were glued to the timber using an epoxy adhesive. The results show that the use of more small-diameter rods per groove, to increase the bond surface area, was not beneficial. A reinforcement percentage of 1.4% in the tension zone with circular routed grooves, led to a mean stiffness increase of 14% and a mean increase in load-carrying capacity of 68%, in comparison to the unreinforced GLT beams. The beams with 1.4% tension reinforcement and 1.4% compression reinforcement showed a mean stiffness increase of 30% and a mean increase in load-carrying capacity of 98.5%. O'Ceallaigh et al. (2014) performed a very similar study, but used of BFRP rods, instead of GRFP rods. Only bending stiffness was assessed and the authors report an average increase in bending stiffness of 16%, compared with unreinforced GLT beams, and note the reduced variability of the results of reinforced GLT beams. No direct comparison between the BFRP and the equivalent GRFP reinforcement (Raftery and Whelan 2014) was made. Raftery and Kelly (2015) also studied a very similar use of BFRP rods glued in longitudinal grooves in the tension side (near surface mounted). In this case, a reinforcement percentage of 1.4% led to a mean stiffness increase of 10% and a mean increase in load-carrying capacity of 23%, in comparison to the unreinforced GLT beams. These are smaller improvements than those reported by Raftery and Whelan (2014) using GFRP rods in beams with the same geometry, even though BFRP rods have higher mechanical properties. As in previous studies, reinforced beams exhibited higher ductility than unreinforced beams due to failure of timber in compression parallel to the grain. The level of ductility was influenced by the distance between the FRP rods and the neutral axis. Also no issues were reported regarding the bond between the BFRP rods and the timber.

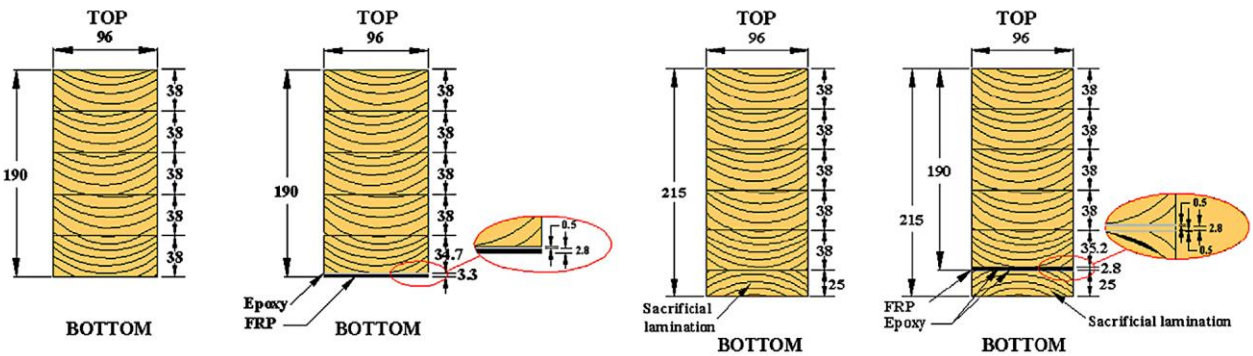


Figure 5.15. Reinforcement schemes studied by (Raftery and Harte 2011).

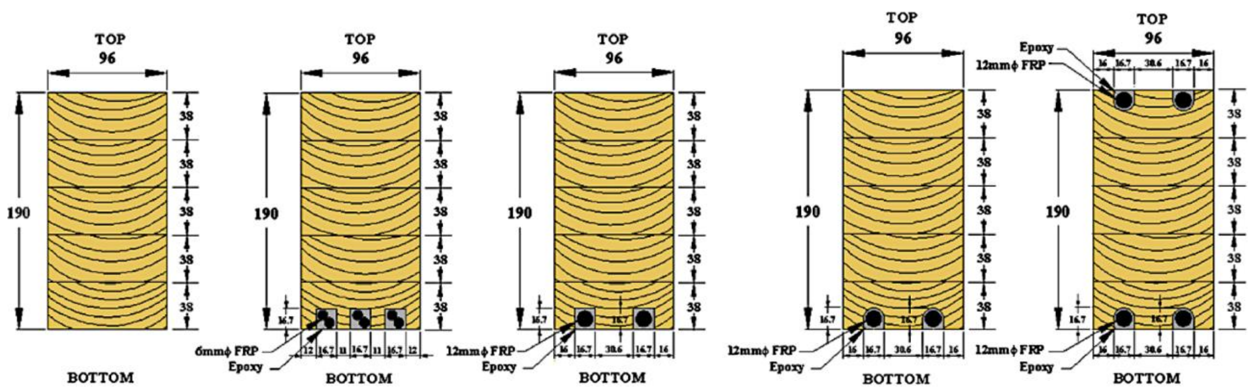


Figure 5.16. Reinforcement schemes investigated by (Raftery and Whelan 2014).

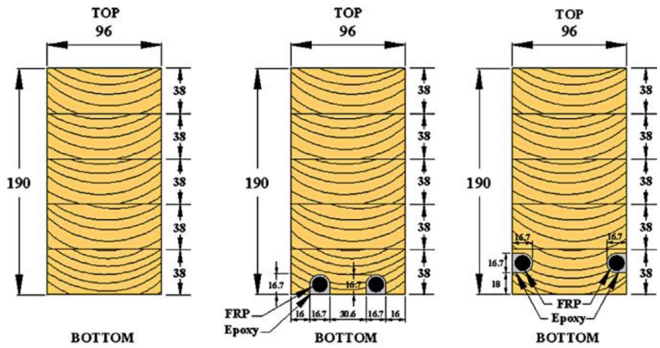


Figure 5.17. Reinforcement schemes investigated by (Raftery and Whelan 2014).

In Portugal, Branco et al. (2014) studied reinforced GLT slabs, i.e. GLT beams bent along the weak axis, which is not a common solution neither for slabs nor for beams. The slabs were tested with two spans and were reinforced in the tension zones (over the central support and at mid span). The types of reinforcement were externally-bonded uni-directional GFRP sheets and near-surface mounted GFRP rods, both glued to the timber using epoxy adhesives. The slabs reinforced with externally-bonded GFRP strips exhibited a 21% increase in load-carrying capacity and a moment redistribution at peak load of about 10%. Increasing the amount of reinforcing FRP sheet led to an increase of 44% in load-carrying capacity and a moment redistribution of about 9% at the peak load. The failure mode of these slabs reinforced with externally-bonded GFRP sheets was brittle, without ductile compression failure of timber in the direction parallel to the grain. On the other hand, slabs reinforced with near-surface mounted rods exhibited significant ductility, but had only between 1 and 10% increase in load-carrying capacity (depending on the amount of reinforcement), and a moment redistribution of about 10% and 25% at peak load. The effect of the reinforcement in the bending stiffness was not reported, but the reinforced slab elements reached higher ultimate displacements.

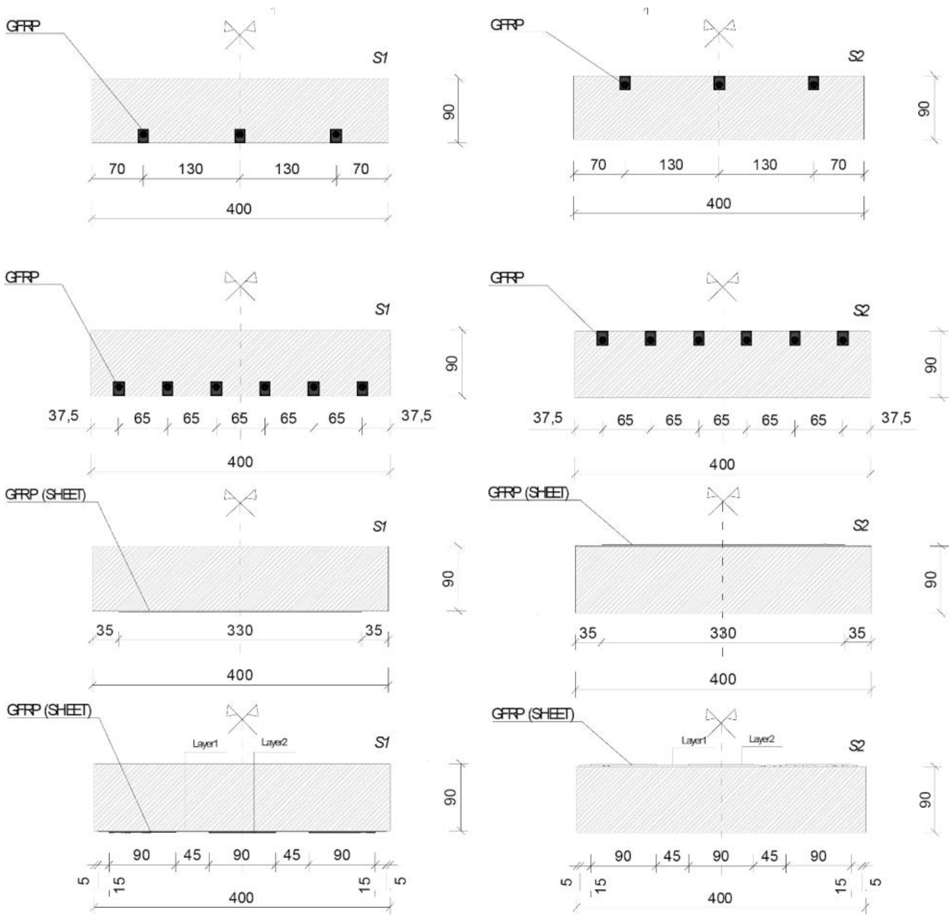


Figure 5.18. Reinforcement schemes studied by (Branco et al. 2014).

In Italy, Fossetti et al. (2015) performed tests on GLT beams reinforced with GFRP cords and with BFRP rods. The GLT beams were of strength class GL 24h and the reinforcements were introduced in longitudinal grooves made on the inside of the outermost lamination in the tension side. Unlike in other studies, the outermost lamination was reinforced before production of the GLT beam and was then installed with the reinforced grooves facing the second timber lamination. Regarding the results, the authors describe the behaviour of the beams reinforced with GFRP cords as "quite unsatisfactory", with no gains in load-carrying capacity and very high variability. This is attributed to problems soaking the cords in the resin. The authors state that the beams were nevertheless able to continue carrying loads even after tensile failure of the outermost lamella, but the force-displacement curves seem linear elastic until a brittle failure occurs. A different behaviour occurred with the beams reinforced with BFRP rods. In this case, a mean increase of 25% is in load-carrying capacity is reported, but the force-displacement curves also show an almost linear elastic behaviour until a brittle failure occurs.

In Australia, Subhani et al. (2017a) and Globa et al. (2018) studied the reinforcement of LVL beams with CFRP composites. LVL is the most common timber-based structural material in Australia and New Zealand. Subhani et al. (2017a) analysed two strengthening schemes: an uni-directional CFRP sheet glued to the tension side of the beam, which is only 45 mm wide; and an uni-directional CFRP sheet glued as a U-wrap around the tension side of beam and up to the middle of the side faces. The CFRO sheet was glued with an epoxy adhesive. The LVL beams reinforced only in the tension side showed a 10% increase in load-carrying capacity, a 14% increase in stiffness, and only 4% increase in ductility, compared with the unreinforced beams, whereas the LVL beams with the U-shaped reinforcement showed a 25% increase in load-carrying capacity, 20% in stiffness, and 30% in ductility. Globa et al. (2018) analysed the behaviour of similar LVL beam, but also under negative moments. The objective of the negative-moment tests was to assess the feasibility of connecting beams supported on opposite sides of a column. The same strengthening schemes of the previous study were analysed: an uni-directional CFRP woven fabric glued to the tension side of the beam, which is only 45 mm wide; and an uni-directional CFRP woven fabric glued as a U-wrap around the tension side of beam and up to the middle of the side faces. In the negative moment tests, the LVL beams were interrupted, but the uni-directional CFRP fabric was continuous over the column. In some negative-moment tests, an additional bi-directional CFRP fabric was wrapped along two sides of the column and over the beams. Regarding the positive-moment tests, the authors report that the uni-directional CFRP fabric applied as a U-wrap on the tension side of the beams increased the load-carrying capacity by 25% and the stiffness by 20%, compared with the unreinforced beams. The authors also report that the U-shape wrap reinforcement improved ductility, leading to a "gradual, progressive type of failure", but most force-displacement curves show a mostly linear elastic brittle behaviour. The reinforcement applied only on the tension zone (45 mm wide) led to increases in load-carrying capacity and stiffness of about 10% and 14%, respectively. Regarding the negative-moment tests, the combination of uni- and bi-directional CFRP fabric applied over the beam-to-column connections increased the load-carrying capacity, reduced the variability of the results, and is stated to have provided "substantial structural continuity between beams". Regardless of the structural benefits of these reinforcement schemes, the economic benefit of using such significant amount of high-quality FRP composites to reinforce common floors beams is not clear, compared with, e.g., simply reducing spacing between the beams.

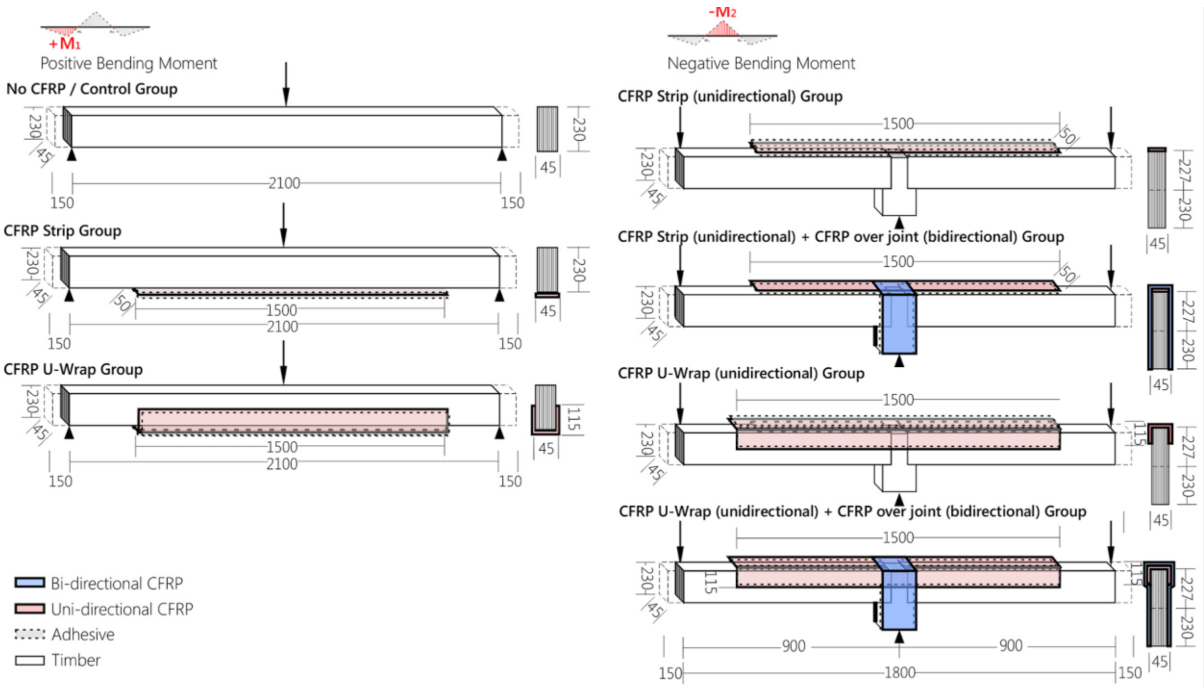


Figure 5.19. Reinforcement schemes investigated by (Globa et al. 2018).

In China, namely at Nanjing Technical University, Yang et al. (2016) and Jiang et al. (2018) have studied the reinforcement of GLT beams and of timber-concrete composite (TCC) beams. Yang et al. (2016) conducted an extensive experimental campaign on GLT beams reinforced with GFRP bars, GFRP plates, CFRP plates, and ribbed steel bars. The strengthening FRP and steel elements were glued to the timber member using an epoxy adhesive. Various strengthening schemes were analysed, comprising near-surface mounted bars and plates (inserted in longitudinal grooves) and FRP plates applied on the outer timber laminations and covered by an additional timber lamination. The results show that the load-carrying capacity and stiffness of the reinforced beams were, on average, 58 and 28% higher, respectively. The beams reinforced with bars reached higher load-carrying capacities than the beams reinforced with plates and the beams with steel reinforcement reached the higher load-carrying capacities. CFRP reinforcement was only slightly better than similar GFRP reinforcement. Beams reinforced with both tensile and compressive reinforcements showed higher stiffness, rather than load-carrying capacity, due to the premature buckling or delamination of the compressive reinforcement. The authors recommend using stiffer compressive reinforcement. Increasing the reinforcement ratios led to timber failures in compression parallel to the grain, in addition to tension timber failures. The force-displacement behaviour of all tested beams seems mostly linear elastic with brittle failures. No bonding problems between FRP or steel and timber were reported. Jiang et al. (2018) tested TCC beams reinforced with a CFRP sheet on the tension side and U-shaped CFRP sheets as shear reinforcement next to the supports. The test results show that the load-carrying capacity, the stiffness, and the ductility increased significantly with the CFRP strengthening. Increasing the thickness of the CFRP sheets increased the bending load-carrying capacity of the beams, up to a limit above which the GLT beams failed in shear rather than bending, even with the U-shaped external reinforcement. Finally, the level of reinforcement had no significant influence on the stiffness of the composite beams.

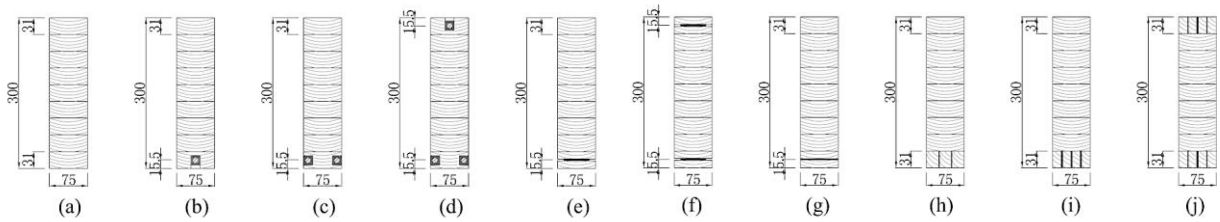


Figure 5.20. Reinforcement schemes studied by (Yang et al. 2016).

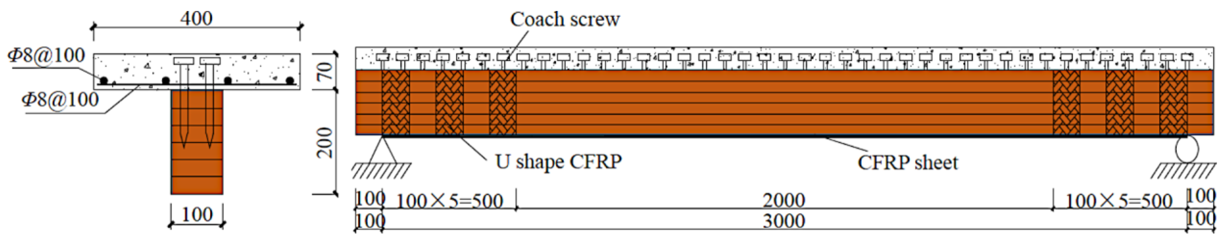


Figure 5.21. Reinforcement scheme investigated by (Jiang et al. 2018).

In the UK and Italy, Corradi et al. (2017) tested more than 200 relatively small scale solid timber fir (*abies alba*) and oakwood (*quercus*) reinforced beams. The beams were reinforced on the tension side, with the reinforcement fibre textile and epoxy resin applied by hand lay-up. The authors report that the reinforcement increased the strength up to 122% and reduced the values of the coefficient of variation of the results up to 63%. Failure was always brittle and in timber in tension. No significant difference was observed between the GFRP and CFRP reinforcements. In Spain, de la Rosa García et al. (2016) tested small scale Valsain pine (*pinus*) solid timber specimens reinforced with uni-directional BFRP and CFRP fabrics and bi-directional CFRP fabric. The reinforcement fibres were applied with an epoxy resin and glued as a U-shaped wrap on the tension side of the beam. The tests were only performed to assess the influence of the reinforcement on the stiffness. As expected, the results show that strengthened beams exhibit a higher stiffness. The bi-directional CFRP fabric led to a stiffness increase far greater than other tested fabrics.

In Iceland and Norway, Thorhallsson et al. (2017) studied the behaviour of GLT beams reinforced with uni-directional BFRP mats and GFRP laminates in the tension zone. The GLT beams were of strength classes GL 32h and GL 30c and the FRP reinforcement was glued on the tension face using an epoxy resin. The GL 32h beams were reinforced with 0.5% (FRP area to timber area ratio) of either BFRP or GFRP composites; the GL 30c beams were reinforced with 0.75% of BFRP composites. In comparison with the unreinforced beams, the reinforcement with GFRPs exhibited the higher increases in stiffness and load-carrying capacity, 21 and 57%, respectively, whereas the same reinforcement ratio of BFRP led to increases of 20 and 43%, in stiffness and load-carrying capacity, respectively. Reducing the strength class of GLT from GL 32h to GL 30c while simultaneously increasing the reinforcement ratio of BFRP from 0.5 to 0.75% actually led to increases in stiffness and load-carrying capacity of only 11 and 37%. Since no force-displacement curves are provided by the authors, it is not possible to evaluate if the reinforcement is able to introduce any ductility.

In Switzerland, Blank (2018) studied the behaviour of Norway spruce (*picea abies*) GLT beams reinforced with uni-directional GFRP and CFRP fabrics. The objective was to produce hybrid beams that were able to undergo extensive plastic deformations and redistribute bending moments. The FRP fabrics were installed in recesses made in the timber laminations to be used in the tension zone and a one-component polyurethane adhesive certified for production of GLT was used to simultaneously form the fibre matrix and the bond to the timber laminations. Hybrid timber-FRP beams with various strengthening schemes were tested in four-point bending: beams with a single strengthened timber lamination as the outermost lamination; beams with three strengthened timber laminations as the three outermost laminations and with varying reinforcement ratios. The objective of having the strengthening in various timber laminations instead of a single highly reinforced lamination was to avoid stress concentrations and "bridge" over defects in a wider tension zone. The results show that only the beams with the highest reinforcement ratios did not exhibit brittle failures (i.e. reinforcement ratios of 0.6% to GFRP and 0.4% to CFRP). The brittle failures were either due to failure of the reinforcement (not enough strength of the reinforcement) or uncontrolled crack growth in timber (not enough stiffness of the reinforcement, influenced not only by the MOE of the FRP composite but also by the activated bond length). Blank (2018) reports that beams with low reinforcement ratios exhibit these two types of brittle failure, but that CFRP-reinforced beams failed more often due to limited strength of the reinforcement (i.e. failure of the reinforcement), whereas GFRP-reinforced beams fail more often due to limited stiffness of the reinforcement (uncontrolled crack growth in timber). The beams with high reinforcement ratios exhibited failures that are more ductile, due to failure of timber in compression parallel to the grain, but this increased ability to deform plastically was limited by shear failures in timber. This shows that even though bending reinforcement might increase the load-carrying capacity and stiffness and even introduce some ductility, it simultaneously makes other brittle failure modes more prominent.

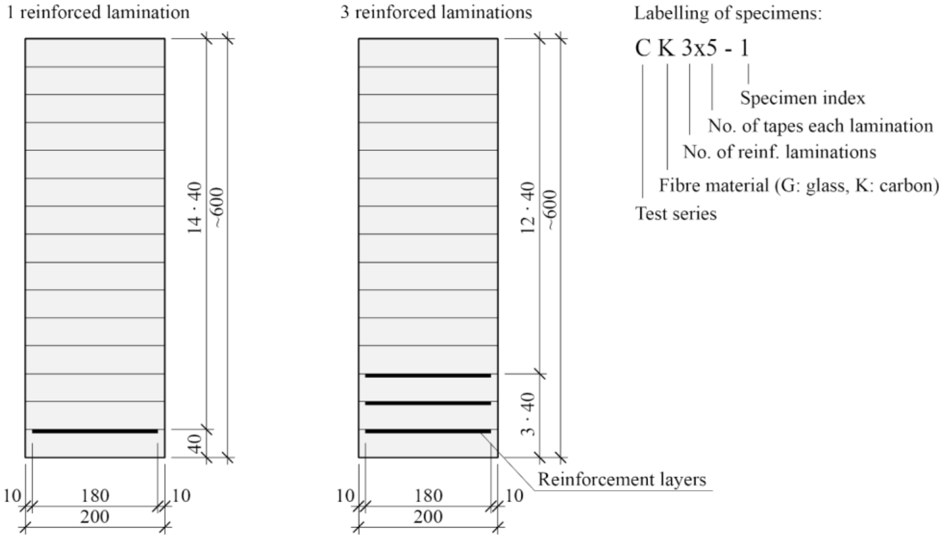


Figure 5.22. Longitudinal tension reinforcement schemes investigated by (Blank 2018).

5.1.1.2 Bending – pre- and post-tensioned strengthening

Pre- and post-tensioning of structural members has traditionally been done with steel rods or strands using mechanical anchoring systems. Some of the issues with using steel are the initial and time-dependent stress losses (e.g. slip at the anchorages, relaxation of steel) and the increased corrosion that occurs in steel members in tension. Replacing the steel tensile components with FRP composites could have some advantages, namely regarding corrosion resistance and reduced initial and time-dependent stress losses. Unlike the pre-/post-tensioning systems with steel tensile components, FRP-based systems do not usually rely on mechanical anchorages, because it is difficult to achieve a good grip without damaging the FRP composite in the direction perpendicular to the fibres, but are instead based gluing the FRP plate or rod to the structural member. Mechanical anchorages for FRP-based systems have been recently developed for FRP composite plates (Mohee and Al-Mayah 2017), but rely on bulky anchor heads that would anyway be difficult to attach to timber members. Bonding pre-stressed steel elements (usually rods or strands) to timber presents some shortcomings, namely because due to the high difference in the modulus of elasticity of both materials, small elastic and creep deformations in timber will induce large tension losses in steel.

The early research on pre- and post-tensioned strengthening, i.e. active strengthening, of timber members began in the U.S.A. in the 1960s and continued until the late 1990s. The first was reported by Bohannan (1964), in the U.S.A., who tested GLT beams post-tensioned with unbonded steel strands. The steel strands were not centred in the cross section, but in holes in the tension zone (i.e. longitudinal grooves made in the lamellas before production of the GLT beams), to avoid having stress concentrations in a zone where the shear stresses are high. Results showed a 31% increase in strength and a corresponding 50% decrease in variability.

In 1985, Engebretsen (1991) filed a patent outlining a pre-tensioning strategy based on gluing the passive FRP lamination while cambering the timber beam. The FRP lamination is activated when the cambering mechanism is removed and the beam tries to return to its initial configuration.

Almost 30 years later, still in the USA, Triantafillou and Deskovic (1992) studied the use of externally-bonded pre-tensioned uni-directional CFRP sheets on small scale solid timber specimens. The pre-tensioned CFRP sheet was bonded to the tension face and the specimens were tested in 3-point bending. Pre-tensioning was achieved by gluing steel elements at the ends of the CFRP sheets, which were then attached to a fixed anchorage at one end and to a hydraulic jack at the other. The timber specimen was then glued to the tensioned sheet. The maximum pre-tensioning force was assessed by load-and-release tests, in which the pre-tensioning force was slowly reduced until debonding or failure of timber in shear occurred (the allowed pre-tensioning force is then the difference between the initial pre-tensioning force and the pre-tensioning force at which debonding or timber failure in shear occurred). Compared with unreinforced specimens: the reinforced but not pre-tensioned specimens exhibited higher load-carrying capacity, stiffness and ductility; the pre-tensioned specimens exhibited an even higher load-carrying capacity and stiffness, but similar ductility levels (Figure 5.23).

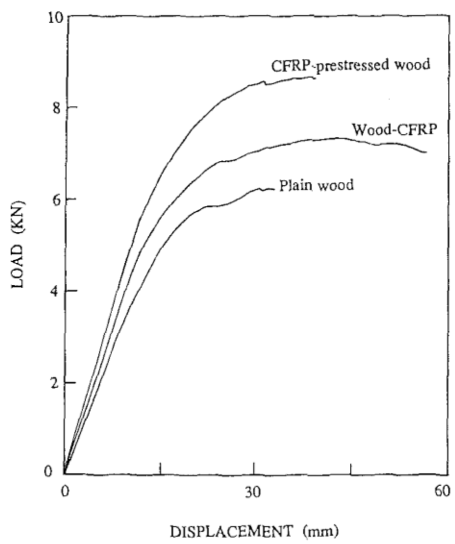


Figure 5.23. Three-point bending load versus crosshead displacement curves for plain, reinforced, and prestressed wood (Triantafillou and Deskovic 1992).

Only a few years later, Galloway et al. (1996) tested GLT beams with internally bonded pre-tensioned Kevlar sheets in the tension zone (Figure 5.24). The FRP reinforcement consisted of a woven tape of bare uni-directional Kevlar fibres (with and without pre-tensioning) glued between selected timber laminations, using the same adhesive that was used to glue the other timber laminations together. GLT beams with higher-quality laminations in the tension zone were also tested. The bond shear strength was assessed by shear tests on glued timber blocks, with the reinforcement in the glue line. The results seem to show that pre-tensioning the reinforcement did not lead to higher load-carrying capacities than using the reinforcement without pre-tensioning, unless higher quality timber laminations were used around the pre-tensioned reinforcement glue lines. Nevertheless, the benefits of pre-tensioning the reinforcement are not at all clear, since no tests were performed on GLT beams with higher-quality timber laminations around the passive reinforcement, and the results on the beams without reinforcement but only higher-grade timber laminations are not reported.

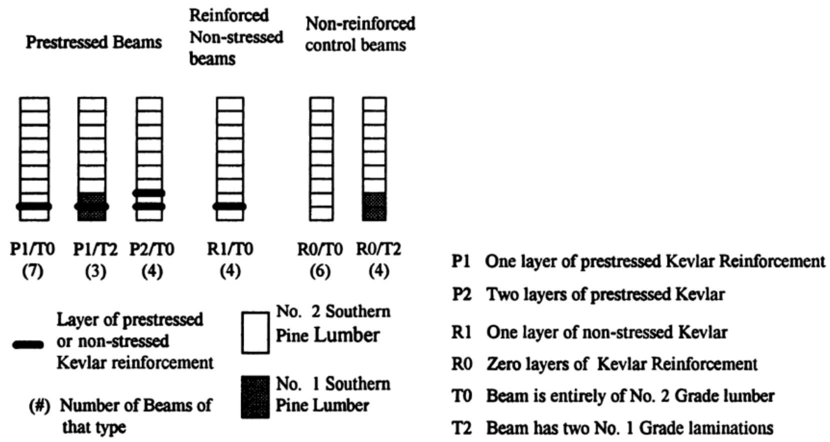


Figure 5.24. Test matrix of reinforced, pre-stressed, and control beams (Galloway et al. 1996).

In the 2000s, research on pre- and post-tensioned strengthening of timber members was being conducted worldwide. In the UK, Rodd and Pope (2003) initiated research on pre-tensioned FRP strengthening of timber members, by studying how it could be used as a means of better utilising low quality wood in GLT timber beams. These authors used pultruded GFRP sheets, because of their lower MOE compared to that of steel, which meant that the sheets had to be strained to a much higher level than steel in order to reach their limiting strength. Therefore, losses due to elastic and time dependent shortening of the timber after release the pre-tension force would be relatively smaller than those with steel tendons. The use of GFRP sheets ("flat strips") was due to their relatively larger surface area than traditional round or square cross-section of steel tendons, which would lead to lower bond stresses and reduce the risk of delamination. Guan, Rodd, and Pope (2005) tested a single pre-tensioned hybrid FRP-timber beam. They used a pre-tensioned GFRP sheet positioned between the outermost timber laminations in the tension zone (Figure 5.25), following a pre-tensioning method similar to that of Triantafillou and Deskovic (1992). The behaviour of the beam was linear elastic with a brittle failure due to a failure in the outermost timber lamination. The authors note that it is likely that pre-tensioned timber beams will always behave in a brittle manner if no additional ductile reinforcing materials are used or if the tensile zone is not over-reinforced to force the compressive zone into plastic behaviour.

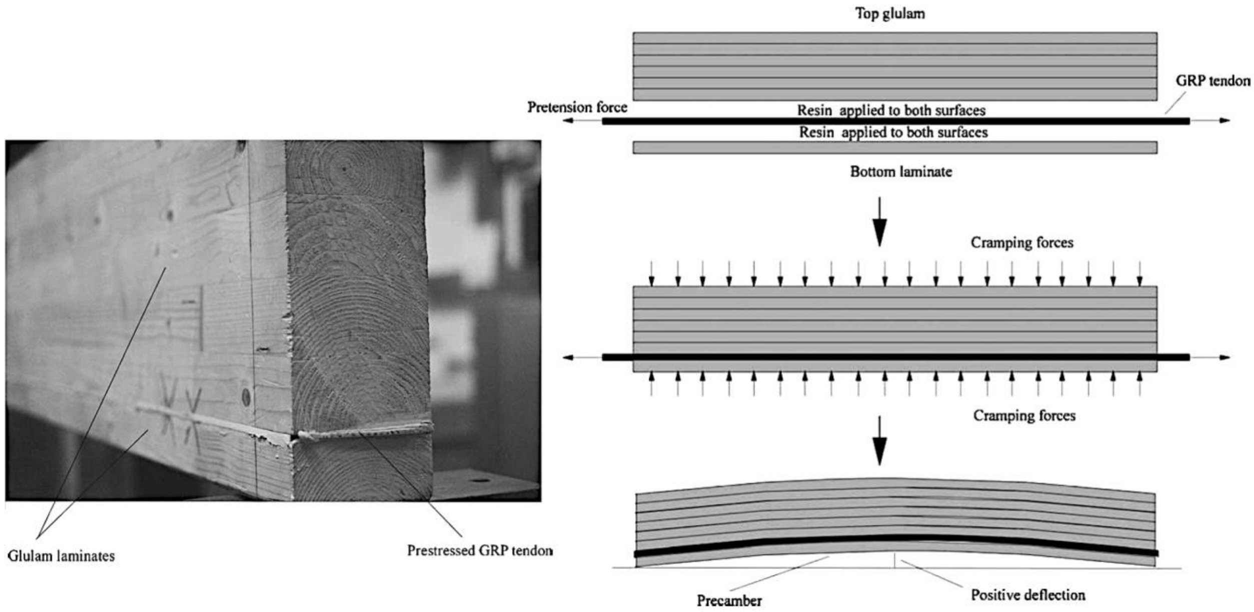


Figure 5.25. End view of the pre-stressed glulam beam and schematic pre-stressing processes applied by (Guan et al. 2005).

In Italy, Borri et al. (2005) performed tests on solid timber beams strengthened with pre-stressed CFRP laminations on the tension side (Figure 5.12). The solid timber beams were 200×200x4000 mm^3 and the pre-stressed CFRP lamination was glued with an epoxy adhesive. The pre-stressing was applied by using Engebretsen's (1991) method of gluing the passive FRP lamination while cambering the timber beam, in which the FRP lamination is then activated when the cambering mechanism is removed and the beam tries to return to its initial configuration. The bending load applied by Borri et al. (2005) was about 25–35% of the ultimate bending load-carrying capacity of the solid timber beam. The results showed that the pre-stressed beams exhibited a similar load-carrying capacity and stiffness as the timber beams reinforced with passive CFRP laminations. Borri et al. (2005) note that these results were unexpected, since previous tests on small-scale specimens had shown significant increases in stiffness, but showed the need to perform test on a structural scale.

In Switzerland, Brunner and Schnüriger (2005) tested GLT beams pre-tensioned with high-strength CFRP laminates, but using the gradient anchoring system developed at Empa by Stöcklin and Meier (2001). The idea behind this anchoring system was to reduce the high shear stresses that occur at the ends of the strengthening strips. This was achieved using a special device that allowed to gradually reduce the pre-tensioning forces towards the ends of the strips, increasing the development length of the pre-tensioning force so that the shear stresses remained within adequate limits. The CFRP laminates were applied on the tension face of the GLT beams and glued with a specially developed epoxy adhesive. The results of the four-point bending tests showed that all the tested beams (i.e. the unreinforced GLT GL 32 beams, the non-tensioned hybrid GLT-CFRP beams, and the pre-tensioned hybrid GLT-CFRP beams) exhibited a linear elastic behaviour followed by brittle failures. The authors state that this was because the amount of reinforcement was not enough to induce significant plastification in the compressive zone. Nevertheless, the pre-tensioned beams exhibited a slightly higher bending load-carrying capacity (34%) than the strengthened but not pre-tensioned beams (22%), compared to the unreinforced GLT beams. Regarding bending stiffness, there was not significant difference between the pre-tensioned and the strengthened but not pre-tensioned beams, but both were about 20% stiffer than the unreinforced GLT beams. A few years late, Brunner (2008) performed additional tests with higher reinforcement ratios, in order to force the compressive zone into plastic behaviour and, therefore, add some ductility to the behaviour of the pre-tensioned beams. These new tests were made on GL 24 GLT beams and three layers of the same pre-tensioned CFRP strips were applied in sequence, instead of one single strip. During the tests, the outer CFRP laminations are reported to have delaminated prematurely, causing load-drops of about 10-15% (tests were probably performed under displacement control), but the beams were able to reach higher loads until timber laminations failed in the tension zone. Nevertheless, no significant ductility seems to have been added by the higher reinforcement ratio. The author concludes that the delamination problem must be solved before strengthening with several multiple layers of pre-tensioned stressed CFRP laminations can be employed. An alternative would be to use a single wider and/or thicker CFRP lamination, but these were apparently not being produced at the time.

Also in Switzerland, Lehmann et al. (2006) and Lehmann (2015) studied the pre-tensioning strategy outlined by Engebretsen (1991), based on gluing the passive CFRP lamination while cambering the timber beam. In this case, the CFRP laminated is activated when the cambering mechanism is removed and the beam tries to return to its initial configuration. The level of cambering and corresponding stresses in the CFRP lamination were assessed based on preliminary tests on small-scale specimens. The tested full-scale beams were made from GL 24h GLT and the gluing of the CFRP laminations involved a complex procedure that involved a heating system to cure the adhesive while clamps pressed the CFRP lamination against the timber surface. The results of four-point bending tests showed that early delamination of the CFRP laminations did not occur, that the bending stiffness of the pre-tensioned beams increased on average by 14% and the load-carrying capacity by approximately 30%. This value might be an overestimation because it is not based on a comparison to tests of unreinforced GLT beams, but on a comparison to the load at which the first "significant partial failure with load redistribution from timber to CFRP" occurred. Failure is reported to have always started in the timber laminations in tension and no relationship between pre-stressing and ductility could be established, with many test specimens exhibiting brittle failures. Since no tests were performed on reinforced beams without pre-tensioning, the conceivable benefits of pre-tensioning cannot be compared to the much simpler strengthening alternative of simply applying the CFRP laminations with pre-tensioning.

In Portugal, Balseiro (2007) used the same pre-tensioning technique to strengthen GLT GL 24h beams with CFRP laminates glued using an epoxy adhesive. The test results showed that, compared to the unreinforced GLT beams, the load-carrying capacity was 26% for reinforced but not pre-tensioned beams and only 19% for the pre-tensioned beams. The initial linear elastic stiffness of all the tested beams was very similar. The presence of passive reinforcement led to higher ductility levels, even though a high variability was observed in this regard. On the other hand, the effect on ductility by pre-tensioning the reinforcement could not be clearly established (like in the tests by Lehmann (2015)).

In the U.S.A., Dagher et al. (2010) tested GLT beams reinforced with pre-tensioned GFRP laminations in the tension side. The PRF adhesive was applied to the GFRP reinforcement after pre-tensioning it and that the GLT beam was then placed on top of the reinforcement and clamped to it, applying an average clamping pressure of 1.0 MPa. However, it is reported that "after clamping, but before the adhesive began to cure, the pre-tensioning (…) forces were released". It is not clear why this was done and very high pre-stressing losses could be expected. However, prestressing-induced camber of the beams was observed, meaning that not all pre-stressing forces were lost. In fact, Dagher et al. (2010) report a 2% loss of the original pre-stressing after 12 days (measured through strain gauges installed in the GFRP laminations). Therefore, it seems to indicate that the clamping pressure during curing was enough to hold the prestressing forces. The pre-stressed beams exhibited a load-carrying capacity approximately 95% higher than the GLT beams and 38% higher than the conventionally reinforced beams.

In Italy, De Luca and Marano (2012) tested GLT beams reinforced with pre-tensioned (i.e. active) and not pre-tensioned (i.e. passive) bonded steel rods. These authors glued the steel reinforcement both in the tension and compression zones, but the pre-tensioned reinforcement was only applied in the tension zone (Figure 5.26). The same polyurethane adhesive was used in all configurations. The results of the four-point bending tests showed that, compared to the unreinforced beams, the beams with passive reinforcement showed an average increase of 48% in the load-carrying capacity, whereas the beams with active reinforcement in tension showed an increase of only 14%. Regarding the initial linear elastic stiffness, compared to the unreinforced beams, beams with passive reinforcement showed an average increase of 26% and beams with active reinforcement in tension showed an increase of 38%. The reinforced beams tend to show a slightly non-linear behaviour shortly before failure, but the reinforcement does not introduce any significant ductility[†].

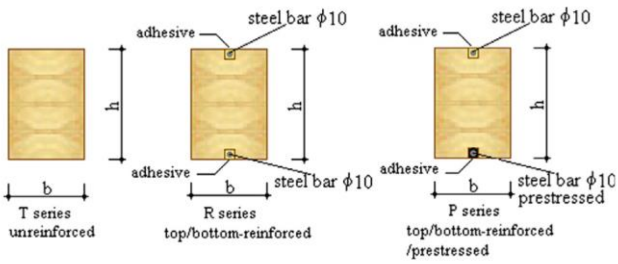


Figure 5.26. .Details of the cross section types investigated by (De Luca and Marano 2012): unreinforced GLT beams; GLT beams with passive reinforcement (tension and compression) and; GLT beams with active reinforcement (tension) and passive (compression).

In the UK, McConnell et al. (2015) tested GLT beams strengthened with pre-tensioned un-bonded and bonded BFRP rods (Figure 5.27). In this case, the authors used GLT beams of strength class GL 28c and an epoxy adhesive to glue the tendon to the beam. The results of four-point bending tests showed, somewhat surprisingly, that the presence of bonded but passive tendons did not increase the load-carrying capacity of the beams. The authors stated that this could be due to the variability of timber properties, but it seems more likely that the load-carrying capacity of this system is governed by the outermost timber lamination and that the reinforcement is not stiff enough to "bridge" initial cracks that might develop in the timber laminations. In addition, the very small width of the timber laminations (only 45 mm) increased the influence of any defect, since the stresses cannot be redistributed so easily. The load-carrying capacity of these reinforced GLT beams exhibited, however, slightly less variability than the unreinforced GLT beams. The post-tensioned unbonded reinforcement (load-transfer to the GLT beam through steel plates on the end surfaces) also did not increase the load-carrying capacity of the beams, compared to the unreinforced GLT beams. Finally, the beams with bonded pre-tensioned BFRP rods

[†] The values of ductility calculated by the authors are not based on the displacement at the failure load, but at the ultimate load, which occurred after significant load drops (when the reinforcement was still holding the timber member together but the carried load was much smaller than the maximum load).

exhibited a slightly higher load-carrying capacity (about 15%) than the unreinforced GLT beams. The bending stiffness was approximately the same in all tested configurations, as can be seen in the published force-displacement curves. The bending behaviour of all the tested beams is linear elastic until brittle failure of timber in tension. In a previous study, McConnell et al. (2014) made the same tests, but using steel rods as reinforcement. In this case, the average increase in load-carrying capacity compared to the unreinforced GLT beam was 30% for the passive (i.e not pre-tensioned) bonded steel reinforcement, 18% for the pre-tension but unbonded steel reinforcement, and 40% for the pre-tensioned bonded steel reinforcement. Regarding initial elastic stiffness, the pre-tensioned but unbonded steel reinforcement exhibited the same stiffness as the unreinforced GLT beam, but the configurations with passive and active bonded steel reinforcements exhibited 17 and 23% higher values, respectively. The authors state that the reinforced beams exhibited ductile failure modes "characterised as compressive shear", but it is not clear what this actually means and no significant ductility can be inferred from the published force-displacement curves.

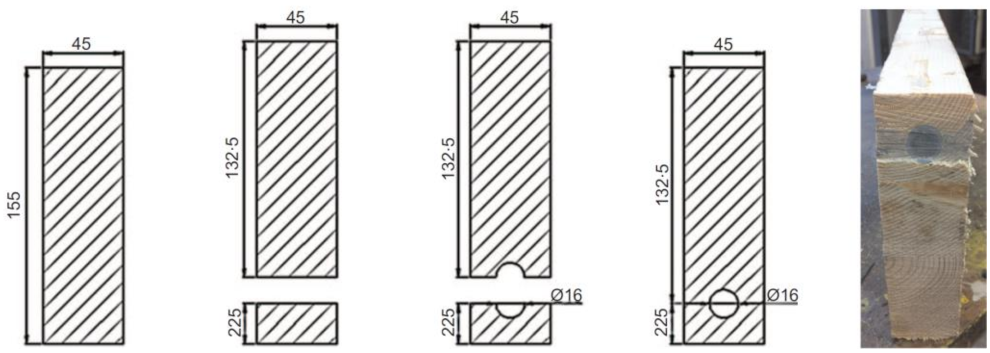


Figure 5.27. Diagram of initial beam preparation and final cross section (McConnell et al. 2015).

In Sweden, Kliger et al. (2016) used on GLT beams a method initially developed by Al-Emrani and Haghani (2014) to glue pre-tensioned CFRP laminates to concrete members. This method allowed reducing the level of pre-tensioning towards the ends of the reinforcement FRP, similarly to the method developed by Stöcklin and Meier (2001) for concrete members and used by Brunner (2008) in GLT beams. In the beams tested by Kliger et al. (2016), a pre-tensioned laminate was placed in a groove filled with epoxy in the tension zone. The authors state that, compared to unreinforced GLT beams, the increase in load-carrying capacity of the beams in four-point bending tests was between 11 and 18% (for increasing pre-tensioning forces) and the increase in stiffness was between 33 and 48% (also for increasing pre-tensioning forces). However, since no comparison is made to strengthened but not pre-tensioned beams, the improvement caused by the pre-tensioning cannot really be assessed.

In Iceland and Norway, Thorhallsson et al. (2017) tested GLT beams strengthened with pre-tensioned BFRP mats in the tension zone. GLT beams were of strength class GL 30c and the reinforcement was glued to the outermost timber lamination, using an epoxy resin, while cambering the beam (as done by Lehmann et al. (2006)). Compared to the GLT beams with passive BFRP reinforcement, the pre-tensioned BFRP reinforcement increase the load-carrying capacity by 20% and the stiffness by 8%, on average. The reinforced beams cannot be directly compared to the unreinforced GLT beams because these were of strength class

GL 32h. However, if this aspect is disregarded, the average increase in load carrying capacity, compared to the unreinforced GLT beams of a higher strength class was 64% for the beams with pre-tensioned BFRP reinforcement and 37% for the beams with passive BFRP reinforcement. Regarding stiffness, the reported increases are 19% for the beams with pre-tensioned BFRP reinforcement and 11% for the beams with passive reinforcement.

5.1.1.3 Shear and tension perp. to grain (incl. notches at supports)

The first studies on using FRPs to reinforce timber members loaded in shear and in tension perpendicular to the grain were conducted in Scandinavia by Blom and Bäcklund (1980), Enquist et al. (1991), Larsen et al. (1992), Botten (1993), Dahlbom et al. (1993), Hallström (1996), and Hallström and Grenestedt (1997) and were mostly related to curved beams and beams with large openings. Large openings in beams are often required, e.g. to let ventilation ducts through, but the shear and perpendicular-to-the-grain tension stresses that develop around these openings can lead to premature brittle failures of the beams. Also curved beams, due to their shape, develop stresses in the radial direction, i.e. in the direction perpendicular to the grain. Enquist et al. (1991) performed tests on curved and pitched cambered beams reinforced with externally-bonded GFRP. According to (Schober et al. 2015), the perpendicular-to-the-grain reinforcement led to a considerable increase in load-carrying capacity compared to unreinforced beams. Tensile failure of the GFRP and debonding are reported to have occurred during the tests. Reinforcement against tension perpendicular to the grain was apparently also achieved by using glued-in GFRP rods inserted in the radial direction, but the results were not reported. According to Hallström (1996) and Hallström and Grenestedt (1997), the methods prescribed at the time to reinforce timber beams were to glue steel bolts near the hole, to limit crack propagation and redistribute stresses, or to glue and nail wood-based panels around the hole. The reinforcement with internal glued-in rods can cause problems when moisture and temperature changes occur, e.g. inducing cracks due to differential shrinkage of timber and steel. The reinforcement with wood-based panels can have a negative impact on the aesthetics of the structure. Therefore, reinforcement with FRP with MOEs closer to that of timber and lower visual impact was seen as an interesting field. Blom and Bäcklund (1980) reinforced GLT beams with circular holes using AFRP fibres and an epoxy resin. The reinforcement is reported to be have been successful from a mechanical point of view, not from an aesthetical perspective. Larsen et al. (1992) testing curved and cambered GLT beams reinforced with GFRP glass fibre reinforcement were able to double the load-carrying capacity and change the failure mode of the beams. Dahlbom et al. (1993) continued the work and performed additional tests on GFRP-reinforced end-notched beams, triplicating their load-carrying capacity using GFRP reinforcement, and single-bolt connections, reducing the end-distance by 75% with no loss of load-carrying capacity. Botten (1993) conducted similar tests on bolts at mid-span of GLT beams reinforced with GFRP, reportedly being able to reduce the edge distance up to a third, but also running into bonding problems. Hallström (1996) performed tests on GLT beams with holes reinforced with EBR GFRP reinforcement around rectangular and circular holes (Figure 5.28), using a mostly transparent polyester resin. In this case, the author states that the GFRP reinforcement was effective at avoiding shear failures in GLT beams with holes. For beams with circular holes, a 1 mm layer of reinforcement on both sides of 90 mm-wide beams completely overcame the negative effects of the holes in the present load case. In beams with rectangular holes, it is reported that a change in failure mode was achieved and that the load-carrying capacity was improved between 140 and 190%.

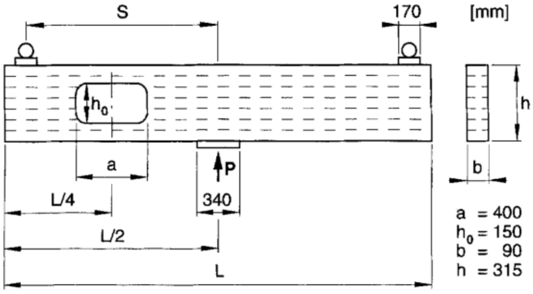
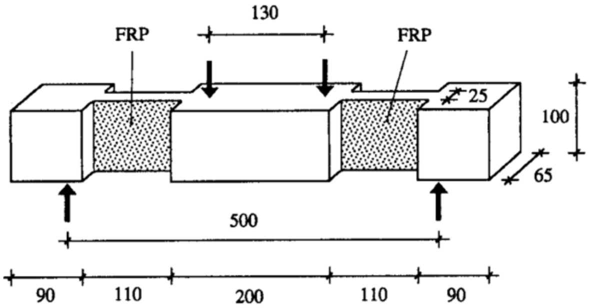


Figure 5.28. Configuration of GLT beams with rectangular holes tested by (Hallström 1996).

In Greece, (Triantafillou 1997, 1998) performed shear tests on small-scale GLT specimens reinforced with externally-mounted epoxy-bonded CFRP fabrics. The published force-displacement curves show an increase in the load-carrying capacity when the fibres of the reinforcement are aligned with the grain direction of timber and an increase in ductility when the fibres of the reinforcement are in the perpendicular direction (Figure 5.29). No effect of the reinforcement on the stiffness was observed.



Beam design (1)	CFRP (2)	t (mm) (3)	h_{frp}/h (4)
C	—	—	—
L1/1	one ply at 0°	0.167	1
L2/1	two plies at 0°	0.334	1
LV2/1	two plies at 0°/90°	0.334	1
V1/1	one ply at 90°	0.167	1
L1/0.6	one ply at 0°	0.167	0.6
L2/0.6	two plies at 0°	0.334	0.6

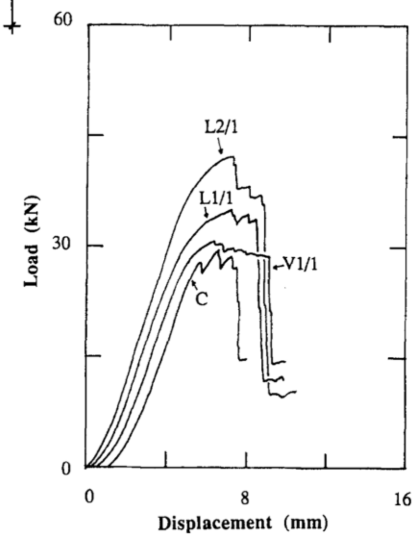


Figure 5.29. Geometry of the test specimens, tested configurations, and load-displacement curves presented in a study by (Triantafillou 1997).

In France, Coureau (2001) studied strengthening techniques for notched beams. Notches close to supports are critical zones, prone to brittle shear and perpendicular to the grain failures due to stress concentrations. The tests were performed on structural-size GLT beams (90×310×2575 mm^3), using the test set-up in Figure 5.30 and under force-control. The results showed that the externally-bonded GFRP strips increased the load-carrying capacity by 103 and 197%, for strips with widths of 43 and 85 mm, respectively. The author states that the increase in load-carrying capacity is due to the fact that the reinforcement "bridges" over the crack that is formed at the notch. Möhler and Mistler (1978) performed similar tests, but used nailed steel plates and nailed or glued wood-based panels instead of externally-bonded FRPs. The results show that the stiffer reinforcements, e.g. glued plywood panels, led to a higher increase in load-carrying capacity.

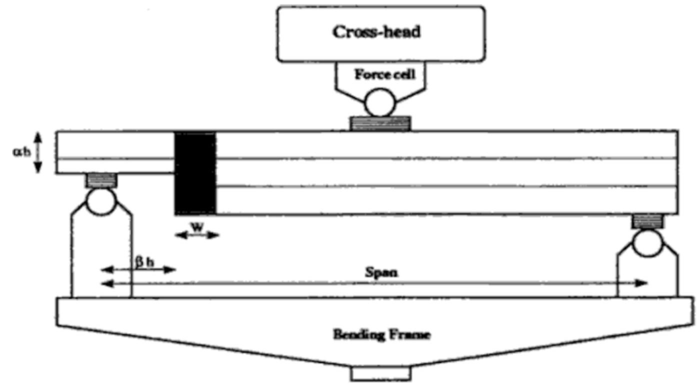


Figure 5.30. Bending/shear test set-up for notched beams (Coureau 2001).

In the U.S.A., Radford et al. (2000, 2002) studied the effect of vertical pultruded GFRP rods (which the authors call "shear spikes") in the shear behaviour of two stacked solid timber members, without any other form of physical attachment between them (Figure 5.31). It should be noted that the test specimens were not of structural size and that the tested configuration is not representative of most beams in service. However, this study was focused on reinforcing "span timbers" of timber bridges that were assumed to be damaged and mostly unable to carry significant shear forces. The authors tested steel nails inserted in the pre-drilled holes and glued-in pultruded rods (using an epoxy adhesive). Beams made from a single solid timber element were also tested. Regarding stiffness, the authors report that, as expected, the reinforcements inserted close to the supports (i.e. in positions R4, R5, and R6 in Figure 5.31) contributed more than the reinforcements closer to mid-span. Compared to the two stacked beams without any shear reinforcement, the glued-in GFRP rods gave a greater increase in initial linear elastic stiffness than the steel nails, but the adhesive might have spread between the two stacked beams. The authors report that gluing the two stacked beams together using the same epoxy used in the GFRP rods led to the same increase in stiffness as the glued-in rods. A second round of tests specimens was prepared with wax paper between the stacked beams and in this case the stiffness is reported to be "somewhat lower" than in the case without wax paper. Increasing the number of glued-in rods is also reported to have increased the stiffness. Regarding load-carrying capacity, given the very limited number of tests and the high variability that is reported, no major conclusions can be derived, other than that the single-element solid-timber beam and the two-element epoxy-glued beam seem to exhibit higher values than the beams with reinforcements.

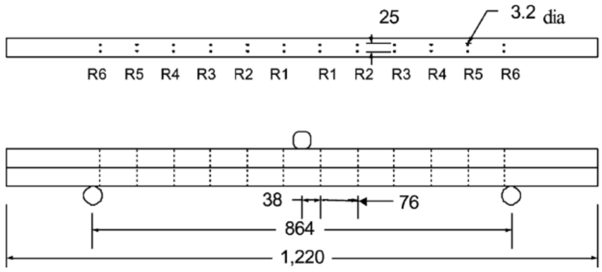


Figure 5.31. Specimen geometry for shear spiked 2×2 testing investigated by (Radford et al. 2002).

Still in the U.S.A., Kasal and Heiduschke (2004) studied the use of light GFRP composite tubes to strengthen small-scale curved GLT beams in the radial direction. Due to their shape, these structural members develop stresses in the radial direction, i.e. in the direction perpendicular to the grain. A few preliminary tests to compare the withdrawal strength of the glued-in FRP tubes and steel rods showed that the latter exhibited lower variability and somewhat higher values, but the authors argue that this "extra capacity may not be required in radial reinforcement". Regarding the four-point bending tests on the small-scale curved GLT beams (Figure 5.32), the results showed that the load-carrying capacity of the beams with glued-in GFRP tubes increased between 46 and 98% (only three tests were performed), compared to the unreinforced beams. Failure mode is reported to have changed from tension perpendicular to the grain at mid-span to lateral buckling and shear close to the supports.

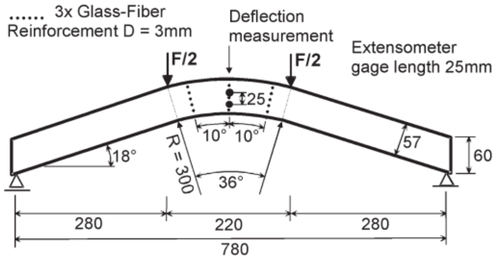


Figure 5.32. Schematic of the laminated arch model testing setup used by (Kasal and Heiduschke 2004).

Also in the U.S.A., Gentry (2011) made a feasibility study on "off-the-shelf" hybrid GLT-FRP beams, with heights of 200-400 mm, widths of 50-100 mm, and spans limited to 10 m. The author performed bending/shear tests on small-scale specimens and four-point bending tests also on small-scale specimens. In the small-scale tests (Figure 5.33a), the pultruded uni-directional GFRP smooth rods were glued using the same epoxy adhesive that had been used in the production of the GLT specimens. The author reports that, in these tests, only 50% of the unreinforced GLT specimens failed in shear and that the obtained shear strengths are much higher than the values obtained on similar material tested in accordance with ASTM D143 - 14 *Standard test methods for small clear specimens of timber*. The reported coefficient of variation is 18%, which is also quite high. The reinforced specimens reached slightly higher load-carrying capacities with a slightly lower variability, and exhibited slightly less shear failures. The authors attribute these small changes to the beneficial effect of the reinforcement. The use of GFRP rods with smaller diameter (4.6 mm, instead of 6.4 mm) led to an increase in load-carrying capacity, but, as the author notes, this might be because the smaller holes increase the net cross section of the timber laminations, since the number of GFRP rods was the same. Reducing the diameter even further (3.2 mm) caused the rods to fail in shear with the timber laminations. Increasing the number of rods, by reducing their spacing, is reported to have led to the highest increase in load-carrying capacity. However, since the author compares the performance of the various reinforcement schemes using the characteristic 5th percentile values of Weibull distributions fitted to the whole range of test results, the comparison of values from the tails of the distributions is questionable.

The small-scale four-point bending tests (1500 mm span, Figure 5.33b) were performed on different types of specimens:

- GLT beams made from timber laminations of strength grade no. 1 and no finger joints;

- GLT beams made from timber laminations of strength grade no. 2 (lower quality than grade no. 1), finger joints, and a longitudinal GFRP lamination in the tension zone;
- GLT beams made from timber laminations of strength grade no. 2, finger joints, longitudinal GFRP lamination in the tension zone, and the vertical glued-in GFRP rods;
- low-grade GLT beams, finger joints, longitudinal GFRP lamination in the tension zone, and the vertical glued-in GFRP rods.

Given the quite different nature of the test specimens, comparing the obtained results is not straightforward. Regarding load-carrying capacity, the strengthened specimens exhibited lower variability. The specimens with the longitudinal tension reinforcement but no shear reinforcement reached the same load-carrying capacity as the higher-grade GLT beams. This seems to show that the flexural strengthening was, in this case, able to compensate the use of lower quality lamellas. However, the specimens with shear reinforcement in addition to the longitudinal tension reinforcement exhibited a reduction of 11% of the load-carrying capacity. The authors report that this was because failures often initiated at the holes of the glued-in shear reinforcements. It could be argued that for structural-size beams, the reduction of net cross section would be smaller and higher load-carrying capacities could still be reached, but the results clearly show a possible unintended effect of this type of shear reinforcement‡. On the other hand, the authors report that the shear reinforcements were "able to hold the entire beam together" after delamination of the longitudinal reinforcement, towards the end of the tests.

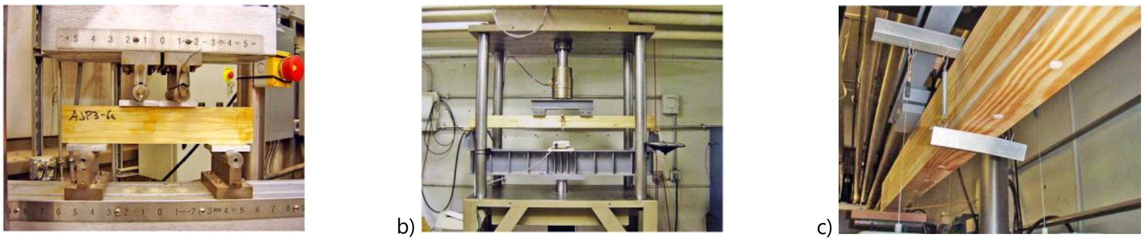
a)  b) c)

Figure 5.33. Tests performed by Gentry (2011): a) bending/shear test; b) four-point bending test; c) small-scale bending specimen with GFRP longitudinal and vertical reinforcements.

In Switzerland, Widmann et al. (2012) studied different types of shear reinforcement on short GLT beams (140×600×2500 mm^3). These GLT beams were of strength class GL 24h, but were produced with a defect, i.e. the middle timber lamination was glued only on 1/3 of its width (to reduce its shear load-carrying capacity). To prevent unwanted premature bending failures, the outermost timber laminations were made from high-strength larch (*larix decidua*) wood and, to prevent failures in compression perpendicular to the grain, the load application and support zones were reinforced with screws (except the notched beams). The types of reinforcement included self-tapping steel screws inserted at an angle of 45° to the grain (i.e. timber fibres) or, alternatively, externally bonded uni-directional CFRP sheets (glued using an epoxy adhesive and with the fibres at an angle of 45° to the grain (Figure 5.34). The beams were tested in three-point bending and the test programme comprised:

- testing the unreinforced beam (to assess the shear load-carrying capacity of the unreinforced beam);
- reinforcing the failed side of the beam;

‡ A similar outcome occurs when the beneficial effect of tension reinforcement in preventing bending failures is cancelled by shear failures induced at approximately the same load level.

- testing the beam again (to again assess the shear load-carrying capacity of the unreinforced beam, since failure occurs on the unreinforced side);
- reinforcing the other side of the beam;
- testing the beam a third time to finally assess the load-carrying capacity of the reinforcement.

This procedure allows evaluating the ability of the reinforcement to repair a failed beam, but also its ability to increase the load-carrying capacity of a beam with faulty glue lines or cracks. The test results showed that strengthening with externally-bonded CFRP sheets led to increases in load-carrying capacity of 60 and 77%, for a single CFRP sheet on each face and for two sheets on each face, respectively. However, since the beams reinforced at both ends did not fail in shear (but in compression perpendicular to the grain), the increase in the shear load-carrying capacity was higher than these values. The reinforcement with self-tapping screws led to increases in load-carrying capacity between 37 and 70%, depending on the number of screws that were used, which shows that, regarding load-carrying capacity, both strengthening techniques are equivalent.

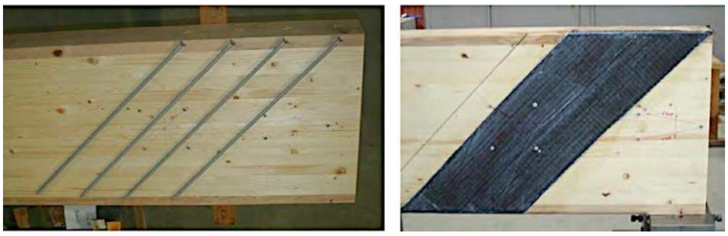


Figure 5.34. Reinforcement with self-tapping screws and with CFRP sheets investigated by (Widmann et al. 2012).

Also in Switzerland, Jockwer (2014) studied the reinforcement of notched beams, with a cross section of 90×315 mm^2, using self-tapping screws or externally-bonded CFRP (Figure 5.35). Compared to the unreinforced notches, the externally-bonded CFRP increased the load-carrying capacity between 180 and 230%, which is between what was reached by the reinforcement with internal self-tapping screws. The load-carrying capacity of the notches with externally-bonded CFRP was limited by debonding of the reinforcement, as in the tests performed by Enquist et al. (1991). The author reported that gluing the CFRP strips in the vertical direction (i.e. at 90° to the grain) led to failures in the lower part of the notch, due to the limited bonding length and limited resistance of timber in rolling shear. Gluing the CFRP strips at 45° led to failures in the upper part of the notch, because of the extensive deformations of timber under compression perpendicular to the grain in this zone and the limited deformability of the adhesive.

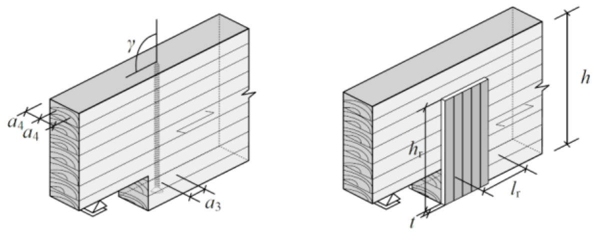


Figure 5.35. Notched beams with externally-bonded reinforcement studied by (Jockwer 2014).

Still in Switzerland, Blank (2018) observed that shear failures governed the behaviour of GLT beams with longitudinal tension reinforcement and studied the effect of adding additional shear reinforcement (Figure 5.36). The shear reinforcement was composed by glued-in steel rods inserted at an angle of 45°, which went through the longitudinal FRP reinforcement. The author reports that the formation and propagation of shear cracks could not be prevented, but that this "did not cause immediately a severe load drop or a significant shear dislocation in the specimens" and that "the threaded rods were pulled slowly through the specimens". However, it is very likely that this only occurred because the tests were performed under displacement control. Under force control, a brittle failure without any stress redistribution would most probably have occurred.

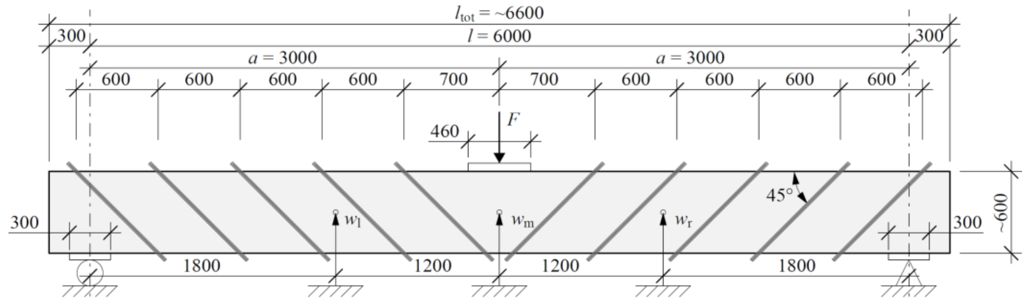


Figure 5.36. Set up of three-point bending tests on specimens with longitudinal and shear reinforcement (Blank 2018).

In Spain, Morales-Conde et al. (2015) tested small-scale solid timber specimens reinforced with an internally-bonded GFRP plate. These specimens were tested under unsymmetrical three-point bending, with the strengthened end under higher shear forces (Figure 5.37). The results showed that, compared to the unreinforced specimen, the strengthened specimens exhibited load-carrying capacities approximately 10% lower, except for the specimens strengthened with the longer GFRP plate. According to the authors, the specimens did not exhibit shear failures, but bending failures, therefore it is difficult to derive conclusions regarding reinforcement with bonded slotted-in FRP plates at structural scale.

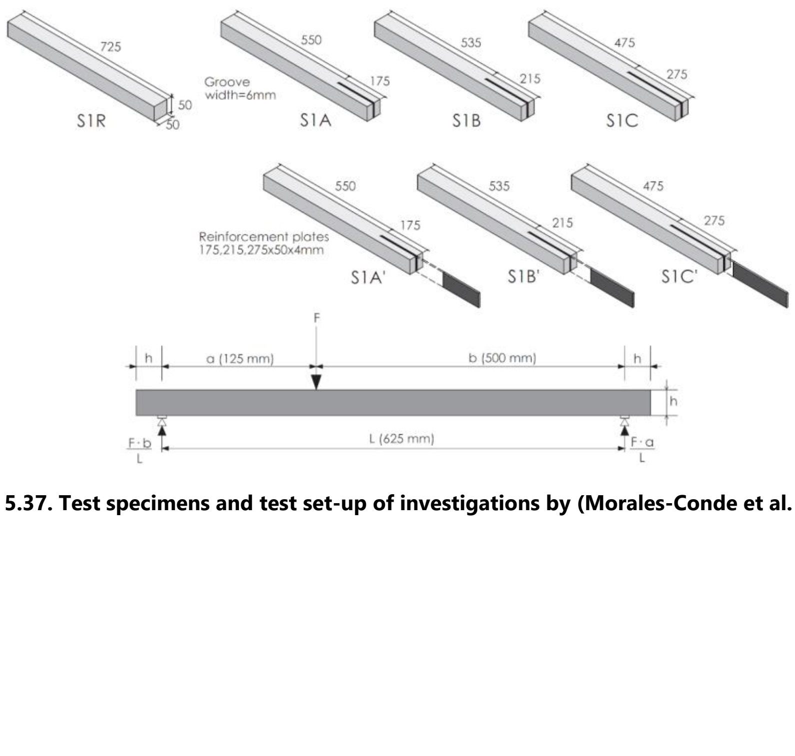


Figure 5.37. Test specimens and test set-up of investigations by (Morales-Conde et al. 2015).

5.1.2 Columns or members under compression parallel to the grain

Unlike the extensive research that has been conducted on the strengthening of timber beams with FRPs, research on the strengthening of timber columns has been remarkably rare. This is most likely related to the mechanical behaviour of structural timber in compression parallel to the grain, which is only moderately compromised by defects (i.e. exhibits less variability then the behaviour in tension and bending, Figure 5.38) and is much less prone to brittle failures. Therefore, the major concerns regarding the structural behaviour of timber members under compression parallel to the grain are the buckling-related aspects that might limit the load-carrying capacity. Buckling phenomena are prevalent in slender members (e.g. narrow columns), which are vulnerable to global buckling, or in members with slender cross sections (e.g. tubes), which are vulnerable to local buckling. Resistance to buckling can be more efficiently achieved by increasing the bending stiffness $E{\cdot}I$ of the cross section or of the most slender parts of the cross section. Another aspect that contributes to increase the buckling resistance is to increase the stiffness of the connections at the end of the member. Stiffer connections will reduce the lateral deformations and, therefore, reduce the second-order effects responsible for buckling failures.

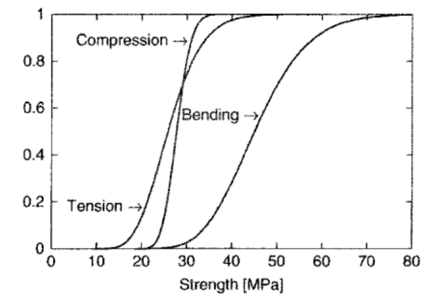


Figure 5.38. Cumulative distribution of the strength of structural timber for different loading modes (Thelandersson 2003).

In Japan, Tetsuro et al. (2004) and Tanaka et al. (2006) evaluated the buckling resistance of unstrengthened GLT, hybrid GLT-steel, and GLT-CFRP and columns. The GLT columns had a 105×105 mm^2 cross section composed by four Douglas-fir (*pseudotsuga menziesii*) timber laminations. The S400 6×65 mm^2 steel plates were fixed to the tension and compression faces using screws, whereas the 1.4×105 mm^2 CFRP sheets were bonded using a resorcinol resin (Figure 5.39). The results showed no differences between the stocky 1 m-long columns. For longer columns, the steel reinforcement led to higher increases in load carrying capacity (approximately 2.5 to 3 times higher) than the CFRP reinforcement (approximately 1.3 times higher). The authors report that failure of the columns strengthened with CFRP sheets was "more brittle than steel-reinforced columns" and was due to tensile failure of timber and debonding of the CFRP on the compression face.

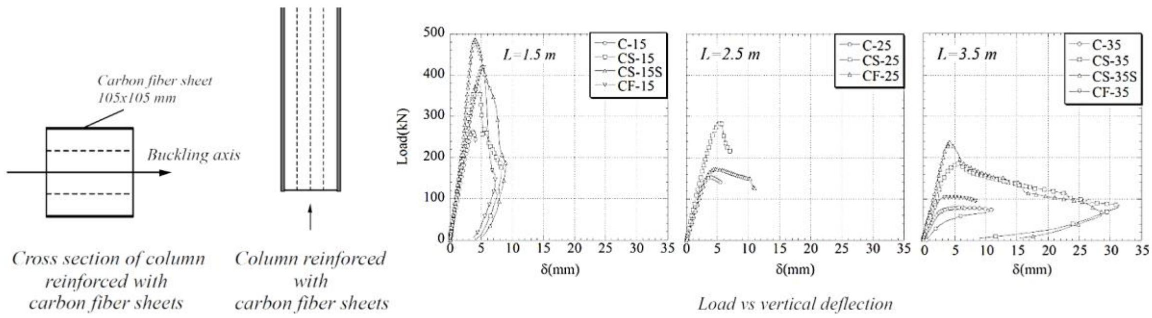


Figure 5.39. Geometry and vertical load-displacement curves of pinned-pinned hybrid GLT-steel (CS) and GLT-CFRP (CF) columns (adapted from Tanaka et al. (2006)).

Najm et al. (2007) studied the effect of different types of CFRP and confinement strategies on the behaviour of small-scale solid timber specimens (170 mm long and 57 mm in diameter), which the authors misleadingly call "timber columns", under compression parallel to the grain. As expected, the results showed an increase in compressive stiffness and load-carrying capacity (Figure 5.40) with increasing amount of wrapping. The reported strains were apparently based on displacements measured along the whole length of the specimens (instead of only in the central part), which tends to give higher stiffness values because of the additional confinement effect provided by the friction between the loading plates and the end surfaces of the specimens. Nevertheless, the reported results can hardly be extrapolated to structural timber columns.

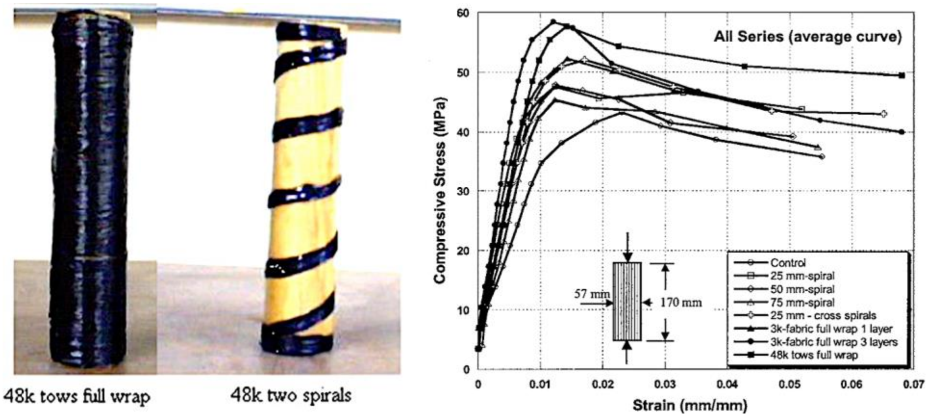


Figure 5.40. Typical column specimens and average tress–strain behaviour from a study by (Najm et al. 2007).

In Canada, Nagaraj (2005) and Taheri et al. (2009) studied the effect of different types of strengthening on the buckling behaviour of GTL columns. The authors performed tests on unreinforced, partially-strengthened, and fully-strengthened low-grade spruce GLT columns (Figure 5.41). The test specimens had lengths between 0.95 and 2.35 m and 110×110 mm^2 cross sections. The reinforcement was composed by GFRP or CFRP plates glued to the tension and compression faces of the buckling members (Figure 5.41). The 2.35 m-long specimens approach what could be considered a minimum size for common structural timber columns, but fixed-fixed boundary conditions would be difficult to implement in practice. Nevertheless, there are other structural members under compression (e.g. truss diagonals) for which the results of the tests on the pinned-pinned columns with lengths between 0.95 and 1.85 m can provide valuable insights. The authors concluded that strengthening with FRP could offer "incremental increase in the strength and stiffness of glulam columns" and favoured the partial strengthening over the central third of the column length as optimal regarding strength characteristics. Based on a preliminary cost analysis, the authors concluded that partially-strengthened GLT columns "could be considered as feasible" and that fully-strengthened columns may provide cost advantages over unstrengthened columns in the case of relatively deep cross-sections.

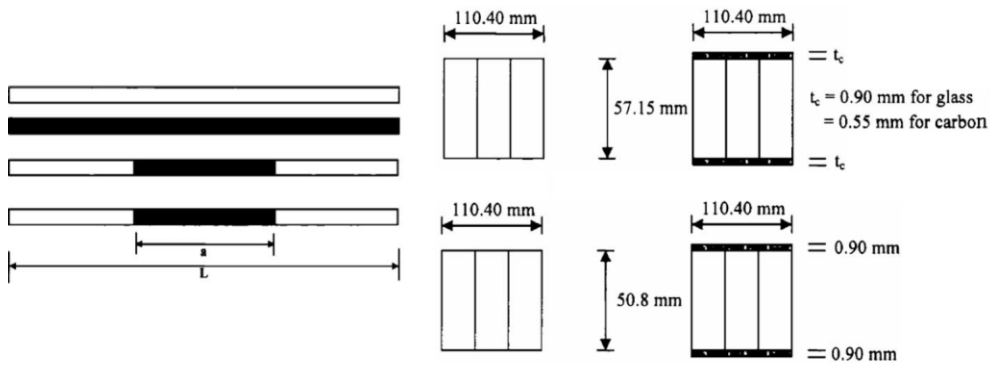


Figure 5.41. Strengthening schemes along the length of the column (unstrengthened, fully- strengthened, and partially-strengthened, i.e. length of the FRP laminates equal to a third of the column length and applied at the mid height) and in the cross section (unstrengthened and strengthened in the tension and compression faces) (Taheri et al. 2009).

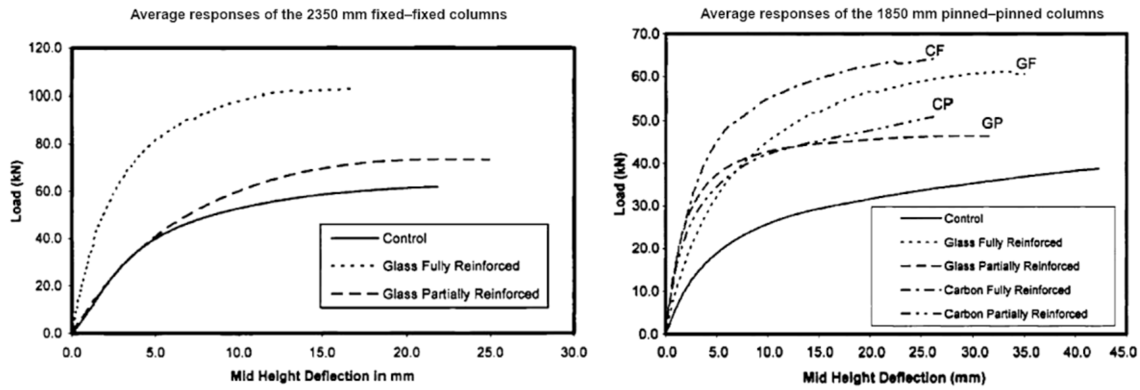


Figure 5.42. Average responses: 1.85 m pinned-pinned columns (left); 2.35 fixed-fixed columns (right) (Taheri et al. 2009). The vertical scale is different in the two graphs.

In Germany, namely at TU Dresden, Heiduschke and Haller (2010a; b) developed a light-weight hybrid timber-FRP structural element composed by a moulded timber-based tube (made from small-sized densified spruce wood elements) around which a FRP composite is wound (Figure 5.40a). The idea was that the timber tube could prevent local buckling of the thin-walled FRP composite and provide a permanent winding core and that, on the other hand, the FRP composite could significantly contribute to the improvement of the load-carrying capacity, stiffness, and durability of the timber tube (Figure 5.40b). These structural members were extensively tested in compression (Heiduschke and Haller 2010a; b), bending (Haller et al. 2013), and torsion (Haller et al. 2013), with GFRP and CFRP composites wound with different layouts and thicknesses. The results showed that the load-carrying capacity of the timber-only tubes could be significantly improved by a relatively small amount of reinforcement (around 5% of the tube thickness), due to the confinement provided by the FRP composite that strengthens timber in the direction perpendicular to the grain and allows reaching higher compressive strengths. The FRP composite does not seem to provide additional ductility (the tests were performed under displacement control and even so no significant ductility was observed), but seems to avoid the brittle failure of the timber tube into small fragments (Figure 5.40b). Haller et al. (2013) and Wehsener et al. (2013) showed an application of the tubes in a built-up shaft of a small wind turbine (Figure 5.40c). Longitudinal connections between tubes based on large finger joints led to reductions of approximately 10% in the load-carrying capacity, compared to continuous tubes. To connect the tubes to other structural elements, the authors recommend steel connectors and small-diameter dowel-type fasteners (Figure 5.40c). The use of GFRP composites has additional aesthetical advantages. More recently, Hartig et al. (2016) tested hybrid timber-FRP tubes made from beech (*fagus sylvatica)* wood, which has a significantly higher strength than Norway spruce (*picea abies)*, but lower dimensional stability, or densified beech wood, which can reach even higher strengths. The results again showed that a small amount of circumferential reinforcement working as confinement can suppress a complete simultaneous brittle failure of the whole tube, but does not introduce any significant ductility.

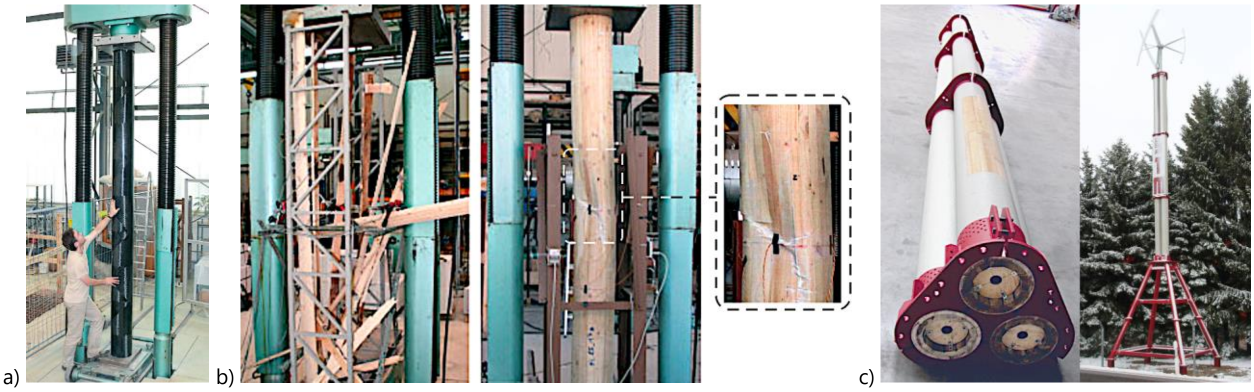


Figure 5.43. Hybrid timber-FRP tubes: a) buckling test of a 3.8 m-long column (adapted from Heiduschke and Haller (2010b)); b) failure of a timber-only tube (left) and a hybrid timber-FRP tube (right) (adapted from Heiduschke and Haller (2010b)); c) Shaft of moulded wooden tubes for wind energy plant (adapted from Haller et al. (2013)).

5.1.3 Other applications

5.1.3.1 Thin-walled structural members

In Australia, increased availability of low-grade hardwood has led to investigations on the development of higher-value end-uses for this material, namely through veneer-based products. The combinations of FRP with these veneer-based products was studied by Hansen et al. (2016), who focused on small-scale (i.e. less than 1 m) foldable elements, and later by Fernando et al. (2018), who focused on more traditional panel-type elements with moulded thin-walled sections. The amount of FRP compared to that of wood is much higher in these thin-walled members (almost the same amount of wood and GFRP, in terms of cross-sectional area, was used) than it is in the other examples presented so far. The authors tested the developed member in compression, assuming they could be used as wall elements, and reported that the hybrid FRP-timber member exhibited a load-carrying capacity that was two times that of a similar members without GFRP laminations, but both exhibited brittle failures.

5.1.3.2 Shear walls

In the USA, in the University of Maine, Cassidy (2002) and Cassidy et al. (2006) developed a hybrid GFRP-OSB panels for timber-frame shear walls. The hybrid-panel consisted of thin outer OSB panels with a woven GFRP tape sandwiched between the OSB panels at the edges. The objective of this local reinforcement was to increase the resistance and energy-dissipation capacity under cyclic loading of the nailed connection between the panel and the timber frame members. The results showed that the hybrid GFRP-OSB panels appear to have potential for increasing the energy dissipation and load-carrying capacity of light-frame shear walls, by improving the behaviour of the nailed panel-to-timber connections. However, taking full advantage of this potential is limited by other factors, such as limited withdrawal strength and failure under low-cycle fatigue of the nails, and requires stiffening and/or strengthening of other components of the shear wall. In addition, the edge reinforcement makes further cuts and adjustments to the panels very difficult, which can complicate the production of the walls, on site or in a workshop. Finally, no economic analysis was done comparing the developed walls with standard walls with better OSB panels and nails.

5.1.3.3 Timber boards

Chen (1999), Spaun (1981) applied fibreglass impregnated by a phenol-resorcinol formaldehyde (PRF) adhesive to reinforce finger joints and both bending and tensile strengths were reported to have increased from 10% to 40% in comparison to the unreinforced finger joints.

In France, Khelifa et al. (2015b) performed very few tests on finger-jointed spruce (*picea abies*) timber boards strengthened with CFRP. The boards were tested in four-point bending, with the finger joint at mid-span and the CFRP reinforcement externally bonded to the tension zone with an epoxy adhesive. The reinforcement led to increases in the bending resistance and stiffness of the finger-jointed boards, but still far from the performance of the continuous non-finger-jointed timber boards. The presence of reinforcement did not change the elastic-brittle failure mode of the finger-jointed boards. The study was made to calibre finite-element models (Khelifa et al. 2015a) and not to study a specific strengthening scheme.

In Switzerland, Fernando et al. (2016) produced elements composed by two timber boards glued together with a BFRP layer in between. The specimens comprised defect-free boards, boards with a hole drilled in the middle, boards with a hole and also knots, boards without hole but with knots, and different thicknesses of the BFRP layer. The wood species, the type of BFRP reinforcement, and the type of adhesive are not mentioned. The specimens were tested in tension. The results showed that the increase in strength and stiffness of strengthening with BFRP was only significant for specimens with timber boards with holes reducing the net cross section by 50%, i.e. for weaker timber sections. The presence of the BFRP-layer did not change the elastic-brittle failure mode of the boards, but allowed the boards to exhibit a second post-failure (i.e. after the timber boards failed) increase in load-carrying capacity, during which the applied load was transferred only by the BFRP layer. However, this only occurred after very significant displacements.

5.2 Connections

As described in Subsection 2.5 *Connections*, the most common types of connections in modern timber structures are connections with dowel-type fasteners in shear and slotted-in steel plates. For high-performance applications, connections with (GiRs) are also commonly employed. The use of other types of connections (e.g. nail plates, adhesively bonded connections, grouted connections, manufacturer-specific connectors) are mostly limited to very specific or niche applications (Schober and Tannert 2016; Vallée et al. 2017). The use of FRPs in timber connections did not lead until now to the development of new connections typologies, but is mostly limited to the replacement of existing components, usually made from steel, by components made from FRP composites or the use of FRP as strengthening against brittle failures in timber. The following sections deal mostly with the use of FRPs in connections with dowel-type fasteners and with GiRs.

5.2.1 Connections with dowel-type fasteners

The use of FRP composites in timber connections with dowel-type fasteners has followed two main approaches: (i) use components made from FRP composites as alternative to steel-based components (i.e. fasteners and slotted-in plates); (ii) reinforce the timber in the connection area through an externally bonded FRP layer. The first approach avoids major changes in the production and design procedures, even though parts made from FRP composites cannot be so easily machined and adapted as steel parts and design guidance is not always available. The load-carrying capacity of such connections is usually smaller than that of equivalent connections with steel parts, but in some cases they might be an appropriate solution (e.g. in chemically harsh environments, or when materials with non-magnetic and/or non-conductive properties are required). The second approach tries to improve the behaviour of timber in the connection zone, where it is loaded under very high localised stresses, also in the directions in which it exhibits a weak and brittle behaviour, but introduces additional complex steps to the productions of the timber members. It would also be difficult to implement in the common case of slotted-in steel plates, which are usually made by cutting the slots in a single timber members and not by side gluing two or more members together. Given these limitations, the most common form of local reinforcement is using steel self-tapping screws, which strengthen the timber members against brittle failures and increase the embedment strength (Bejtka 2005; Lathuillière et al. 2015), even though the additional steel parts might compromise the fire resistance of the reinforced connections (Palma et al. 2013, 2016a; Palma 2016).

5.2.1.1 Strengthening of timber members using FRP composites

The study of strengthening the timber members of timber connections with dowel-type fasteners took place during the 1990s.

In the early 90s, in Denmark and Sweden, (Enquist et al. 1991), Larsen et al. (1992, 1994), Dahlbom et al. (1993), and Larsen and Enquist (1996) studied the reinforcement of timber members against brittle perpendicular-to-the-grain failures using externally bonded GFRP composites. Dahlbom et al. (1993) and Larsen et al. (1994) showed that effective strengthening can be achieved by gluing glass fibres (either in the form of a mat with randomly orientated short fibres or as woven material, where most of the fibres may be oriented in a single direction) to the timber surface using a polyester adhesive/matrix. The glass fibre-polyester reinforcement has the advantage of being almost invisible because of transparency and small thickness. Hallström (1996) reported that Dahlbom et al. (1993) performed tests on GFRP-reinforced end-notched beams, triplicating their load-carrying capacity, and also on single-bolt connections, reducing the distance from a bolt to the end of a beam by 75% with no loss of load-carrying capacity. Larsen and Enquist (1996) studied the strengthening of nailed connections using unidirectional GFRP externally bonded with a polyester adhesive and showed that it increased ductility and, to a lesser extent, also the load-carrying capacity, when compared to un-reinforced connections. In addition, this strengthening could be used to reduce fastener spacing and edge and end distances (Gustafsson 2003).

In Switzerland and Germany, Chen and Haller (1994), Chen (1995, 1999), also studied the mechanical behaviour of connections with steel dowel-type fasteners and strengthened timber members. Chen (1999) strengthened the timber members by gluing different amounts of GFRP fabrics to both surfaces of the timber members in the shear planes, using an epoxy adhesive. He then performed embedment tests, under compression and tension, as well as shear tests and tension perpendicular to the grain tests on unstrengthened and strengthened timber specimens, followed by tests on steel-to-timber dowelled connections with outer steel plates. The results showed clear increases in strengths in all tests and also in ductility. The embedment, tensile, and shear strengths of hybrid GFRP-timber members could be estimated based on the volume fraction of GFRP (thickness of GFRP divided by the total thickness of the strengthened specimen). The load-carrying capacity of timber connections was estimated using the Eurocode 5 formulas and the modified embedment strength. A few years later, Haller and Birk (2006) investigated more complex tailor-made reinforcements (Figure 5.45, left), using loop-like structures made of glass, aramid, or carbon and biaxial knitted fabrics, glued with epoxy adhesive. The results showed very significant increases in embedment strength and ductility (Figure 5.45, right).

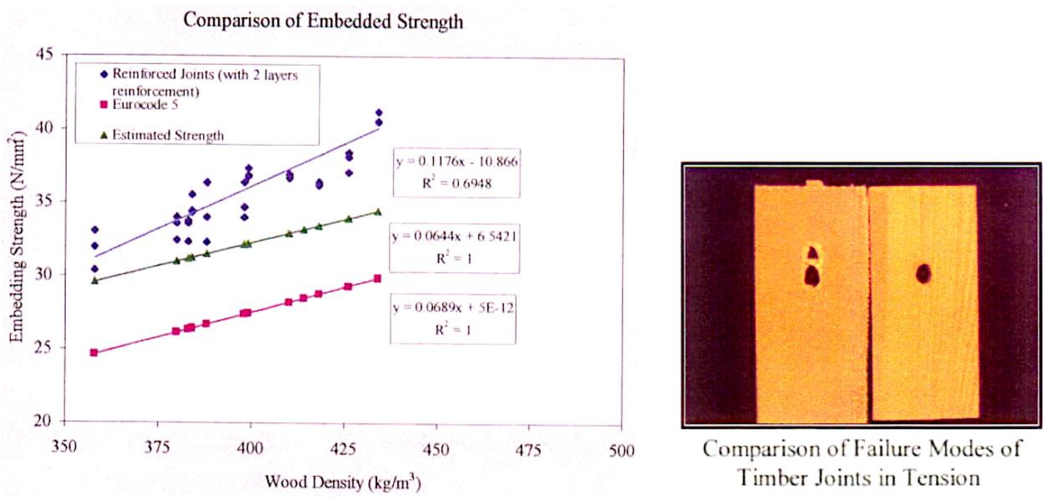


Figure 5.44. Effect of reinforcement and density on embedment strength (left) and embedment test in tension (right), showing the change in failure mode caused by the GFRP layer: the splitting failure of the unreinforced joint is replaced by a shear-out failure and the load-carrying capacity is increased by more than 40% (adapted from Chen (1999)).

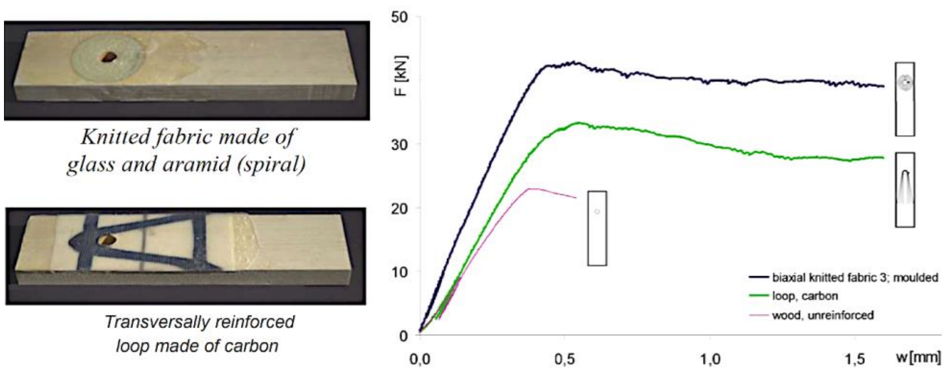


Figure 5.45. Strengthened timber members (left) and results of embedment tests (right) from a study by Haller and Birk (2006).

In the U.S.A., Windorski et al. (1997) and Soltis et al. (1998) studied the effect of strengthening the timber members of a dowelled connection with externally bonded GFRP layers, to avoid brittle failure modes in timber. The studied connections were single-fastener steel-to-timber connections with external steel plates, loaded in the directions parallel and perpendicular to the grain. The results showed an increase of the load-carrying capacity and ductility with increased number of GFRP layers. The largest incremental increase is reported to have occurred when adding the initial layer of GFRP to the non-strengthened connection, with additional layers increasing the resistance at smaller rates. For parallel-to-grain loading, the application of two GFRP layers is reported to have changed the failure mode of the timber members from brittle to ductile. For perpendicular-to-grain loading, no change in failure mode occurred with the strengthening of the timber members, but higher loads were reached (Figure 5.46).

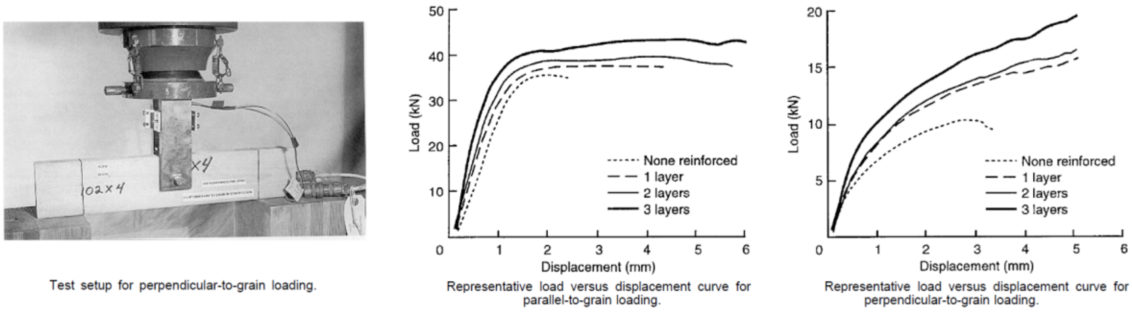


Figure 5.46. Test set-up for loading perpendicular-to-the-grain and load-displacement curves for loading parallel and perpendicular to the grain as reported in a study by Soltis et al. (1998).

More recently in South Korea, Kim et al. (2013a) studied the embedment behaviour of different layups of textile glass fibres and wood. The used textiles were of plain weaving type or of diagonal cloth type. The embedment tests were performed in tension, using fasteners with diameters of 12 or 16 mm, with an end distance of seven times the fastener diameter. The results showed increases in embedment strength of 10 to 20% for volume ratios of reinforcement between 1 and 2%.

5.2.1.2 Use of FRP-based components as alternative to steel components

The first studies on replacing steel parts by non-metallic parts started in the early 1980s, but were mostly focused on using timber-based parts, not FRP-based ones. In Switzerland, Gehri (1982) conducted an extensive study on timber trusses with steel-to-timber dowelled connections with multiple shear planes. In this study, exploratory tests were performed on timber-to-timber connections with internal plywood panels and hardwood dowels. The test report stated that these connections reached load-carrying capacities and ductilities adequate for structural purposes, due to the deformations in the wooden dowels. Their load-carrying capacity was approximately 2/3 of the load-carrying capacity of the trusses with internal steel plates and steel dowels, and collapse was due to failure of the plywood panels. In Germany, Epple (1982) also studied replacing the steel components with wood-based components (densified wood and synthetic resin composition boards).

Steiger (2014) reports on a study performed by Timmermann and Meierhofer (1994) and Meierhofer (1994, 1999) on splice connections with glued slotted-in FRP plates (Figure 5.47). Since the strength of the FRP laminates was actually much higher than the transmittable shear force, GFRP was used instead of the more expensive CFRP laminates. An epoxy-type adhesive was used to glue the plates. Different connection configurations were tested and it was observed that the load carrying capacity increased with the number of glued slotted-in plates, until the tensile load-carrying capacity of the reduced timber cross section became the limiting factor. Bending tests on the connections showed a remarkable bending stiffness and load-carrying capacity in bending, almost comparable to those of an unspliced member.

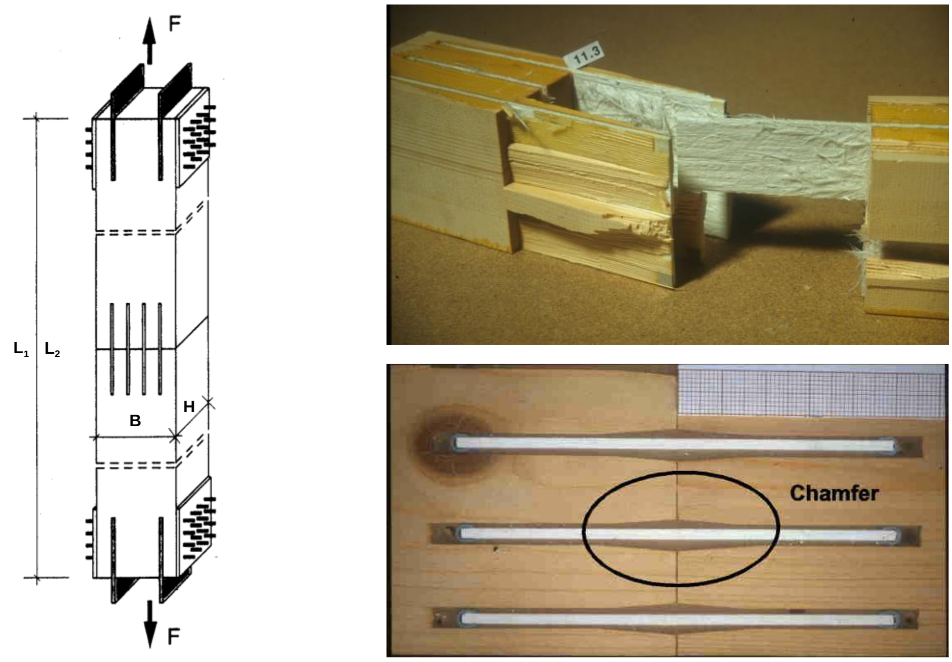


Figure 5.47. Tensile tests on glued-laminated GFRP-spliced specimens in structural sizes (stress concentration was reduced by enlarging the width of the groove near the end of the member; this increased the strength of the connection by as much as 25%) (Steiger 2014).

In Canada, Erki (1995) studied the behaviour of bolted FRP-to-FRP connections, without timber members, to compare the performance of threaded steel bolts and pultruded threaded GFRP bolts. The tested connections were single-fastener three-member connections, with side and internal members made from pultruded GFRP sheet, 13 and 25 mm thick, respectively. The results showed that steel bolts led to higher load-carrying capacities but failures of the GFRP sheets, whereas GFRP bolts failed with little damage to the jointed GFRP sheets. The threads in the GFRP bolts were mentioned as particularly week. The GFRP sheets were strongly unidirectional (composed by alternating layers of uni-directional fibres and a randomly orientated continuous strand mat) and therefore their behaviour was strongly dependent on their orientation.

In the UK, Drake et al. (1996) performed embedment tests on LVL using steel and GFRP dowels. The tests showed that the GFRP dowels did not remain straight during the tests, as the steel did, and also failed, unlike the steel dowels, which makes comparing the results difficult. Nevertheless, unlike the steel dowels that loaded the entire length of wood approximately equally finally leading to a splitting failure, the GFRP dowels bent and crushed the outer layers of LVL first. The measured maximum loads were marginally higher for the GFRP dowels than for the steel dowels, but that might be due the fact that GFRP fasteners deform and are then also loaded in tension. Therefore, it is difficult to draw conclusions regarding the embedment behaviour of GFRP dowels and regarding comparing FRP and steel dowels. A few years later, Drake et al. (1998, 1999) presented more results, including embedment-type tests, but with multiple fasteners. The results showed that, in comparison to steel fasteners, the load-carrying capacity was not reduced by the presence of multiple GFRP fasteners in a row, which was attributed to the reduced stiffness of the GFRP fasteners, that allowed for a better load sharing between the dowels. Moment-resisting connections with GFRP dowels and plates are reported to be more ductile and reach similar load-carrying capacities as steel dowelled connections. More recently, Thomson (2010) and Thomson et al. (2010a; b) also studied the feasibility of non-metallic dowelled timber connections with slotted-in steel plates. After some exploratory studies, the authors focused on replacing the steel dowels with GFRP dowels and the steel plates with panels of densified veneer wood (DVW) (Figure 5.48). However, glass fibres and polyester resin exhibit a poor performance at high temperatures (Brandon et al. 2013, 2015; Brandon 2015) and there are other composites with a better mechanical performance that might be more adequate. The results showed that a connection with GFRP dowels and a slotted-in DVW plate, reached 50-60% of the load-carrying capacity of an equivalent metallic connection, while also reaching similar levels of ductility. Given the lower stiffness of the GFRP dowels, spacing between fasteners could be reduced, compared to what is specified for steel dowels.

Figure 5.48. Failed timber-to-timber and panel-to-timber (with GFRP dowels) connections in tests performed by Thomson (2010).

In Switzerland, Palma et al. (2016b) also studied non-metallic connections with FRP-based dowels and slotted-in DVW plates made of beech (*fagus sylvatica*). In this study, various types of FRP-based dowels were tested and more direct comparisons to equivalent connections with metal parts (including reinforcement with screws) were made. Fire exposure tests were also performed. The results showed that dowels made from glass fibres and an epoxy matrix behaved significantly better than cheaper dowels with glass fibres and a polyester matrix and even reaches higher load-carrying capacities than expensive dowels with carbon fibres and an epoxy matrix (Figure 5.49). The tests on dowelled connections with a slotted-in DVW plate showed that the connections with steel dowels reached higher load-carrying capacities, but exhibited lower levels of ductility (Figure 5.50, left). Nevertheless, a fairer comparison would be if the steel dowels were of lower strength and had failed in failure mode III (i.e. with three plastic hinges). The connections with GFRP dowels and a DVW slotted-on plate reached about 60% of the load-carrying capacity of a similar sized connection with steel dowels and slotted-in plate, but exhibited low levels of ductility (Figure 5.50, right).

Reinforcement of the connection with steel parts with self-tapping screws significantly increased the load-carrying capacity and led to levels of ductility in the range of those exhibited by the connections with GFRP dowels and a DVW slotted-on plate, showing the great potential of this type of reinforcement. Fire exposure tests showed that the steel parts conduct more heat into the cross section, leading to higher charring and eventually to a faster loss of load-carrying capacity in fire than the connections with non-metallic parts (Figure 5.51). Design rules based on the Johansen-Larsen approach (Johansen 1949; Larsen 1973) were also developed, assuming that nominal plastic hinges were formed in the GFRP fasteners, which makes designing these connections based on EN 1995-1-1:2004 straightforward. The design models also showed that, for the fastener spacings prescribed in EN 1995-1-1:2004, no reduction of the load-carrying capacity to take into account the effective number of fasteners has to be considered.

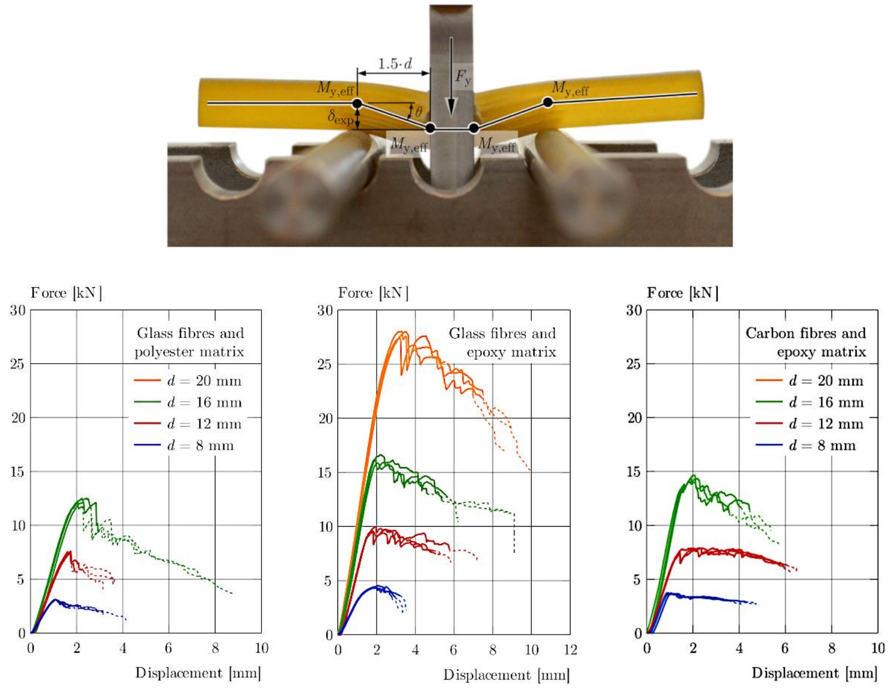


Figure 5.49. Test set-up (above) and force-displacement curves of the "apparent horizontal shear strength tests" on FRP rods (below) (Palma et al. (2016b)).

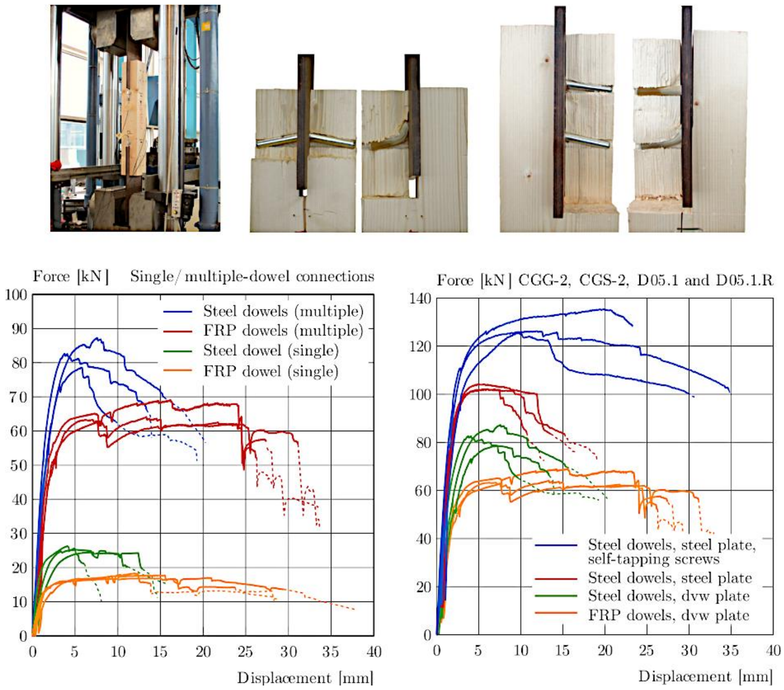


Figure 5.50. Failed DVW-to-timber connections (with steel and GFRP dowels, above) and force-displacement curves of the connection tests (including connections with steel and FRP parts, below) (Palma et al. 2016b).

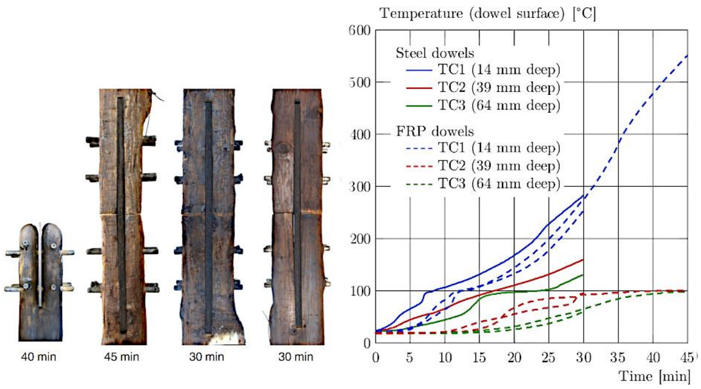


Figure 5.51. Steel-to-timber and DVW-to-timber connections after fire exposure (left) and fastener temperatures at various depths from the fire-exposed surface (right) (Palma et al. 2016b).

5.2.2 Glued-in rods (GiRs)

The most commonly used material for connections with glued-in rods (GiRs) is steel, because it allows designing the connections to exhibit a ductile failure mode (yielding of steel in tension or bending). Most used steel rods have metric threads or at least ribbed surfaces, which increase the adhesion area and mechanical interlocking. Metric threads in addition, allow easy assembling with other steel components. Less common is the use of FRP rods, even though they can also perform well in terms of ease of manufacturing, light weight, and statically efficient connections (Tlustochowicz et al. 2010). FRP rods are significantly stronger in tension than steel rods (and have much higher strength-to-weight ratios), but both BFRP and GFRP rods have lower modulus of elasticity than steel and should, therefore, be more compatible with most timbers. BFRP and GFRP rods are cost-effective options, but BFRP has higher tensile strength and slightly better corrosion resistance than equivalent GFRP rods. Unlike with steel GiRs, in which failure should be by yielding of the rods, FRP rods are brittle and failures should occur in the timber close to the glue-timber interface (Steiger et al. 2015). This requires the use of adhesives with good viscosity and gap-filling properties, such as EP or PFA, that the timber is freshly drilled and cleaned, and the FRP abraded and wiped down with a solvent (Steiger et al. 2015). This is because FRP rods tend to have a smooth or only slightly textured surface, relying more on actual adhesion than on mechanical interlock (Mettem et al. 1999). The cost of using FRP rods is higher than using steel rods and they cannot be easily processed (e.g. cut, or threaded) or connected to other structural members. The main failure modes to consider in connections with FRP GiRs are: failure of the rod itself, which is brittle, unlike with steel rods; localized timber failures around the rod (e.g. shear in the direction parallel to the grain); failure in the adhesive (also called "cohesive failure"); failures in the adhesive-timber and adhesive-FRP interfaces (Vallée et al. 2017).

The first studies on the use of FRP GiRs were conducted in Germany by Müller and von Roth (1991), who compared the withdrawal strength of steel threaded rods, steel reinforcement bars for concrete, and GFRP (with polyester resin) rods. The results showed that FRP rods failed mostly in the rod-adhesive interface and that the withdrawal load-carrying capacities were lower than those of GiRs made of steel, but all three types of rods were adequate for structural applications.

In the U.K., simultaneously with research on FRP dowels, studies were being conducted on using glued-in FRP rods by Mettem et al. (1999), Harvey et al. (2000), and Harvey and Ansell (2000). These studies showed the great performance variability between supposedly similar adhesives from different producers and the massive variability of the FRP mechanical properties, depending on the precision in terms of composition and manufacture. The results also showed that the withdrawal capacity tended to increase linearly with the bondline thickness and a minimum bondline thickness of 2 mm was recommended. FRP materials appear to have a useful role for the execution of bonded-in rod type connections. Perpendicular and parallel-to-the-grain tests on GiRs produced similar pull-out failure loads (Figure 5.52). Tests on moment resisting connections showed brittle failures with withdrawal of the dowel, except if the timber was locally loaded in compression perpendicular to the grain in another zone of the connection before withdrawal of the GiRs (as in a beam-to-column connection). Therefore, as Mettem et al. (1999) state in their conclusions, FRP rods "require different design considerations and should not necessarily be considered as direct substitutes for steel rods". Development in the U.K. continued mostly through the work of Madhoushi and Ansell (2004, 2008a; b), who studied GiRs under fatigue loading. The results showed that to reach 10^6 cycles, moment-resisting connections with GiRs needed to keep the load level at approximately 30% of the static load-carrying capacity.

More recently in the UK, Toumpanaki and Ramage (2018) compared the pull-out behaviour of GFRP and CFRP GiRs. The GiRs had a diameter of 10 mm and were glued in the direction parallel to the grain in 70×70×55 mm^3 spruce (*Picea abies*) solid timber blocks, cut from solid timber elements of strength grade C 24. The bond length was always 50 mm, i.e. five times the diameter, but different glue-line thicknesses were evaluated. The pull-out tests were performed in a pull-compression configuration (Figure 4.3, top left) and under displacement control (both the static and the cyclic tests). Toumpanaki and Ramage (2018) reported that most failures occurred in the timber-adhesive interface. The CFRP GiRs exhibited slightly higher pull-out forces, but overall the bond strength (pull-out force divided by the surface area of the hole) was approximately the same for all configurations. In the cyclic tests, the CFRP rods exhibited a slight reduction of the pull-out forces, compared to the static tests, whereas the GFRP GiRs exhibited a slight increase, but given the variability of the test results, no conclusions regarding resistance under static and cyclic loading can be made. The GFRP GiRs exhibited higher stiffnesses under load levels between 10 and 40% of the failure load, which corresponds to a load level often associated with serviceability limit states.

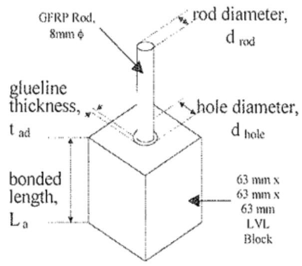


Figure 5.52. Single-ended pull-out test specimen (Mettem et al. 1999).

In Italy, Micelli et al. (2005) studied the use of CFRP GiRs as longitudinal reinforcement for moment-resisting end-grain connections (Figure 5.53). The tested timber members were made of Norway spruce (*picea abies*) GLT and had a cross section with 120×200 mm^2. The CFRP rods had a diameter of 13 mm. The connection was tested in four-point bending and the specimens exhibited bending stiffnesses similar to that of the continuous beams. The load-carrying capacity is reported to have increased with the anchorage length of the rod, reaching up to 90% of the load-carrying capacity of the continuous beams.

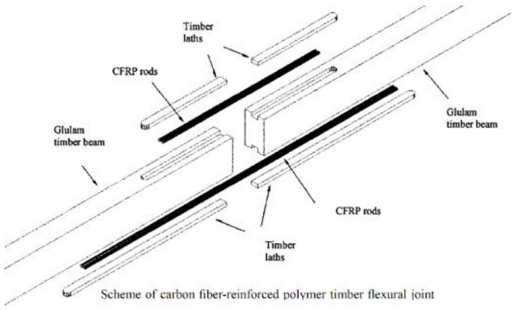


Figure 5.53. Connection with GiRs inserted in longitudinal grooves investigated by Micelli et al. (2005).

In Ireland, O'Neill et al. (2014) made pullout-bending tests on BFRP rods. Unlike in pull-pull withdrawal tests of GiRs, pullout-bending induces bending of the FRP rod, usually reducing the withdrawal capacity. These tests might be more appropriate if the rod is expected to experience bending under loading, which is seldom the case. The FRP rods had diameters of 12 mm and were glued with an epoxy adhesive. The results showed a clear increase in withdrawal capacity with increasing bonded length (Figure 5.54).

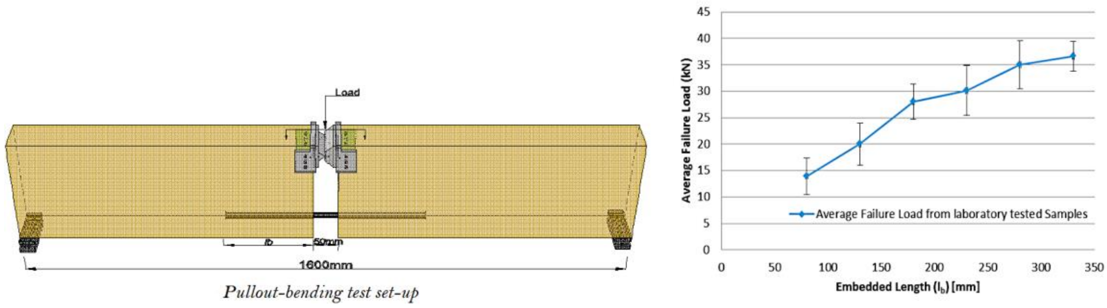


Figure 5.54. Test setup and results of pullout-bending tests performed by O'Neill et al. (2014).

In Canada, Zhu (2014) made fatigue pull-pull tests on GFRP glued with a PUR adhesive to Douglas-fir (*pseudotsuga menziesii*) glulam. The results showed that the fatigue resistance of the GiRs was substantially reduced when the fatigue loading passed 65% of the static load-carrying capacity and a relationship between fatigue resistance and anchorage length could be determined.

In South Korea, Song et al. (2018) tested moment-resisting connections with GFRP rods in larch (*larix*) GLT. Preliminary withdrawal tests allowed optimising the bond lend to six time the rod diameter and the bondline thickness to 3 mm. The connections exhibited an elastic behaviour followed by brittle failures.

5.3 Patented products and production methods

Various hybrid timber-FRP products, both structural and non-structural, have been patented at least since the early 1990s. The interest in strengthening with FRP composites has died out since the early 2000s, not only for timber but also for steel and concrete structures, and given the lack of significant economic success of such products in the construction industry, except for some niche applications, no significant new products or production methods have been patented since then. It is not clear if some of the patents would stand if challenged, e.g. adding longitudinal reinforcement during production of a GLT member.

Some of the most interesting patents, in chronological order, are:

- "Devices for load carrying structures", by Engebretsen (1991), outlining the idea of inducing a permanent stress state in the reinforcement by applying it while the beam is deformed;
- "Reinforced laminated timber", by Gardner and Eaton (1991), regarding the use of steel rebars in longitudinal grooves, between timber laminations, in the tension and compression zones of GLT beams;
- "Method of manufacturing glue-laminated wood structural member with synthetic fiber reinforcement", by Tingley (1995), presenting a method of producing strengthened GLT beams that consists of fixing the FRP reinforcement to the timber laminations before production of the GLT beam;
- "Bolted wood connections", Soltis and Ross (1996), regarding bonding a reinforcing material to the timber surface around a fastener hole to provide local reinforcement;
- "Use of synthetic fibers in a glueline to increase resistance to sag in wood and wood composite structures", by Tingley (1998b), related to adding discontinuous or continuous fibres to the glue lines of GLT beams during production, to improve the behaviour of the glue line;

- "Reinforced wood structural member", by Tingley (1999a), on FRP laminations with an cellulose-based cover applied during production of the laminate, which would make it easier to glue the FRP lamination to a timber substrate, using common adhesives for GLT production;
- "Wood I-beam with synthetic fiber reinforcement", by Tingley (1999b), regarding the reinforcement of the timber flanges of built-up I-beams with FRPS;
- "Reinforced composite wooden structural member and associated method", by Covelli et al. (2000), regarding the production of reinforced GLT members using previously prepared reinforced timber laminations, in which the longitudinal reinforcement is installed in longitudinal grooves, therefore separating the reinforcement process from the GLT production process;
- "Prestressed wood composite laminate", by Karisallen and Tynes (2000), regarding the production of a pre-stressed timber composite lamination, comprising alternate thin timber laminations and pre-stressed FRP laminations, that can then be introduced during normal production of GLT beams;
- "Wood composite panels for disaster-resistant construction", by Dagher and Davids (2004), regarding a timber-based panel with externally bonded FRP-based reinforcement, applied along the perimeter of the panel, at its corners, or where stress concentrations are likely to occur, namely where fasteners are driven through the panel.

6 Reinforcement of structural timber elements using FRP composites

This Section deals with reinforcement/repair of existing timber structures or structural timber members using FRP composites, i.e. mostly externally-bonded (EBR) or near-surface-mounted (NSMR) FRP composites. In some cases there might be an overlap with the previous Section on *hybrid elements*, but the current Section focuses mostly on studies conducted on existing structures, or members taken from existing structures, with the objective of developing techniques to be applied on site.

6.1 Members

6.1.1 Bending

In Canada, Gentile (2000) and Gentile et al. (2002) studied the bending behaviour of Douglas Fir (*pseudotsuga menziesii*) solid timber beams, taken from an old bridge that had been in service for over 40 years, reinforced with GFRP bars. The solid timber beams had 200×600 mm^2 cross sections and the GFRP rods were installed in grooves cut on the tension face or in the side face, 50 mm above the bottom (Figure 6.1). Only the central 6.0 m of each beam were reinforced and the GFRP rods were glued using an epoxy adhesive. The beams were tested under four-point bending. The results showed almost no influence of the reinforcement on the bending stiffness, but a significant increase in the load-carrying capacity. Since only one specimen was tested for each reinforcement scheme, the results should be looked at with caution. One of the reinforced beams ("Beam FS-1" in Figure 6.1) exhibited a brittle failure and the other two exhibited significant plastic deformations, due to failure of timber in compression parallel to the grain. However, as previously mentioned, it is not clear whether this is a result of applying the load under displacement-controlled loading regime during the tests. It is worth noting that in this study the beams were taken to the lab where the reinforcement was installed under controlled conditions. Nevertheless, the proposed reinforcement technique was used to reinforce 75 Douglas fir creosote-treated beams in a bridge in Canada. The reinforcement was applied on site, without interrupting the traffic on the bridge. To ensure an adequate curing of the epoxy, the bridge was covered and continuously heated. A loading test was performed before the reinforcement of the beams and a loading test was planned to be conducted afterwards, but its results were not reported. Still at the University of Manitoba, Amy and Svecova (2004) continued studying the reinforcement of creosote-treated Douglas fir timber beams salvaged from timber bridges. In this study, the beams had small notches close to the supports and were reinforced with bending (longitudinal GFRP rods) and shear reinforcement (inclined GFRP glued-in rods), both glued with an epoxy adhesive. The results showed that the bending reinforcement did not increase the average load carrying capacity of the beams, which was attributed to shear failures, and to the fact that the unreinforced beams were apparently of a higher grade. The introduction of the shear reinforcement in addition to the bending reinforcement finally led to an increase of approximately 20% in the load-carrying capacity. The presence of reinforcement did not introduce any relevant increase in ductility and did not prevent brittle failure modes.

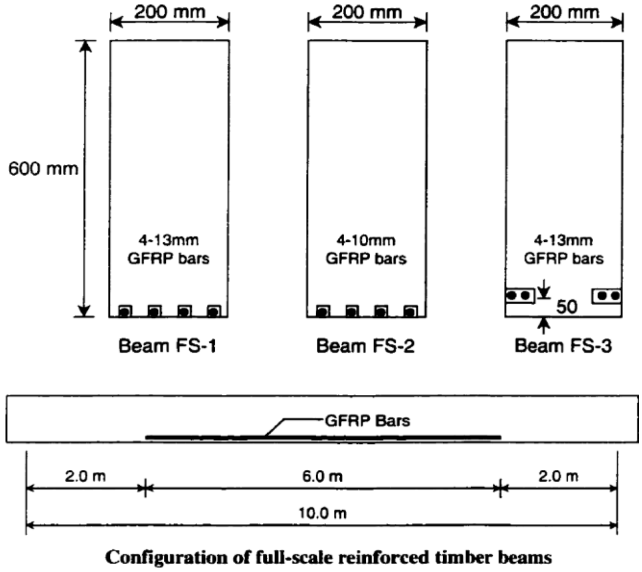


Figure 6.1. Douglas Fir solid timber beams, taken from an old bridge that had been in service for over 40 years, reinforced by GFRP bars (Gentile (2000)).

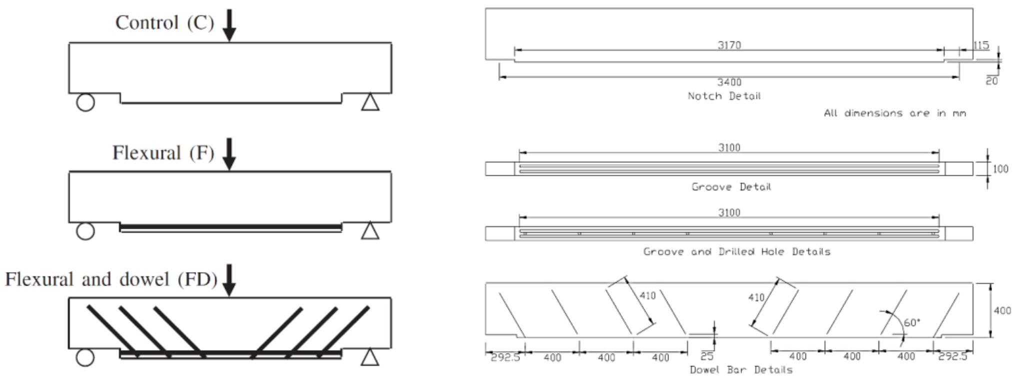


Figure 6.2. Reinforcement of notched solid timber beams (Amy and Svecova (2004)).

In the U.S.A., Buell and Saadatmanesh (2005) studied the effect of wrapping creosote-treated Douglas fir solid-timber beams with bi-directional CFRP fabric (Figure 6.3). The beams were recovered from a decommissioned bridge and then reinforced and tested in the lab. The CFRP fabric was glued with an epoxy adhesive. Four-point bending and three-point shear tests were performed. The results showed that wrapping with CFRP fabric increased the bending stiffness by around 20%, bending load-carrying capacity by approximately 45%, and shear load-carrying capacity increased between 40 and 70%. The reinforcement did not, however, change the brittle failure modes. The beams wrapped using a single piece of fabric along the whole length performed better than the beams wrapped with several overlapping wraps.

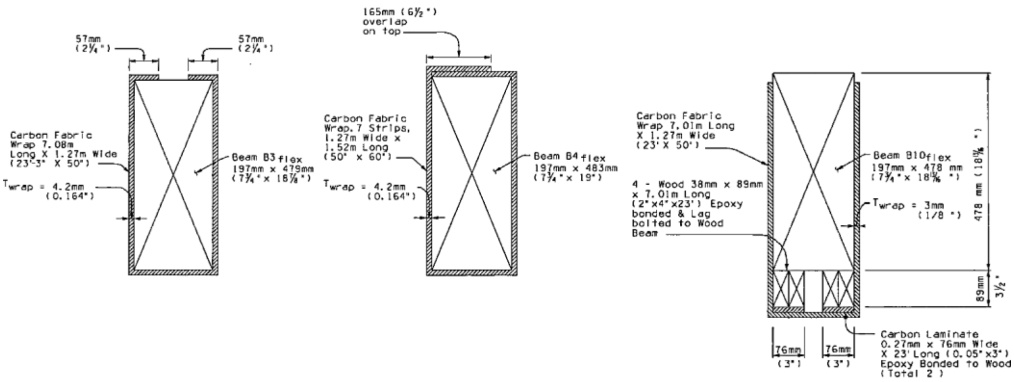


Figure 6.3. Wrapping schemes (Buell and Saadatmanesh (2005)).

In Germany, Schober and Rautenstrauch (2007) studied different reinforcement techniques for timber beams in existing timber floors under bending loads. Solid timber spruce (*picea abies*) beams recovered from 100-year old floors and ceilings of a residential house were reinforced and tested in the lab. The beams had an average cross-sectional area of 18×17 cm^2 and were reinforced with surface-bonded CFRP laminations or near-surface-bonded laminations (Figure 6.4), glued with an epoxy resin. The beams were tested under four-point bending. The results showed an increase in elastic bending stiffness (the elastic stiffness of the unreinforced beams was experimentally determined before their reinforcement). Given the small number of replicas, the high variability of the results, and the lack of tests on unreinforced beams from the same source, it is difficult to compare the performance of the various reinforcement schemes. However, it is worth noting that 7 of the 12 test specimens exhibited shear failures and 6 of the 12 specimens exhibited failures in the CFRP reinforcement. The reinforcement also did not consistently improved ductility in a significant amount.

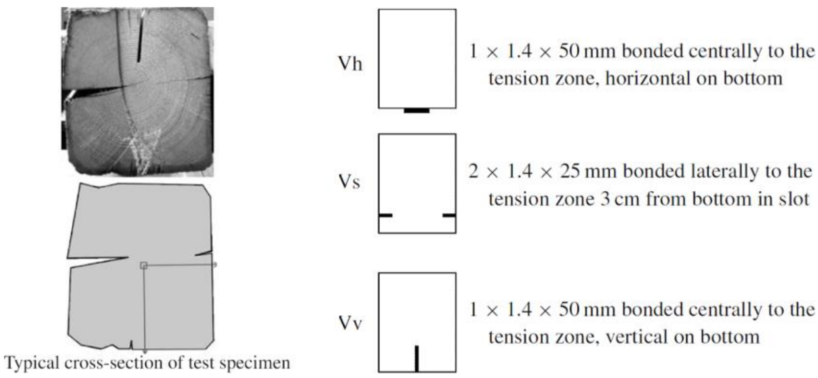


Figure 6.4. Reinforcement schemes investigated by Schober and Rautenstrauch (2007).

In Poland, Nowak et al. (2013) tested 100-year old solid timber ceiling beams reinforced with CFRP laminates. The 4000 mm-long timber beams were made of Scots pine (*Pinus sylvestris* L.) and had a 120×220 mm^2 cross section. Longitudinal groves were cut on the side faces of the beams to simulate different types of damage and the CFRP laminates were introduced in slots cut from the top of the beams, to simulate on-site conditions in which the underside of the beams could not be accessed. The beams were tested under 4-point bending and the tests performed under force-control, however the reported loading rate would lead to failures being reached in 25 to 35 min. In addition, the presented force-displacement curves exhibit drops in the applied load, which is inconsistent with a force-controlled test. The results showed increases in the bending load-carrying capacity between 20 and 80% and increases in stiffness between 10 and 30%, compared to the unreinforced beams. However, for the same reinforcement scheme the results have very significant variations (up to almost 50%). The authors mention that the beams exhibited brittle failures in timber and no failures were observed in the reinforcement or in the timber-reinforcement interface.

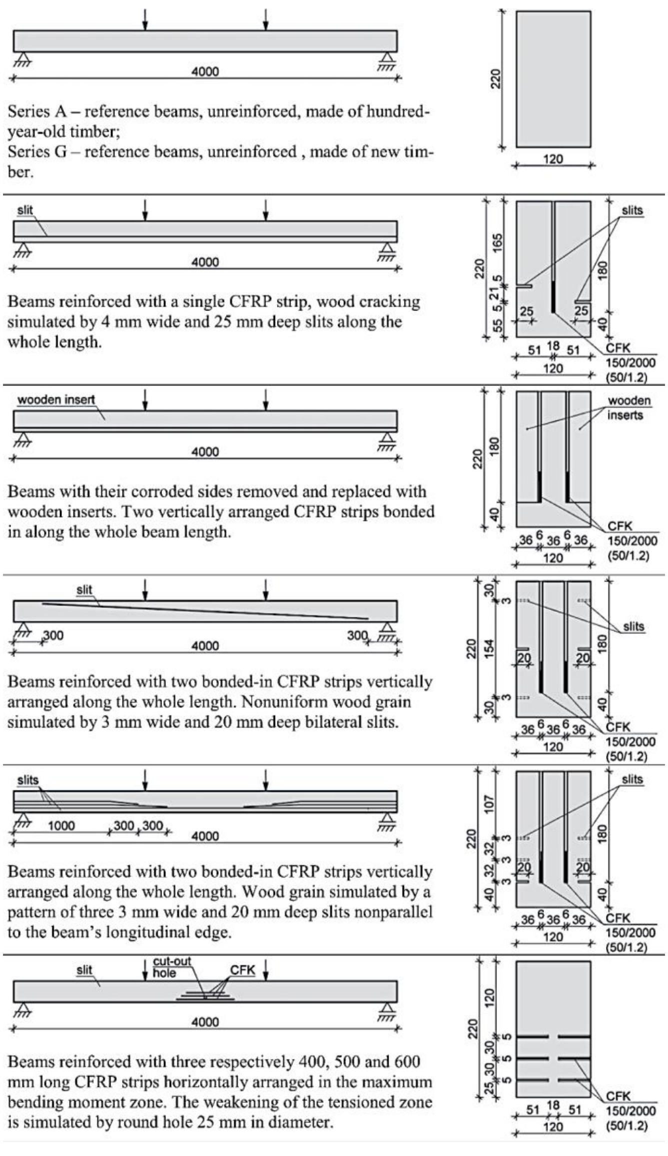


Figure 6.5. Reinforcement schemes investigated by Nowak et al. (2013).

In the U.S.A., Kim et al. (2013b) studied the reinforcement of a 30-year-old floor made with Douglas fir (*pseudotsuga menziesii*) solid timber joists. The joists had a length of 2.44 m and a 38×138 mm^2 cross section. Single joists were tested, as well as 1.5 m-wide floor assemblies with three parallel joists (Figure 6.6). Two types of reinforcement were tested, two layers of CFRP strips or six layers of CFRP sheets, both glued with epoxy adhesives. Some of the beams were intentionally damaged by cutting a notch at mid-span of some of the beams. Since only a single test of each type was conducted, it is difficult to derive reliable conclusions.

CFRP-strengthening significantly increased the load-carrying capacity of single joist between 30 and 180% and also the bending stiffness of the beams without notches. The authors report that the CFRP sheet led to a higher increase in load-carrying capacity then the CFRP strip. The load-carrying capacity of the floor assembly with three beams was lower than three times the load-carrying capacity of the single joists. Once a failure occurred in a joist, the system was not able to redistribute the loads

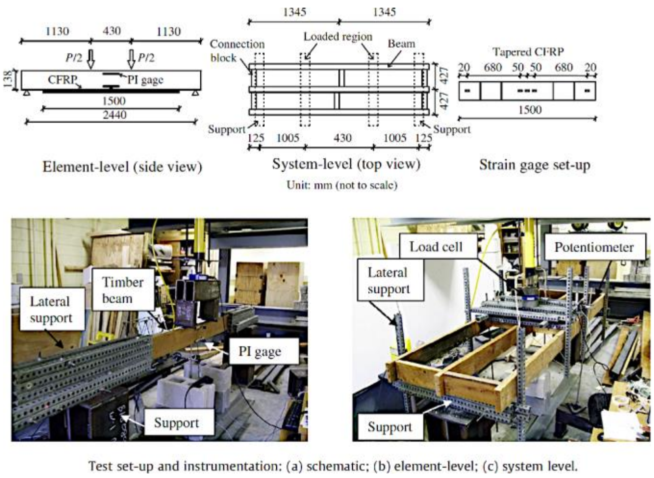


Figure 6.6. Test setup for single joists and floor assemblies (Kim et al. (2013b)).

In Spain, Rescalvo et al. (2017) studied the reinforcement of small-scale 200-year-old solid timber elements made of Scots pine (*Pinus sylvetris* L.). The elements were 1288 mm long and had a 75×145 mm^2 cross section. Different types and amounts of CFRP reinforcement were tested, applied in four different reinforcement schemes (Figure 6.7). The elements were subjected to three-point bending tests under displacement control. The elements presented large defects and two thirds would have to be rejected for structural purposes according to the Spanish visual strength grading rules. The results are difficult to interpret because no force-displacement curves are presented, but stress-time curves (in which the stress is supposedly the maximum stress calculated assuming a linear stress distribution in the cross section), and because the authors also apply a correction to account for the influence of density (increasing the strength of elements with lower densities). Without this correction, some of the reinforced elements even exhibit lower strengths then the unreinforced elements. Nevertheless, some of the elements reinforced with wrapping and longitudinal lamination consistently reached higher strengths.

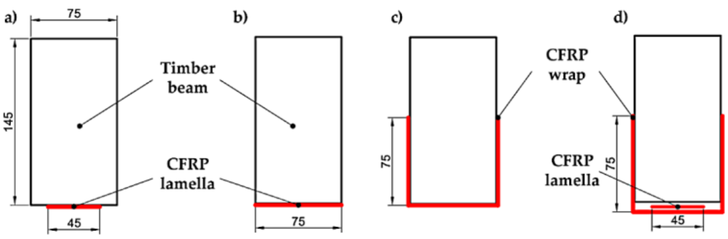


Figure 6.7. Reinforcement schemes (adapted from Rescalvo et al. (2017)).

6.1.2 Shear and tension perp. to grain (including notches at supports)

In Canada, Svecova and Eden (2004) studied the reinforcement of creosote-treated solid timber Douglas Fir (*pseudotsuga menziesii*) beams, cut from salvaged beams taken from a road bridge that was decommissioned after almost 40 years in service. The objective was to develop a reinforcement strategy that would increase bending and shear load-carrying capacity by 30%, so that the beams could support the maximum legal traffic load. The tested specimens were 2000 mm long and had a 100×200 mm^2 cross section. The shear reinforcement comprised GFRP rods, 255 mm long and with a diameter of 16 mm, inserted vertically and glued with an epoxy adhesive. One configuration with smooth steel rods, with a diameter of 12 mm, was also tested. The bending reinforcement comprised GFRP rods with a diameter of 5 mm, inserted in longitudinal groves made on the side faces of the beam (Figure 6.8). The beams were tested under four-point bending and displacement-control. The results on the beams with only shear reinforcement showed that having the shear reinforcement along the entire beam, i.e. not only in the shear zones but also in the bending zone, increased the bending load-carrying capacity by about 34% compared to the unreinforced beams. Having the shear reinforcement only in the shear zones only increased the bending load-carrying capacity by 17%. Regarding shear reinforcement with steel rods, debonding occurred in the steel-adhesive interface, most likely because the adhesive was appropriate for GFRP but not for steel, but nevertheless the beams exhibited an increase in bending load-carrying capacity of 25%, compared to the unreinforced beams, which is higher than the 17% increase observed for the equivalent reinforcement scheme with GFRP rods. Finally, increasing the spacing of the GFRP shear reinforcement from half of the beam height to a spacing equal to the beam height did not reduce the bending load-carrying capacity. With shear and bending reinforcements, the bending load-carrying capacity increased about 50% compared to the unreinforced beams. In this case, the bending load-carrying capacities were not influenced by the spacing of the shear reinforcements (equal to half of the beam height or the beam height) or by having the bending reinforcement extended into the shear zones. The authors report that some ductility was observed in the beams with shear and bending reinforcement, due to compressive failures in timber. No bond problems to the timber or FRP surfaces were reported. The work on shear reinforcement by Amy and Svecova (2004) has already been presented in Subsection 6.1.1 (Figure 6.2). The results showed that the introduction of inclined shear reinforcement in addition to bending reinforcement led to an increase of approximately 20% in the load-carrying capacity, but did not prevent brittle failure modes.

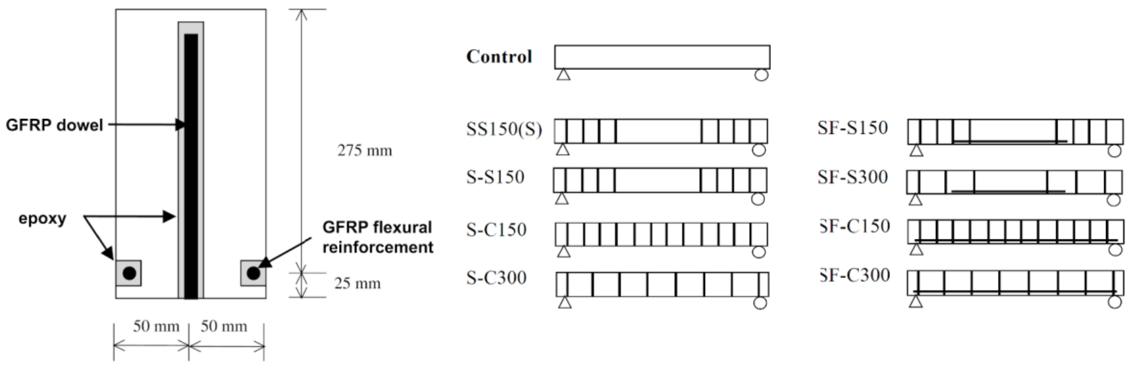


Figure 6.8. Reinforcement schemes studied by Svecova and Eden (2004).

In the U.S.A, Lamanna et al. (2007) and Akbiyik et al. (2007) made a similar study, in which they reinforced failed creosote-treated solid timber beams obtained from a railroad bridge that was in service for several decades. The beams were 4.6 m long, had 0.2×0.4 m^2 cross sections, and were probably made from Douglas fir (*pseudotsuga menziesii*), even though the species is not reported. The studied shear-reinforcement strategies were mostly based on the used of steel-based parts, but mechanically fixed GFRP side plates were also tested (Figure 6.9). It must be noted that the reinforcement was applied after the beams were tested until failure, but not until total collapse of the beam, so there would be some residual load-carrying capacity. The tests were performed under four-point bending and displacement-control. Only one specimen of each reinforcement scheme was tested, which given the variability of mechanical properties makes comparisons difficult. Given the adopted procedure and the lack of repetitions, the test results are difficult to analyse. The reported failure modes show that most of the unreinforced beams failed in shear and that the reinforced beams failed mostly in tension (Figure 6.10). However, this does not mean that the reinforcement was successful in preventing shear failures. In fact, given that most of the beams had extensive shear cracks after the first test to failure, and therefore, a very reduced shear stiffness, a second shear failure in the reinforced beams would be very unlikely. The reinforced beams behaved as timber-timber composite beams, with the reinforcement acting as a shear connector. Given the high variability observed in the load-carrying capacities of the unreinforced beams, between 30 and 65 kN (purple columns in the chart in Figure 6.10), the reinforcement schemes cannot be directly compared to each other in terms of their ability to restore the initial load-carrying capacity of the beam. This is because in the case of originally weak beams the reinforcement would seem to perform better than in the case of strong beams, simply because the threshold it was being compared against was lower. Also for this reason, the reinforcements cannot be directly compared to each other regarding the load-carrying capacity that they reached. Lamanna et al. (2007) and Akbiyik et al. (2007) compare the load-carrying capacity of the reinforced beams against the estimated load carrying capacity of a solid timber beam with the original geometry and a bending strength for "dense select structural southern pine", which is about 200 kN. Only three configurations reach this level (C2-R, C10-R, and C14-R), but there are also the unreinforced beams that exhibited the higher load-carrying capacities, so conclusions about the reinforcement might be biased. Finally, comparing reinforcement schemes of beams that reached the same unreinforced load-carrying capacity (e.g. C4, C7, and C14) might allow deriving unbiased conclusions about the performance of the reinforcement schemes. In this case, the configurations with GFRP panels mechanically attached to the side of the beam reached the highest load-carrying capacity, followed by the reinforcement with inclined lag screws, and finally the one with the epoxied vertical steel bolts.

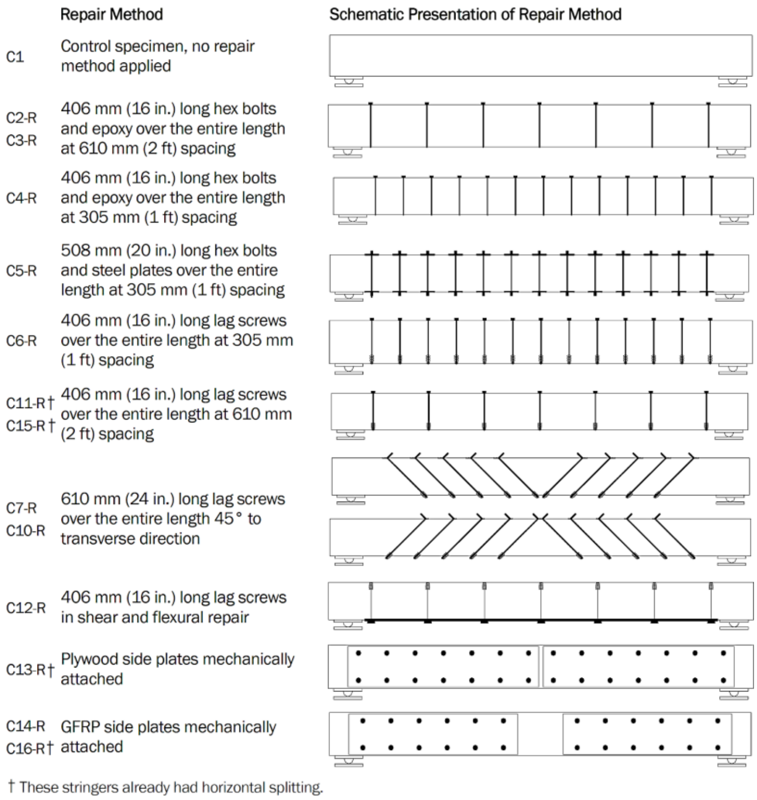
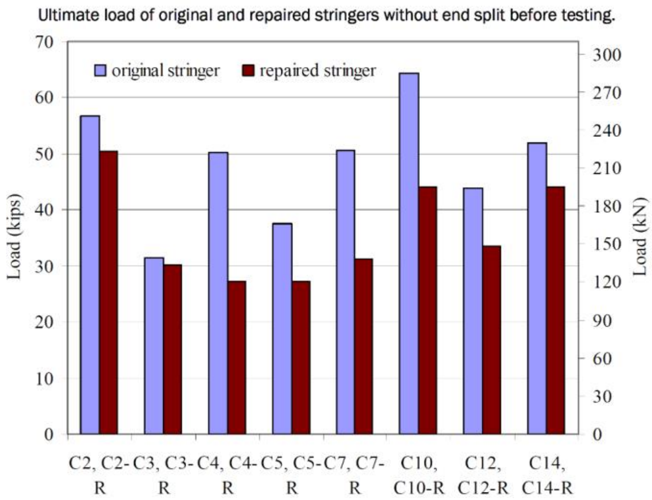


Figure 6.9. Reinforcement schemes tested by Lamanna et al. (2007).



Failure modes of original timber stringers.

C1	Horizontal shear
C2	Horizontal shear
C3	Horizontal shear
C4	Cross-grain tension
C5	Simple tension
C6	End Splitting before testing
C7	Horizontal shear
C10	Horizontal shear
C11	End Splitting before testing
C12	Cross-grain tension
C13	End Splitting before testing
C14	Horizontal Shear
C15	End Splitting before testing
C16	End Splitting before testing

Failure modes of repaired timber stringers.

C2-R	Simple Tension
C3-R	Horizontal shear
C4-R	Cross-grain tension
C5-R	Simple Tension
C6-R	Simple Tension
C7-R	Simple Tension
C10-R	Simple Tension
C11-R	Simple Tension
C12-R	Horizontal shear
C13-R	Simple Tension
C14-R	Compression
C15-R	Simple Tension
C16-R	Simple Tension

Figure 6.10. Load-carrying capacity and failure modes of the various reinforcement schemes (Lamanna et al. (2007)).

6.1.3 Columns

In China, Song et al. (2012) performed buckling tests on solid timber columns with a longitudinal slot, cut to simulate a weakened column. The columns had a length of 1800 mm, a cross section 200×200 mm^2, and were made from Douglas fir (*Pseudotsuga menziesii* var. *menziesii*, strength or grading class not specified). The main reinforcement strategies were based on the use of fully threaded screws (diameter of 6 mm and length of 200 mm) inserted perpendicularly to the slot, but a test specimen with FRP wraps was also tested (the type of FRP is not specified) (Figure 6.11, left). In one of the specimens with screws and in the specimen with FRP wraps, the slot was filled with "wood straps and glue", which is not very clear. The tests were performed under force control and the columns were centred in the testing machine, whose plates could rotate, allowing the column to bend in the direction perpendicular to the slot (Figure 6.11, right). Three replicas of the configuration with screws and gap not filled with "wood straps and glue" were tested, but only one test was performed for the other configurations, i.e. massive column, column with slot, column with screws and filled slot, column with FRP and filled slot. Given the variability of the mechanical properties and the limited number of tests, it is difficult to derive reliable conclusions from the tests. The results showed that the massive column reached 846 kN and the unreinforced column with a slot reached 571 kN. The three columns with a slot and reinforced with screws reached reached 675, 736 and 895kN (average of 767 kN). The column reinforced with screws and with the filled gap reached 812 kN and the column reinforced with FRP wraps and the filled gap reached 835 kN. However, the high uncertainty about the mechanical properties of the timber could mean that the differences observed between the various reinforcement strategies are not due to the reinforcement, but due to the quality of the timber of each tested column.

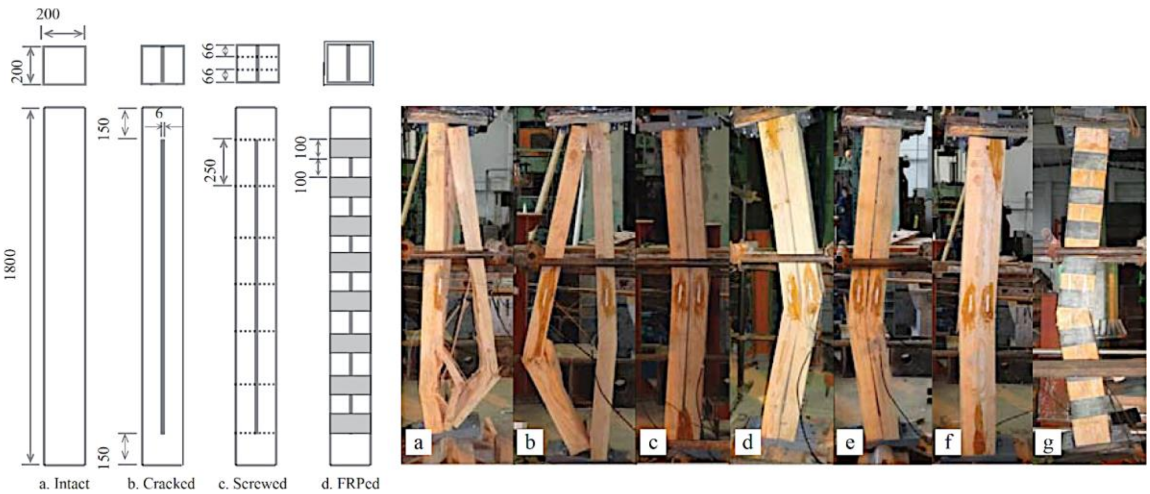


Figure 6.11. Reinforcement schemes (dimensions in mm) and failed columns (Song et al. (2012)).

7 Other aspects of the behaviour of timber-FRP elements

7.1 Long-term performance

7.1.1 Creep

In the U.S.A., Plevris and Triantafillou (1995) performed creep tests on reinforced timber beams in bending and reported that the creep behaviour was primarily dominated by the creep in wood. For constant environment, it was reported that the amount of reinforcement plays a significant role in controlling deformations, but not the type of reinforcement (CFRP or GFRP), and that there is a slight transfer of stresses for timber to the FRP reinforcement with time. For variable temperature and humidity conditions, the role of reinforcement in controlling the deflections was observed to be much more pronounced and the transfer of stresses from timber to the FRP reinforcement with time was considerable. Therefore, FRP composites with low strength under sustained tension, like GFRP, should be used with caution. Galloway et al. (1996) evaluated the creep behaviour of Kevlar prestressed timber beams and reported that the prestressed Kevlar reinforcement did not appear to reduce the deflections over time. This is attributed to the small amount of reinforcement used. Tingley et al. (1996) and Tingley and Gai (1998) monitored the long-term behaviour of FRP-reinforced timber beams in a road bridge. The beams were reinforced with CFRP/AFRP (reinforcement ratios of 0.86 to 1.04%) and, according to Kasal (2012), no significant deflection increase was found over a period of 3 years. Dagher et al. (1998b) performed "accelerated creep" tests on GFRP-reinforced GLT beams, at small ranges of temperature and moisture, and reported no significant difference between the creep of reinforced and unreinforced beams. Davids et al. (2000) assessed the creep deformations in GFRP-reinforced GLT beams, made from Douglas-fir (*pseudotsuga menziesii*) and western hemlock (*tsuga heterophylla*) , for over 650 days under four-point bending in a sheltered environment. The results showed that, compared to the unreinforced specimens, the reinforced beams did not exhibit increased creep displacements despite their higher loading. An increase in the stresses in the reinforcement is also reported, but still far from the ultimate capacity.

In Sweden, Kliger et al. (2007, 2008) performed creep of tests on reinforced spruce (*picra abies*) boards (70×45×3000 mm^3) in four-point bending. The boards were reinforced with a steel strip or CFRP lamination, glued with epoxy adhesive, and exposed to two-week cycles of high (90%) and low (30%) relative humidity during two and half months. The results showed that the reinforced beams exhibited overall lower creep deformations than the unreinforced beams and that the increasing deformations due to creep occurred almost exclusively during the dry part of the cycles. This was attributed to longitudinal shrinkage in timber, which does not occur in the steel or CFRP reinforcement. Kliger et al. (2007, 2008) concluded that this effect would be larger the more one-sided reinforcement was used and that it could be mitigated by also reinforcing the compression side of the beams.

In Canada, Yahyaei-Moayyed and Taheri (2011) performed creep tests on small scale (38×38×432 mm^3) timber specimens reinforced in tension with AFRP composite laminate, glued with an epoxy adhesive. The creep tests ran for one month and were performed under constant environmental conditions. The presented results do not allow to directly compare the reinforced and unreinforced specimens, but finite-element models of reinforced GLT beams (calibrated with the test results) seem to show no beneficial influence of the reinforcement on the creep deflections and that the prestressing of the reinforcement would experience losses up to 17%.

In Ireland, O'Ceallaigh et al. (2016, 2018a; b) studied the creep behaviour of BFRP-reinforced GLT beams in bending, under constant conditions corresponding to Service Class 1 (as defined in EN 1995-1-1). The GLT beams (98×125×2300 mm^3) were made from Sitka spruce (*picea sitchensis*) laminations of strength grade C16, therefore of quite low quality. The BFRP reinforcement comprised two NSM BFRP rods with a diameter of 12 mm and glued with an epoxy adhesive. The tests were performed under four-point bending and the load was adjusted so that all the beams were under maximum bending stresses of 8 $N{\cdot}mm^{-2}$ on the compression face. It was reported that there was no significant reduction in the creep deflection when comparing the unreinforced and reinforced beams, loaded to a common maximum compressive bending stress. The creep deflection was governed by the stress level in timber and the reinforcement had an insignificant effect. The influence of the reinforcement on the total creep response was reported to be indirect and due to an increase in elastic stiffness. Therefore, the current creep modification factors provided for solid timber or engineered wood products in EN 1995-1-1 may be adequate for FRP-reinforced beams under Service Class 1 conditions.

7.1.2 Durability

The research presented in Section 4.1, regarding the bond behaviour between timber and FRP composites, included several studies in which the shear strength of the timber-FRP interface was evaluated after exposure to wet-dry cycles. The objective of delamination tests, like the ones specified in Annex C of EN 14080:2013, is to simulate the aging of the interfaces by imposing moisture-induced stresses, namely in the direction perpendicular to the grain. The results of those studies revealed that the durability of the timber-FRP interfaces depends most of all on the specific combinations of timber species, adhesive, FRP composite, and application technique. Even adhesives of the same type can exhibit very different levels of delamination and apparent shear strength after being exposed to the wet-dry cycles. Some authors have questioned the excessively harsh test conditions (Rowlands et al. 1986) or that they might not represent aging in service conditions (Lopez-Anido et al. 2005), but timber-to-timber glue lines would be subjected to the same requirements, so the comparative studies were at least fair and consistent. Boothby and Bakis (2008) recognise the numerous studies reporting poor performance of externally-bonded FRP composite systems, regardless of the substrate, but note that "as with conventional materials, the performance of FRP material systems depends on proper design, material selection and workmanship".

In China, Zhou et al. (2015) studied the effect of moisture on the mechanical properties of CFRP–wood composite and concluded that the load-carrying capacity of small CFRP–wood samples (70×70×420 mm^3) significantly decreased with the increase of the duration of exposure to a humid environment.

In Iran, Toufigh et al. (2018) investigated the influence of long-term environmental effects on the bond strength of timber-FRP composites by performing single-lap shear tests on specimens exposed to various environmental conditions. These conditions included acidic, alkaline, fresh water, and seawater solutions for periods of 1 to 12 months, including ultraviolet radiation after 6 months. The results showed that bi-directional CFRP and GFRP sheets exhibited better bond strength compared to uni-directional sheets, in most cases, after exposure to the chemical solutions and ultraviolet radiation. Acidic solutions caused the highest reduction of bond strength, whereas seawater solution cause the smallest reduction. Bi-directional AFRPs are reported to exhibit the highest reduction of bond strength under the effect of very alkaline solutions.

7.2 Calculation models

The calculation models developed to evaluate the behaviour of timber-FRP composites fall mainly in the following categories: equivalent-section models, transformed-section models; fibre-section models; 3D finite-element models; and bond-slip models.

Equivalent-section models, also called transformed-section models, are the simplest and most commonly used to analyse composite cross sections. Equivalent-section models can be used to account for stiffness differences and allow for failures in other than the outermost laminations. In design codes, they are commonly used with linear-elastic material behaviour and a single-point maximum stress failure criterion. They allow estimating the bending stiffness for serviceability limit states and the load-carrying capacity for ultimate limit states. But they can also be combined with non-linear material properties, i.e. non-linear stress distributions can be assumed. Analytical solutions can be derived for selected failure modes, and no iteration steps are required to compute the load-carrying capacity of the cross section. These models have been used by (not exhaustive list): Moulin et al. (1990), van de Kuilen (1991), Romani and Blaß (2001), Gilfillan et al. (2003), Johnsson et al. (2007), Fossetti et al. (2015), Brady and Harte (2008), Persson and Wogelberg (2011), Yang et al. (2016).

Fibre-section models, also sometimes called strain-based models, of composite timber-FRP cross sections are also a very common type of numerical model for research purposes. These models lend themselves easily to the task because the mechanical behaviour (i.e. stress-strain relationship) of different materials, including long-term phenomena, can be easily included in the behaviour of the fibres and subsequently integrated over the cross section, and then over the element, to describe its response. The various stages of non-linear behaviour that occur with increasing curvature of the cross section can be clearly shown and can be easily linked to specific mechanical and geometrical properties. Pure bending or combined bending and compression can both be easily dealt with. These models have been used by (list not exhaustive): Plevris and Triantafillou (1992), Davids et al. (2000), Lindyberg and Dagher (2012), ASTM D7199 − 07 (2012).

Finite-element models, namely using 3D solid elements, have been increasingly used. They are a particularly attractive option to model specific aspects of timber-FRP composites, such as debonding in the timber-adhesive-FRP interfaces. However, the models require many parameters that are not easily available and are often be developed alongside calibration tests (e.g. to obtain fracture energies or to calibrate failure criteria). They have been used by (list not exhaustive): Hallström and Grenestedt (1997), Kim and Harries (2010), Fornander and Nihlmark (2013), Khelifa and Celzard (2014), Dias et al. (2015), Glišović et al. (2016), Guan et al. (2005), Khelifa et al. (2015a).

One of the most important aspects regarding modelling bonded FRP composites are debonding failure modes in FRP-substrate interface. This is usually investigated using **bond–slip models** for the interface, which are usually based on pull-out tests. These models have been developed by (list not exhaustive): Biscaia et al. (2017), Subhani et al. (2017a), Vahedian et al. (2018).

Other models for timber-FRP composites have also been developed (see Cabrero et al. (2010)).

7.3 Fire

The fire behaviour of FRP-reinforced members is always a concern given the recognized poor performance of many adhesives and polymer matrices when exposed to high temperatures. In this case, "high" is related to the glass transition temperature of the polymers, which can be much lower (lower than 100 °C, sometimes even close to 50°C than those that develop during a fire. Martin and Tingley (2000) performed fire resistance tests on FRP-reinforced and unreinforced GLT beams under standard fire exposure. The loads applied during the tests seem quite high (above the design loads). The results showed that placing the FRP reinforcement between timber laminations increased the fire resistance, compared to directly exposed FRP reinforcement. The different FRPs (matrix and fibres) had no significant influence in the fire resistance, but different matrix formulations (with higher glass transition temperatures) may slightly improve the fire resistance of non-exposed FRP reinforcements. Fire resistant coatings applied to the exposed FRP composites showed no significant improvement of the fire resistance. Williamson (2006) tested beams with an EB FRP composite and also reported no discernible differences in the overall fire performance of different FRP adhesives. The objective of these tests was to show that GLT beams with exposed FRP reinforcement could reach 1 hour of fire resistance to the standard fire curve. However, the reinforced GLT beam height was increased by "10% plus one lamination", therefore, it is not very clear if even without the exposed FRP reinforcement the beams would not have reached the same fire resistance, defeating the purpose of reinforcing them in the first place. Wall et al. (2018) performed bending tests on small-reinforced GLT beams exposed to radiant heat. The FRP reinforcement was protected by a timber lamination. It was reported that the reinforced beams were able to withstand the fire exposure for longer, but the applied load seems to have been the same for both the reinforced and unreinforced beams. Since the reinforced beams were reported to have a higher load-carrying capacity at normal temperature, they should also have been tested under higher load when tested under fire.

7.4 Ecologic aspects, life-cycle assessment (LCA), and public perception

As with many technology-related problems, the public perception of a given technology can have important implications in its success or demise. Therefore, the views of key stakeholders (policymakers, regulators, owners/developers, architects, engineers, contractors, producers/suppliers, and, maybe most important, the end users or target public) on construction technologies should be discussed alongside with technical aspects.

The widespread public perception of timber as an "eco-friendly " material (Markström et al. 2018, 2019) and an increased focus of policymakers on the development of "bio-based" solutions (European Commission 2016), contrasts with the image of the plastics and FRP/composites industries as co-responsible for plastic pollution, which has recently received a great deal of public attention. Developers and architects are, therefore, motivated to employ wood-based products, even if in an unsustainable way, simply because of its association with *eco-friendliness*, whereas the use of FRPs is frowned upon. In addition to the economic aspects, the negative impression of the use of "plastics" may be one of the reasons why the use of FRPs in the construction industry has always been limited to niche applications in repair/reinforcement interventions (and even there, the mostly non-reversible nature of FRP-based applications is often frowned upon).

7.4.1 FRP

In the last decades, FRP composites have been widely used in many engineering industries. Managing FRP waste has become an important issue due to the growing use of FRPs in the construction and transportation industries and more restrictive regulations and public perception. Landfilling FRP waste is still the easiest and cheapest method for managing FRP waste in most countries, even though in many countries, including Germany and Switzerland, have much stricter regulations (Yazdanbakhsh and Bank 2014). The most common application of mechanically recycled FRP waste is to use as filler in new FRP composites and partial replacement of aggregates in concrete and mortars, but this significantly reduces their mechanical properties. Thermosetting FRPs are more difficult to recycle because they cannot be easily remelted, but thermosetting resins are currently used far more often in FRPs since they allow much faster production, have better properties of impregnation and adhesion to the fibres, and guarantee better mechanical performance (Correia et al. 2011). The existing knowledge on the recycling/reusing of FRP waste produced in the construction industry is still very limited. "Closing the loop" in the life-cycle of FRP composites is still an open issue and, therefore, increasing its use in the construction industry must be accompanied by research on effective recycling strategies. Some possibilities may lie with using by replacing epoxy based resins with polyester based resins or preferably bio-resins and the use of natural fibres (André 2006).

7.4.2 Timber

The structural timber can be obtained sustainably, as it is a renewable natural product. It requires little energy for extraction and processing (most energy is used in drying and in producing the required adhesives). The use of native wood species while optimising and minimising transport routes is another ecological advantage. The comparatively low density in relation to the mechanical properties leads to larger possible transport volumes. If not combined with other substances (through impregnation, or surface applications), timber products can be recycled or biodegraded completely and without pollutants.

When biomass (wood) is formed, CO_2 is extracted from the atmosphere and the carbon is incorporated into the biomass. Forests and wood products are thus carbon sinks and contribute positively to the reduction of greenhouse gases.

With proper structural design, sufficient durability of a timber structure can be achieved without the use of chemical wood preservatives. It is ideal that the design lifespan of structural timber matches timber rotation periods, therefore enabling "sustainable-yield logging" (Ramage et al. 2017). Timber elements can be reused several times, e.g. in wood-plastic composites or fibreboard panel, or used as pulp or fuel.

8 Guidelines, standards, and specifications

An extensive list of published design guides, codes and specifications for FRP composites in structural engineering is maintained by the International Institute for FRP in Construction (IIFC) on their website https://www.iifc.org/publications/code-references/ (International Institute for FRP in Construction (IIFC) n.d.). However, this list does not include timber-related documents.

The Swiss standard **SIA 166:2004. Klebebewehrung** covers the use of bonded reinforcement to strengthen the load-carrying structures. The standard shows the possibilities and restrictions of this technology and proposes uniform design methods. The standard is mostly focused on reinforced-concrete structures and has only a few clauses related to timber structures. Regarding bonding of FRP reinforcements on timber, the recommendation is, in the absence of a realistic model, to assess it through testing. It is also mentioned that creep in timber may cause the stresses in the reinforcement to increase over time, which should be accounted for.

The Italian guideline **CNR-DT 201/2005. Guidelines for the design and construction of externally bonded FRP systems for strengthening existing structures – Timber structures** focuses on the use of externally-bonded systems for reinforcement of timber structures. When it was published, there were no international standards on the topic and the objective of the document was to disseminate the state of knowledge at the time, while helping to identify unsolved problems. It explains the basic concepts of reinforcing with FRP and the associated issues, gives guidance on reinforcing structural members under combined bending and axial forces, with particular emphasis on timber floors and their stiffening for in-plane loads. Qualitative aspects related to debonding problems and reinforcement of timber connections are also addressed. Case studies are presented and relevant references to existing design codes are listed.

The American standard **ASTM D7199 – 07 (2012). Practice for establishing characteristic values for reinforced glued laminated timber (glulam) beams using mechanics-based models** establishes a mechanics-based design method to calculate the bending properties (stiffness and load-carrying capacity) of FRP-reinforced GLT cross sections. It does not cover cross sections subjected to combined bending and axial loads, unbonded reinforcement, prestressed reinforcement, or shear reinforcement. Long-term effects are also not covered by the design method. It also describes a minimum set of performance-based durability test requirements for reinforced GLT, as specified in Annex A1 of the standard. Additional durability test requirements shall be considered in accordance with the specific end-use environment. Appendix X1 provides an example of a mechanics-based methodology that satisfies the requirements set forth in this standard.

9 Examples of applications in practice

If special cross-sectional dimensions are required for aesthetic, functional, transport or assembly reasons, FRPs offer economical and modern solutions. In most cases, however, the use of FRPs can avoided by improving the quality and/or increasing the dimensions of the structural elements. This is one of the reasons why fibre-reinforced plastics are used more often for repairs than for new buildings.

9.1 Planned strengthening

Footbridge, Taylor Lake, Oregon, USA, 1992 – Directly above the 24 m long bridge there is a power line. A large assembly crane was therefore out of the question for the installation. In order to enable the bridge to be assembled with a smaller crane, the bridge weight had to be reduced. By using AFRP composites in the GLT main girders, the wood consumption could be reduced by 32%.

Figure 9.1. Footbridge, Taylor Lake, Oregon, USA.

Footbridge, Iwamizawa city, Hokkaido, Japan, 1994 – The 33 m-long pedestrian bridge was the wooden bridge with the largest span in Japan at that time. The maximum cross-section height of 0.9 m required by the architects could only be achieved by using FRPs. The "pure" timber solution would have resulted in a height of the GLT girder of 1.3 m. The reinforced beams were produced in the U.S.A.

Figure 9.2. Footbridge, Hokkaido, Japan.

Road bridge, Clallam Bay Highway, Washington State, USA, 1995 – The load-bearing system of the 50 m-long timber bridge consists of 26 FRP-reinforced GLT beams. The use of CFRP and AFRP composites made it possible to maintain the necessary clearance gauge of the river without having to increase the terrain significantly.

Figure 9.3. Road bridge, Washington State, U.S.A.

Timber hall in St. Erhard, Switzerland, 1999 – The primary structure is formed by five trusses, each with a distance of 20 m between them. The primary beams consist of two 35 m-long GLT beams, which are rigidly connected in the ridge area. They span two bays of 30 m each. The primary beams have been partially reinforced in the zones with maximum bending tensile stresses. For this purpose, several layers of aramid fibre strands were glued between the outermost and second outermost lamella. The gluing was done with conventional resorcinol resin glue, as it is also used for non-reinforced glulam. With the reinforcement technology used, the beam cross-section could be reduced by approximately 25%.

Figure 9.4. Timber hall in St. Erhard, Switzerland.

Joinery hall in Eschenbach, Switzerland, 2000 – By gluing two layers of GFRP laminations over the central supports (negative moments), it was possible to dispense with increased beam height over the support. The GLT beams are 35 m-long and have a quite small cross section.

Figure 9.5. Joinery hall in Eschenbach, Switzerland.

Gymnasium in Kaisten, Swizerland, 2001 – The main beams have a length of almost 40 m and are designed as double beams. The GLT beams are reinforced with aramid fibres above the central columns and in the larger spans.

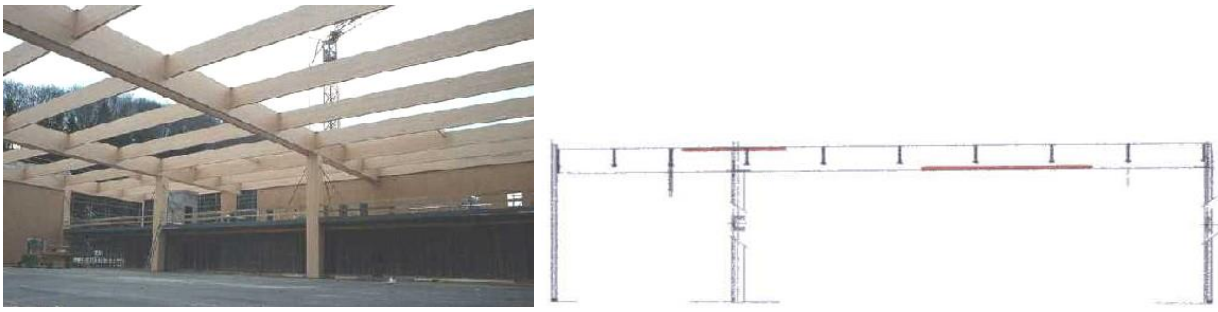


Figure 9.6. Gymnasium in Kaisten, Swizerland.

Industrial hall in Sattel, Switzerland, 2001 – The 47 m-long GLT beams produced with storm-damaged timber are oversized and have GFRP laminations glued in as additional safety measure.

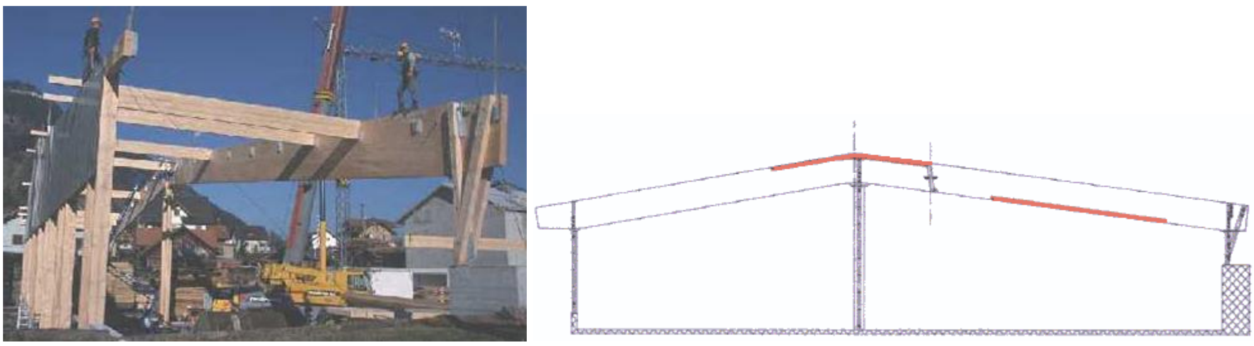


Figure 9.7. Industrial hall in Sattel, Switzerland.

9.2 Reinforcement of existing structures

Restoration of the wooden bridge in Sins, Switzerland, 1992 - The stiffness and load-carrying capacity of the most heavily stressed original oak cross beams were increased with CFRP lamellas. These lamellas were glued to the top and bottom of the exposed beams. The use of CFRP lamellas did not alter the external appearance of this historic structure (Meier et al. 1992).

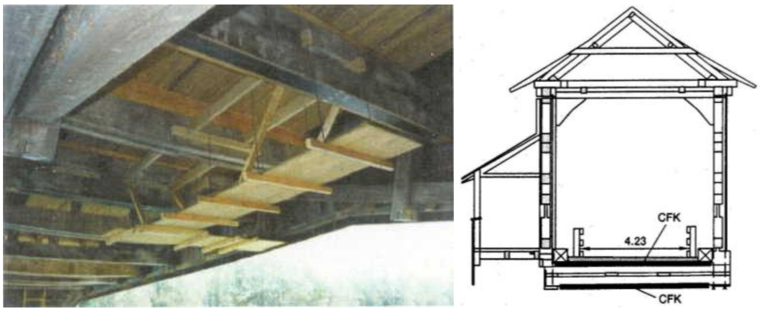


Figure 9.8. Industrial hall in Sattel, Switzerland.

Renovation of a pedestrian bridge, Zumikon, Switzerland, 1998 – In 1998, a 2.5 m-long and 25 cm-high damaged piece of a GLT beam girder was sawn out and replaced by a fitting piece of solid wood. The connection was made by means of gluing and additional vertical screws to absorb transverse forces. As an additional safety measure, above all to minimize the notch effect in the tensile stressed part of the GLT beam, a CFRP lamination was glued to the GLT beams from below over the fitting piece.

Figure 9.9. Reinforcement with CFRP laminates.

Renovation of the timber bridge, Murgenthal, Switzerland, 1998 – Due to insect damage, the bottom chord was reinforced with CFRP lamellas. There was only a time window of a few night-hours available for the construction works, therefore a heating device was used to accelerate the curing process.

Figure 9.10. Reinforcement with CFRP laminates.

Conversion of Eschenbach Monastery, Switzerland, 1999 – The old ceiling beams no longer met the static requirements and had to be reinforced. In order to meet aesthetic requirements, the CFRP lamellas were glued to the bottom of the beams and covered with a cover board. Before the gluing, the wooden beam was relieved by temporary supports.

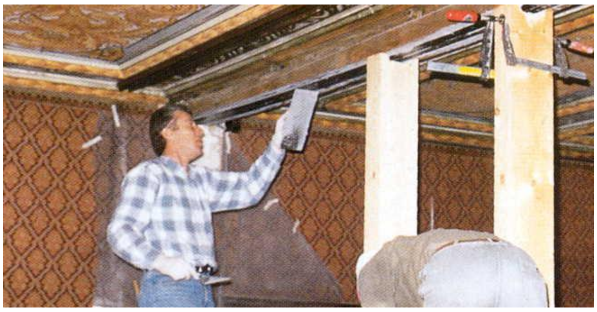


Figure 9.11. Conversion of Eschenbach Monastery, Switzerland.

10 Conclusions

Many attempts to improve the behaviour of timber beams have presented in this report, with different production and cost requirements and varying levels of effectivenesses. In this report, only the effectiveness of the various strengthening methods was evaluated and compared, not the simplicity/complexity of production aspects, even though the differences are clear in most of the cases, and definitely not the relative costs of the various methods. These are also very important aspects and maybe related to some of the reasons why the strengthening of timber members with FRPs always remained a niche market, more related to the reinforcement of existing structures.

The reviewed literature covered several decades and showed that most FRP-based strengthening strategies of timber structural elements had been tried by the late 2000s. The most commonly studied strategies were related to improving the bending behaviour of GLT beams. Improvements in the load-carrying capacity and stiffness of GLT beams were observed by gluing FRP composites directly to the tension face of the beams, by inserting them between timber laminations (therefore protecting them from direct exposure) or in longitudinal grooves (e.g. FRO rods), and also by embedding only the fibres in the timber adhesive (usually PRF). It was also often reported that longitudinal bending strengthening led to other types of unwanted brittle failures, such as shear failures. Studies on strengthening with pre-stressed FRP composites have repeatedly shown no significant improvement over passive (i.e. non pre-stressed) strengthening. An important aspect regarding the behaviour of timber beams reinforced with FRP composites is that, unlike in reinforced concrete, ductility comes from over-reinforcing in tension, to induce compressive parallel to the grain failures in timber. This is because FRP composites are brittle, unlike steel reinforcements.

Regarding shear and tension perpendicular to the grain, research has been much more limited, but strengthening based on applying FRP composites on the side faces of timber members is an effective strategy, but aesthetically unappealing. Internal shear and tensile reinforcement with glued FRP composites do not seem to provide any significant advantage over the nowadays common reinforcement with self-tapping screws or glued-in steel rods GiRs, except maybe if fire is a concern and exposed steel parts are to be avoided.

Research on the strengthening of timber columns with FRP composites has also been limited and tests at the structural scale have shown that steel reinforcements might performed better and that FRP-based reinforcement might only be advantageous in the case of relatively deep cross sections.

The use of FRP composites in timber connections has shown that it can be successfully used to strengthen timber in the connection area or to replace some of the connection components that are usually made of steel. In the first case at the cost of additional production steps and facing the competition of high-performing timber-based panels, and in the second case at the cost of lower load carrying capacities. As GiRs, FRPs have a major difference to steel, which is that ductile failure modes are not possible.

The timber-FRP bond behaviour has been extensively studied and current knowledge allows the development of reliable systems, i.e. combinations of materials, adhesives, and surface treatments. The durability of timber-FRP bonded interfaces is highly dependent on the specific nature of the components that are used and on the bonding procedure, being extremely difficult to make even general statements about types of adhesives or FRPs. Nevertheless, specific combinations have shown that they can fulfil the same performance requirements that are set for timber-timber bond interfaces. Given the wide variety of timber species, the inherent anisotropy and high variability of timber, the development of reliable general bond-slip models is complex and has mostly started only in the last few years.

The long-term behaviour, namely creep, is still an open issue, namely under carrying climate and loading conditions (as it still is for unreinforced timber members). There are only a few available studies, but most of them seem to show no reduction of creep deformations in FRP-strengthened beams.

The fire behaviour of FRP-strengthened timber elements is highly dependent on how protected the FRP composite is within the timber cross section. Since the effect the of the strengthening is significantly reduced when the FRP composite is not located very close to the zone of maximum stresses, the need to protect it against direct exposure to fire usually involves a compromise between increasing the load-carrying capacity at normal temperature and fire resistance. Nevertheless, if exposure to fire is a concern, some types of strengthening with FRP might be more interesting than the equivalent reinforcement with steel parts, since FRPs might not conduct so much heat into the cross section. This is mainly relevant for reinforcement or components that are embedded and go through the entire cross section (or most of it), such as internal reinforcements against shear and tension perpendicular-to-the-grain failures and also for dowel-type fasteners.

Nowadays, FRP-based strengthening of structural timber faces well-established competitors. For longitudinal strengthening (i.e. bending, compression), the main alternative is the use of high-performing timber-based products (e.g. spruce or beech LVL), which can be easily integrated in existing production processes. For transversal strengthening (i.e. shear and tension perpendicular-to-the-grain), the main alternatives are self-tapping screws and glued-in steel rods. Steel reinforcements have a significant advantage of being easily machined and connected to other structural elements and easily allowing for ductile failures.

Finally, and ever more relevant, the ecological aspects related to reusing, recycling, and disposing FRPs is still far from being solved and might pose ever more difficulties to the use of FRP composites. Another aspect that might hinder the adoption of FRP-based strengthening from timber members is that the widespread public perception of timber as an "eco-friendly" material clashes with the image of "plastics", often associated with the omnipresent "plastic pollution".

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