

Failure modes and reinforcement techniques for timber beams – State of the art



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HIGHLIGHTS

- Review of typical materials, cross sections and forms of timber beams.
- Description of general failure modes and their possible causes.
- State of the art of established retrofitting and reinforcement techniques.
- Discussion of case studies for reinforcement techniques.

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ABSTRACT

Highly loaded and large span timber beams are often used for halls, public buildings or bridges. Reinforcement of beams may be required to extend the life of the structure, due to deterioration or damage to the material/product or change of use. The paper summarises methods to repair or enhance the structural performance of timber beams. The main materials/products cross sections and geometries used for timber beam are presented. Furthermore, their general failure modes are described and typical retrofitting and reinforcement techniques are given. The techniques include wood to wood replacements, use of mechanical fasteners and additional strengthening materials/products.

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1. Typology of timber beams

Timber beams can mainly be classified according to the span, the geometry and the material/product used, as summarized in [Table 1](#). The focus within this article is on high-performance, long-span structures. [Table 2](#) gives an overview of typical timber beams in relation to the sizes of the cross section and the span ratio. In Europe, glulam members or block glued glulam members are the main construction elements used for large open span spaces, stadium roofs or bridges in which the primary structure is timber. The typical layered cross section of glulam reaches from 100 to 250 mm in width and up to 2500 mm in depth but also in larger dimensions as block glued glulam. Box or composite beams are alternatives providing a lower self-weight.

2. Failure modes

2.1. General

Structures have to adopt, and transfer external loads to the ground and also to deal with the corresponding internal loads (normal force, shear force and moment). This leads to stresses and deformations in the structure which must not exceed design strength and deformation limits. In designing new structures, a full cross section with minor damage and correct material grades are assumed. However, in existing timber structures the cross section and/or the properties of the material/product of the members can be reduced due to mechanical and biological damage. Both types of damage influence the load carrying capacity and serviceability of single members or the complete construction. Within the assessment of timber structures, damage or failure has to be detected and assessed for the resistance and serviceability of the timber structure. The net cross sections observed at failures or damages must be compared to the designed cross sections.

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Table 1
Classification of timber beams.

Category	Representatives, options
Material/product	Solid wood; glulam, block glued glulam; laminated veneer lumber; plywood; (OSB, LSL); cross laminated timber
Cross section	Solid cross section; box-beam; I-beam; T-beam; C-beam
Geometry	Straight beam; curved beam; tapered beam; truss

The failure analysis on timber structures in Germany carried out by Blass & Frese [1,2] gives a good overview of the distribution of main types of failure classified according to the construction, use and region. Most assessment reports state that the timber structures have been built using glulam beams of quality GL28h (see Table 3). Their shape, however, is more varied with the most common being, by order: straight (154/426), tapered straight (124/426), pitched cambered (90/426) and curved (47/426). 80% of the failure cases could be detected in bending members, followed by 8% in compression members. Furthermore in 75% of the failure cases cracks could be detected. Typical reasons and types of failure are summarized in Figs. 1 and 2. For high performing and long span timber members the typical failure modes are described in detail in the following sections.

2.2. Cracks

The most common type of failure, Fig. 2, was observed as the appearance of cracks in grain direction. The variation of the surrounding climate at a timber beam changes the moisture content and lead to shrinkage or swelling of the cross section. Non uniform distributions of the moisture content over the cross section and/or restraint deformations lead to internal stresses and, if the material strength is exceeded, to cracks in the cross section which can significantly reduce the capacity, Figs. 3 and 4. For the determination of the influence of cracks in timber beams on the residual load carrying capacity or stiffness no comprehensive methods are known. Methods and guidelines for this evaluation are currently under development at the Bern University of Applied Sciences.

The amount and distribution of cracks depends on several factors, such as timber, defects, loading situation, beam shape and

Table 2
Overview of timber beam forms.

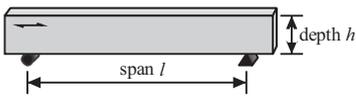
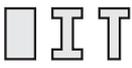
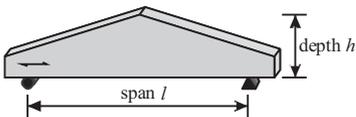
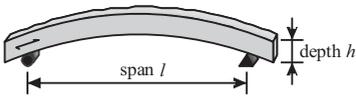
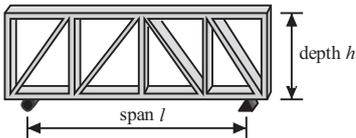
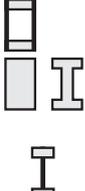
Timber beam form		Cross section	Span, depth ratio
Straight beams			$10\text{ m} \leq l \leq 40\text{ m}$ $h \approx l/17$
Tapered beams			$12\text{ m} \leq l \leq 25\text{ m}$ $h \approx l/15$
Curved beams			$15\text{ m} \leq l \leq 35\text{ m}$ $h \approx l/17$
Trusses			$20\text{ m} \leq l \leq 85\text{ m}$ $h \approx l/10$

Table 3
Most frequent characteristics of the timber structures assessed, from [1,2].

Characteristic	Main result	Corresponding no. of assessments	
Material/Product	Glulam	594	80%
Quality (or equivalent)	GL28h	68	72%
Load type	Bending	470	80%

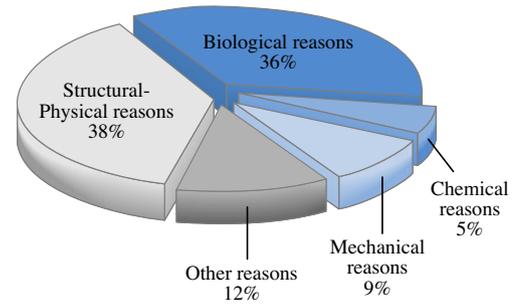


Fig. 1. Reasons for damages, [1,2].

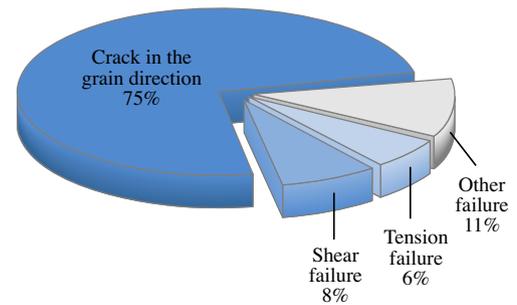


Fig. 2. Types of failure, [1,2].

the glueline quality for glued members. Regarding the distribution of cracks in the timber beams, a summary of their characteristics can be found in Table 4.

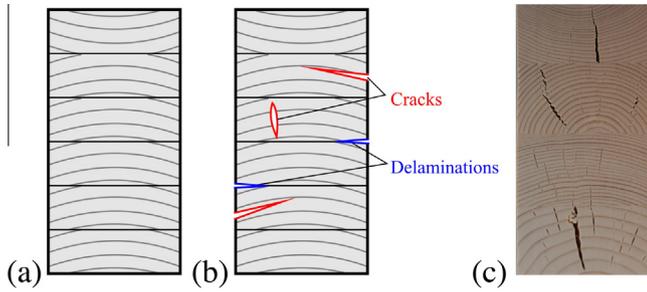


Fig. 3. Glulam cross sections; (a) sketch of undamaged cross section with 100% capacity, (b) sketch of cross section with some possible damage and unknown capacity, (c) real example of cross section with internal cracks.

Note: Failure under tension stress perpendicular to the grain in glulam members has to be distinguished from delamination failure within the glue lines as shown in Fig. 5. Special techniques can be used for the classification of delaminations as shown in [3].

2.3. Bending failure

Bending results in longitudinal tension and compression stresses distributed over the depth of the cross section. The tension stress leads to a brittle failure due to the rupture of the wood fibres, as shown in Fig. 6. Longitudinal compression stress results in elastic and plastic deformations which can be described as ductile and leads to the so-called kink bands.

Due to natural defects, such as knots, the tension strength can be reduced compared to compression strength. Therefore, bending failure is mainly described by brittle failure of timber beams within the tension zone, as shown in Fig. 7. Bending failure is classified as critical and can lead to a single failure of the structural element or the complete construction.

2.4. Compression failure

Failure under longitudinal compression stress occurs mainly in timber trusses, beams or columns. Failure under compression stress perpendicular to the grain can also be described as a ductile failure with plastic deformations and occurs mainly at supports or at loading points where high loads have to be transferred, as shown in Figs. 8 and 9. In both cases, these plastic deformations can further lead to eccentricities and load redistributions within the complete structure and therefore overstress parts of the structure. The overall stability can also be influenced.

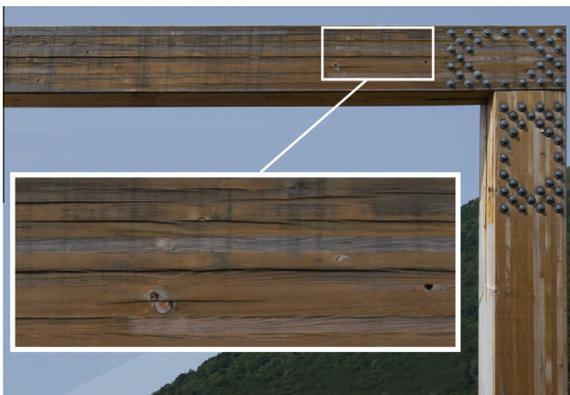


Fig. 4. Cracks in grain direction at a glulam member.

Table 4 Characteristics of cracks and their distributions, from [1,2].

Crack cause	Location; quantity	Length; depth ratio*	Cases (%)
Stress concentration (restrained shrinkage, notches, transverse forces ...)	At the singularity; single	1–10 m; mostly 1.0	35
Normal climate changes	Randomly; numerous	0.1–1 m; 0.1 to 0.4	33
Element quality (glue line or finger joints)	At the defect; depending on its extent		17
Overloading (shear or bending stresses)	Various; single to numerous	1 m; mostly 1.0	15

* Ratio of depth of crack to width of beam.

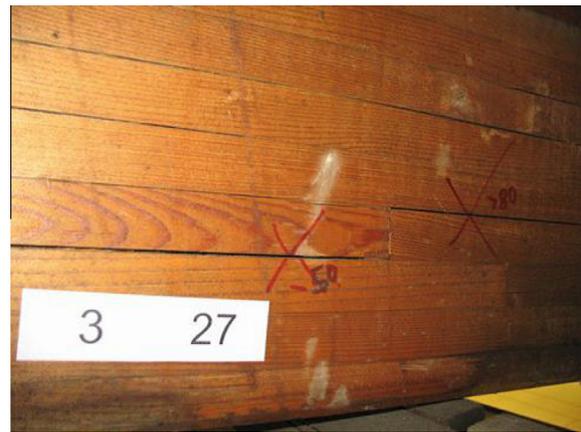


Fig. 5. Delamination at a glulam member.

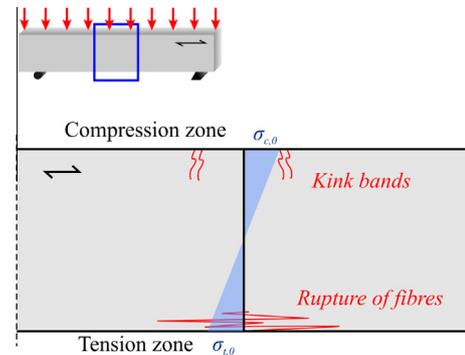


Fig. 6. Principal sketch for bending failure.

2.5. Tension failure

Tension stress has to be considered parallel to the grain and perpendicular to the grain directions. When the tensile capacity of the timber is exceeded, brittle failure occurs. Examples of tensile failure parallel to the grain and perpendicular to the grain are shown in Figs. 10–12. However, due to the low tension strength perpendicular to the grain of wood, which is almost zero due to natural defects, failure under tension stress perpendicular to the grain occurs more often. Therefore, wood products are mostly optimised to increase the tension strength perpendicular to the grain. However tension stresses perpendicular to grain has carefully to be considered in the design. Tension stress perpendicular to the grain occurs in curved, tapered and end-notched glulam members as



Fig. 7. Tension failure under bending.

well as in members with holes, additional connected structural elements or equipment, and at connections loaded perpendicular to the grain.

2.6. Shear failure

In most cases, bending stress and deflection limits govern the design of the members. But for short beams, tapered beams or special loading situations the shear stress can be more important. In general for beams, the shear stress reaches the maximum value close to the supports (Fig. 13). Additionally, end-notched beams and beams with holes can lead to shear stress concentrations, [5,6]. Failure due to shear stress is characterised by a sliding of the fibres and thus cracking parallel to the grain and is considered as a brittle failure. The cracks are mainly closed and therefore hard to detect if they are not at the end of the beam, as shown in Fig. 14.

2.7. Insects and fungi

Decay due to fungi is possible for timber beams with a moisture content close to/or over the fibre saturation point, see Fig. 15. The fibre saturation point varies from wood to wood species and shows a range from 26–32 M% (in mass percentage). The different fungi and their typical appearance and hazard are summarized in [7].

Generally, decay due to insects can occur within a range of wood moisture content above 6 M%, but can be neglected in construction of service class 1 or 2, where technical dried wood members like solid wood, glulam or wood products are used, [8]. The classification and identification of insects is described in detail in [7].

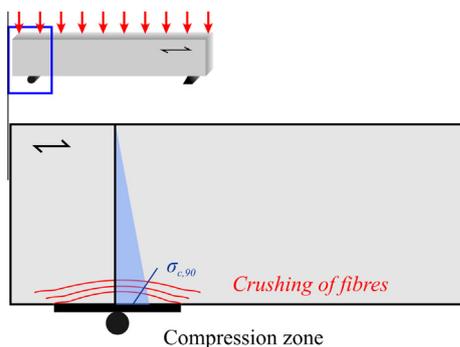


Fig. 8. Principal sketch for compression failure at support.

3. Reinforcement techniques

3.1. General

The following sections illustrate possible reinforcement techniques for timber beams. Detailed descriptions of the different techniques and their design can be found in [9–11]. Reinforcement measures may be required in order to restore the structural capacity of damaged beams or to increase the capacity of intact beams. In the case of damage or decay, the timber beam or parts of the beams have to be replaced as described in Section 3.3. Reinforcement measures to improve the performance of timber beams in bending, shear, and in tension and compression perpendicular to the grain are described.

3.2. Repair of shrinkage cracks or delaminations

Repairs of shrinkage cracks or partial delamination of glue lines in glued laminated timber may be carried out. In many instances, repair of cracks is carried out in conjunction with other reinforcement interventions. For glued laminated timber, it is generally considered necessary to repair shrinkage cracks in regions of high shear stresses and high tensile stresses perpendicular to the grain. The first purpose of the repair is to restore the load carrying capacity of the glulam member. The visible cracks or delamination in glulam members have always to be assessed by an expert before planning the repair process regarding the load carrying capacity of the whole structure, [12].

For crack openings smaller than 10 mm wide and with low to medium fibrosity/splintering the repair can be done by injection of adhesives, [13]. A number of such products with technical approvals for these applications are available. The current regulations according to the requirements in the standards and possible technologies are summarized in [13] for the European market. The methodology of repairing cracks and delamination is shown as well, [13].

To ensure adequate load carrying capacity, the preparation of the bonding surfaces is important to ensure the required quality of the applied technology and to avoid defects. In general, the repair procedure involves cutting out the cracks using a saw, router or grinder to make a clean slot. Before filling with a suitable adhesive, the slot has to be cleaned, optionally brushed with a primer and filling- and ventilation holes have to be prepared. Optional supporting systems for the member may have to be used. Fig. 16 shows the results of applying different technologies for repairing cracks or delamination depending on corresponding adhesives.

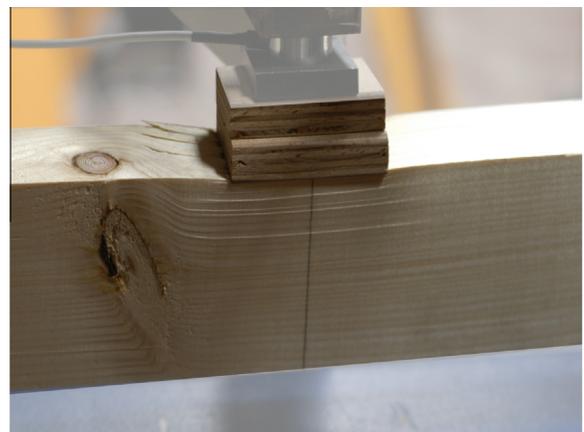


Fig. 9. Compression failure at loading point.

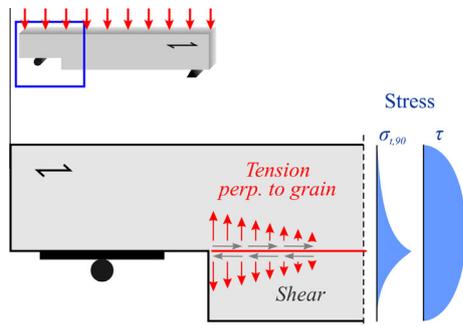


Fig. 10. Principal sketch for tension failure perpendicular to the grain at a notch.

3.3. Replacement of damaged or decayed parts

Timber that has decayed due to fungal or insect attack is porous, fragile and has very poor strength properties [14]. This decay often occurs in localised parts of the beam, such as at the ends where the timber is in direct contact with a masonry supporting wall as seen in Fig. 17. In these cases, the condition of the rest of the beam is generally good. Other types of accidental damage, such as fire damage, may cause a reduction in the member cross-section resulting in inadequate strength and stiffness. Decayed and damaged materials/products should be removed and the member upgraded to restore the load-bearing capacity of the member.

Repair methods include replacing the damaged section with a timber or engineered wood prosthesis connected to the original beam by means of a scarf joint with wooden pegs and/or adhesive (Fig. 18), by means of bonded-in rods or plates (Fig. 19), [16].

The most common retrofit method employed involves replacing the damaged timber with a timber prosthesis which is bonded to the sound timber in the original beam using steel or fibre reinforced polymer (FRP) rods or plates. Using this approach, the scale of the intervention is limited and the load bearing function is preserved. The implementation of this type of repair involves a number of different steps [17]. Initially, the beams are propped. The damaged part of the beam is then removed by cutting either vertically or at an angle of 45° to the vertical, as seen in Fig. 19. Holes or grooves to take the connecting rods/plates are drilled in the beam and the prosthesis and are partially filled with adhesive. The reinforcing elements are inserted into the beam and prosthesis and the adhesive is topped up in holes/grooves if necessary. Additional props are introduced to support the prosthesis. When grooves are used, a timber strip is normally inserted to improve the

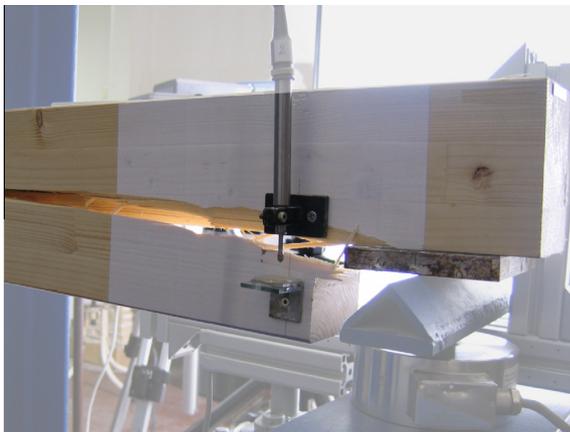


Fig. 11. Tension failure perpendicular to the grain at a notch.

appearance of the repaired member and to provide fire protection. When the adhesive has fully cured, the supporting props are removed.

The prosthesis should be of the same species as the timber to be repaired, or be compatible in terms of its mechanical properties by using, for instance, engineered wood products. The moisture content of the prosthesis should be the same as that of the beam being repaired [16]. The adhesive used is usually a thixotropic epoxy resin and the type used should specially formulated to bond with the timber and the reinforcement. The design of the repair is based on the requirement that the reinforcement should provide the same section load bearing capacity as the section with sound timber.

3.4. Flexural reinforcement

In order to increase the flexural strength and stiffness of beams, reinforcing elements are added that act compositely with the existing member. A large variety of reinforcement configurations are available. The reinforcing elements can be in the form of rods, plates or other structural shapes which are connected to the beam using mechanical fixings or structural adhesives. These reinforcing elements can be placed inside or outside of the member and may be slack or prestressed. The reinforcement material can be a metal, fibre reinforced polymer (FRP) or engineered wood product. Figs. 21 and 22 show some possible configurations for external and internal reinforcement. Apart from the structural requirements, the configuration selected for a particular application may depend on other factors: the presence of decorative ceilings or painting on beams may require that access for reinforcement is restricted to the top or sides of the beam; fire protection requirements may exclude the use of externally bonded plates on exposed surfaces.

As timber beams generally fail in tension in a brittle fashion, positioning of the reinforcement on the tensile face of the beams is very effective for increasing bending strength. With increasing percentage of tensile reinforcement, the neutral axis of the beam moves towards the bottom of the beam. As a result the compressive strain in the timber increases relative to the tensile strain and compressive yielding may occur before the timber eventually fails in tension. The load–deflection response for a timber beam reinforced with carbon fibre reinforced polymer (CFRP) plates, which were inserted from the top, is shown in Fig. 23, [19]. The unreinforced beam A has a brittle response. For the reinforced beams, two of the beams display significant ductility in their response before failure.

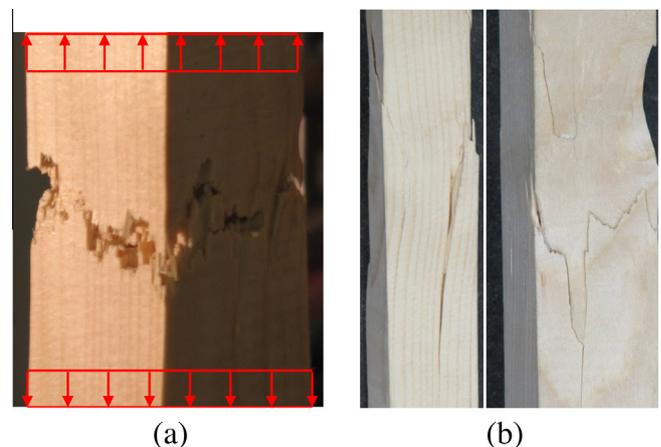


Fig. 12. Tension failure parallel to the grain of experimental tension tests: (a) short-fibred, (b) long-fibred, [4].

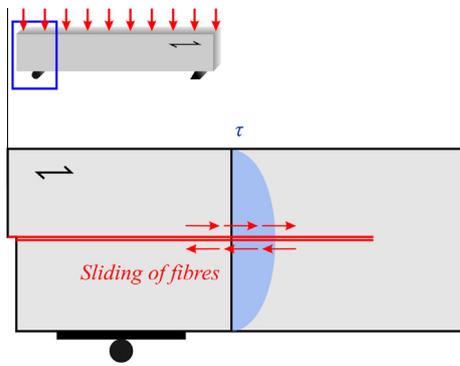


Fig. 13. Principal sketch for shear failure.

Kliger et al. [20] investigated the influence of the distribution of the reinforcement between the tensile and compressive faces of the beam on the bending strength and stiffness. They concluded that for maximum strength, 75% of the reinforcement should be on the bottom face and 25% on the top. To achieve maximum ductility, all of the reinforcement should be placed on the bottom. The maximum stiffness enhancement was achieved when the reinforcement was equally distributed between the top and bottom faces. However, for low percentages of reinforcement the stiffness gain by distributing the reinforcement between the two faces may not justify the additional work involved.

Steel and other metals have been used for reinforcing timber for many years. Mark [21] bonded aluminium sheets to the top and bottom faces of timber beams and reported an increase in the flexural strength and stiffness. Dziuba [22] tested timber beams reinforced with steel rods on the tension face and noted that compressive yielding occurred prior to failure in tension. DeLuca and Murano [23] reinforced spruce beams with 0.82% steel bars and recorded mean increases of 48% in peak load, and 26% in stiffness. Nielsen and Ellegaard [24] investigated the use of punched metal plate connectors as flexural reinforcement for timber but with limited success.

Fibre reinforced polymer in the form of pultruded rods or plates have been the subject of a considerable amount of research for the reinforcement of timber and have been used in practise to reinforce solid timber and glulam structures. Several fibre types are available including carbon (CFRP), aramid (AFRP), glass (GFRP), basalt (BFRP) and steel (SFRP). CFRP [19], [20], [25–27] and GFRP [28–31] have been widely used as externally bonded plates or internally as near surface mounted reinforcement bonded into



Fig. 14. Shear failure at holes.

grooves cut into the beams. CFRP materials have high strength and stiffness properties and, depending on the properties of the unreinforced beam and the percentage of reinforcement used, strength and stiffness increases of over 100% can be achieved. For lower grade timber, less expensive GFRP materials are generally sufficient to provide the required strengthening but the stiffness increase can be limited. Steel fibre reinforced polymer bars have been found to provide a significant increase in capacity and ductility but insignificant improvement in stiffness [32]. The use of FRP materials has a number of advantages over steel due to their light weight, their corrosion resistance, and their ease of handling on site. It should be noted that the routing of grooves to house reinforcement may cause a weakening of the beam as a discontinuity is introduced in the wood fibres in the vicinity of the grooves.

In the design of flexural reinforcement, full composite action between the reinforcement and the timber substrate is assumed. The increase in flexural stiffness and ultimate moment capacity of the reinforced member can then be determined using a classical strength of materials approach as described in [11].

Prestressed steel or FRP plates bonded on the tension face with epoxy resin [33–38] can provide further increases in strength. A pre-camber is introduced in the beam due to the eccentric prestress, which can be offset against the deflection to the external loads. However, this technique is currently not used in practise due to the difficulty in installation and insufficient knowledge of the long-term performance of the prestressed members.

As the flexural capacity of the beam is enhanced, the shear capacity may be exceeded. In these cases, a combination of both flexural and shear strengthening may be required.

3.5. Reinforcement in tension perpendicular to the grain

Failure in tension perpendicular to the grain in timber beams can arise in notched beams, around holes and in curved, tapered or pitched cambered beams. Reinforcement of beams in these situations can be achieved using internal or external reinforcement. Types of internal reinforcement include self-tapping screws, bonded-in or drilled-in threaded steel rods or bonded-in FRP rods or tubes. External reinforcement is achieved by mechanically fixing and/or gluing on sheets of wood-based panels, such as plywood, or FRP sheets or nail plates.

For the case of notched end beams, the stress concentration at the corner of the notch leads to crack initiation and rapid crack propagation results in a sudden brittle failure of the beam as shown in Fig. 12. The high tensile stresses perpendicular to the grain are accompanied by high shear stresses. Different reinforcement methods are illustrated in Fig. 24. The reinforcement can be deployed perpendicular to or at 45° to the beam axis. Due to the presence of high shear stresses, the performance of notched beams reinforced at 45° is superior. This has been validated by a number of experimental investigations [39–42]. Reinforced

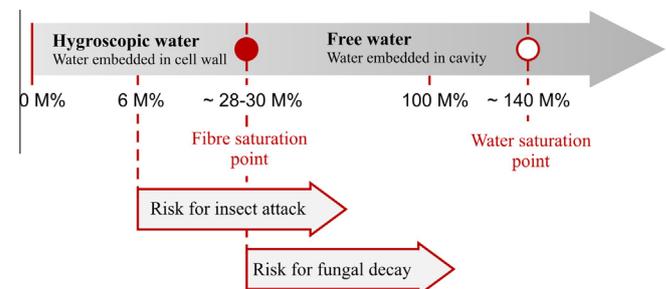


Fig. 15. Risk of insect and fungal decay in relation of the moisture content.



Fig. 16. Result of different repair methods (pouring and injection) for different adhesives, CSA Resin n'H Lungern (left), Purbond RE 3040, RE 3064 (middle), Jowat 692.30 (right), [13].



Fig. 17. Beam end decay [15].

notches have enhanced load-bearing capacity but also display less brittle failure modes than is the case for unreinforced notches.

For screws or glued-in rods, the requirements for minimum edge distances and spacing must be satisfied while keeping the reinforcement as close as possible to the notch corner [9,10]. Externally bonded sheets of FRP or plywood are placed on both sides of the beam and extend over the full height. Screws are normally used to provide the required bonding pressure while the adhesive is curing.

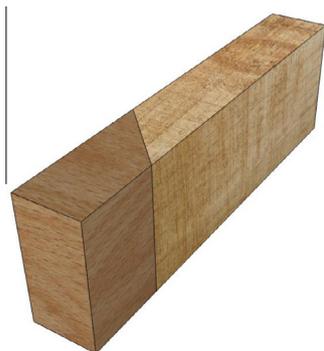


Fig. 18. Prosthesis with scarf connection.

Irrespective of the type of reinforcement used, the usual design approach is to assume that the tensile forces perpendicular to the grain are carried entirely by the reinforcement. For beams with a rectangular notch at the support, the tensile force for which the reinforcement is designed is given by [39,43]

$$F_{t,90,d} = 1.3 V_d [3(1 - \alpha)^2 - 2(1 - \alpha)^3] \quad (1)$$

where V_d is the design shear force and α is the ratio of the reduced beam height at the notch to the total beam height.

For the case of beams with a round or rectangular hole,

$$F_{t,90,d} = \frac{V_d \cdot h_d}{4h} \left[3 - \frac{h_d^2}{h^2} \right] + 0.008 \cdot \frac{M_d}{h_r} \quad (2)$$

where V_d and M_d are the design values of the shear force and bending moment at the section, respectively, h is the beam height, h_d is the hole height and h_r is the distance from the edge of the hole to the top or bottom of the beam [39,43]. Typical reinforcing configurations for beams with holes are shown in Fig. 25.

For curved, pitched tapered or tapered beams, the tensile stress perpendicular to the grain occurs in the apex region, which is highlighted in grey in Fig. 25, [44–46]. Reinforcement of this region can be achieved through the use of screws, glued-in rods or side plates, as shown in Fig. 26.

The design tensile stress, $\sigma_{t,90,d}$, may be calculated as

$$\sigma_{t,90,d} = k_p \frac{6M_{ap,d}}{b \cdot h_{ap}^2} \quad (3)$$

where $M_{ap,d}$ is the design moment at the apex, h_{ap} is the depth of the beam at the apex and k_p is a function of the taper angle, the radius and the depth at the apex [47]. The load to be carried by discrete

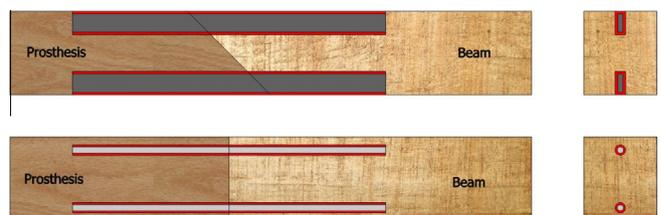


Fig. 19. Beam end repair using bonded in rods or plates.



Fig. 20. GFRP rods inserted in ends of propped beams [18].

connectors, such as screws or glued in rods, is the total of all tensile stresses on an area equal to the connector spacing by the beam width. The capacity of the connectors is determined by the withdrawal capacity and the tensile strength.

The reduced section of the beam due to drilling of holes for the radial reinforcement must be considered in the design. As the portion of the holes below the neutral axis cannot be considered effective in tension, the section modulus in bending is reduced [46].

3.6. Shear reinforcement

The methods available to strengthen beams in shear are the same as those described for reinforcement against tension perpendicular to the grain. These include internal reinforcement in the form of screws and bonded-in rods of steel or FRP and external reinforcement in the form of side plates.

Akbiyik et al. [48] investigated the shear reinforcement of timber stringers with horizontal splits using hex bolts, lag screws, and plywood and GFRP side plates. The bolts were epoxy bonded in vertical holes drilled from the top. The lag screws were installed vertically and at 45°. The plywood and GFRP side plates were attached to the sides of the beams using screws. All repair types were effective with an average increase in the residual shear capacity of 62%. None of the repaired specimens recovered the original undamaged stiffness. The extent of the existing damage had a big influence of the effectiveness of the repair. For the more highly damaged stringers, the use of GFRP side plates was the most efficient method.

Several investigators examined FRP shear reinforcement of beams [42,48–51]. Radford [49] reported an increase in stiffness of over 270% when using epoxy bonded side plates of GFRP with the fibres oriented at $\pm 45^\circ$ to the beam axis. The use of vertical GFRP shear spikes produced a stiffness enhancement of over 160%. Inserting the shear spikes at a spacing equal to the beam depth was found to be the most effective. Gentry [51] also used a combination of FRP flexural plate and FRP shear pins to reinforce glulam. Svecova and Eden [52] used GFRP bars to reinforce beams from a bridge. This resulted in a significant increase in strength and decrease in variability.

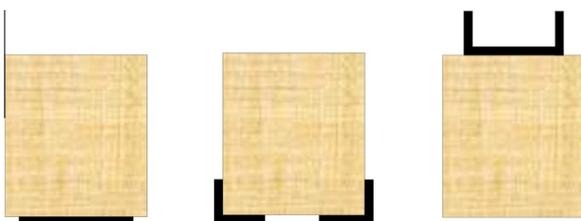


Fig. 21. External reinforcement arrangements.

Widmann et al. [42] investigated the shear reinforcement of delaminated glulam beams. Glulam beams with delaminated middle lamellae were loaded to failure and then reinforced with self-tapping screws or epoxy bonded CFRP side sheets oriented at 45° to the beam axis. Both approaches showed a significant increase in nominal shear strength of the beams. The ultimate shear strength could not be determined as different failure modes were found.

Trautz and Koi [53] described a series of tests performed on glued-laminated beams reinforced with screws using different arrangements to carry tensile and compressive forces. The shear stiffness beams reinforced with screws arranged in a nested pattern with screws carrying loads in tension and compression was superior to that achieved by reinforcing with diagonal tension screws only.

Dietsch et al. [54] describe design approaches for the shear reinforcement of timber beams in the unfractured and fractured states. The types of reinforcement considered are self-tapping screws or threaded rods deployed at an angle to the beam. The models account for the enhancement in shear performance resulting from compression induced perpendicular to the grain by the reinforcement. Comparison with results of experimental tests provided the validation of the shear stiffness and strength predictions. For unfractured beams, an increase in capacity of 20% is achievable when reinforced with threaded rods. For fractured beams, the maximum increase in bending stress compared to the intact beam is 33%.

3.7. Reinforcement in compression perpendicular to the grain

Crocetti et al. [55] undertook experimental investigations of the compressive strength perpendicular to the grain of glulam beams reinforced internally with glued-in steel rods and glued-in hardwood dowels and externally with screwed-on steel side plates. The beams reinforced with glued-in rods all failed in buckling and resulted in a significant increase in both strength and stiffness over the unreinforced beams. The beams reinforced with side plates also produced a significant enhancement in the compression strength.

Blass and Bejtka [39] proposed a design model for the compressive capacity, $R_{90,d}$, of a beam support reinforced using self-tapping screws that accounts for buckling and screw withdrawal. This may be expressed as

$$R_{90,d} = \min \left\{ \begin{array}{l} nR_d + k_{c,90} \cdot l_{ef,1} \cdot b \cdot f_{c,90,d} \\ b \cdot l_{ef,2} \cdot f_{c,90,d} \end{array} \right\} \quad (4)$$

where n is the number of screws, R_d is the lesser of the withdrawal capacity and the buckling capacity of the screw, $f_{c,90,d}$ is the design value of the compressive strength perpendicular to the grain [47], b is the beam width, $l_{ef,1}$ and $l_{ef,2}$ are the lengths as defined in Fig. 27, and $k_{c,90}$ is a load distribution coefficient in the range [1;1.75]

In existing structures, the insertion of screws or glued-in rods at locations of concentrated loading may be difficult to achieve.

3.8. General remarks

Reinforcement methods for the most commonly occurring situations have been described. Other cases, such as tensile and compressive failure parallel to the grain, have not been discussed here. In practical terms, the choice of reinforcement method for existing timber beams will be based not only on the ability of the reinforcement to provide adequate strengthening of the structure but will be constrained by other factors such as aesthetics, need for reversibility, access for repair, and available expertise.

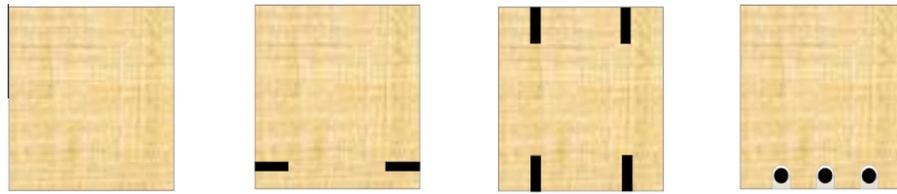


Fig. 22. Internal reinforcement arrangements.

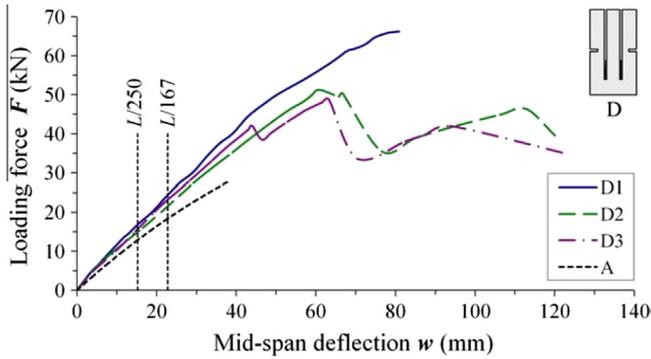


Fig. 23. Load–deflection response of unreinforced (A) and reinforced (D1, D2, D3) beams, [19].

As the reinforcing elements generally have different stiffness, thermal expansion and moisture absorption properties than the timber element, factors such as constrained shrinkage and swelling due to thermal or moisture changes must be considered and if necessary additional thermal or moisture induced stresses should be accounted for in design. Agreed methods for determining these stresses are not currently available. Where the reinforcement results in a significant change in beam stiffness, it is important to consider the consequences for the overall behaviour and load distribution of the entire structure.

4. Case studies

4.1. Clyne Castle, Wales – replacement of decayed parts [18]

Clyne Castle is a Grade II listed building near Swansea in Wales, which was originally built in 1791 but which has had numerous

annexes added over the intervening years resulting in a complex roof structure. Failure of the roof valley drainage system has resulted in prolonged exposure of the roof structure to moisture leading to wet rot in the span beams, hip rafters and ceiling joists.

An upgrade procedure was required that did not interfere with the ornate suspended ceiling. The solution that was adopted was to replace the decayed timber in-situ with a prosthesis made from laminated veneer lumber (LVL). Due to the restriction on access, the connection between the prosthesis and the hip rafters was effected using slots routed in the sides of the prosthesis and the rafter into which six 16 mm diameter high tensile steel rods were bonded using a two-part epoxy structural adhesive. Fig. 28 shows the prosthesis in place with the rods inserted in the slots prior to topping up with adhesive.

The span beams were repaired using an LVL prosthesis that was attached to the beams using six 20 mm diameter steel rods. The rods were bonded into holes drilled into the end grain of the beams and into matching side slots in the prosthesis, as shown in Fig. 29. Due to the lighter loading carried by the ceiling joists, it was sufficient to use a C24 softwood timber prosthesis that was bonded to the joists using two 12 mm GFRP rods. Fig. 20 shows the rods inserted in the joists before adding the prosthesis.

4.2. Sins Bridge, Switzerland – flexural reinforcement [56,57]

Sins Bridge is a historic two-span timber arch bridge over Reuss River at Sins in Switzerland. It was originally built in 1807 and the eastern side was rebuilt after being blown up during the 1852 Civil War. It comprises two equal spans of 30.8 m and was designed for horse-drawn carriages. In 1992, it was upgraded to carry 20 tonne vehicles. This involved the installation of a new 200 mm thick transversely prestressed timber deck and the strengthening of two transverse cross-beams with CFRP laminates.

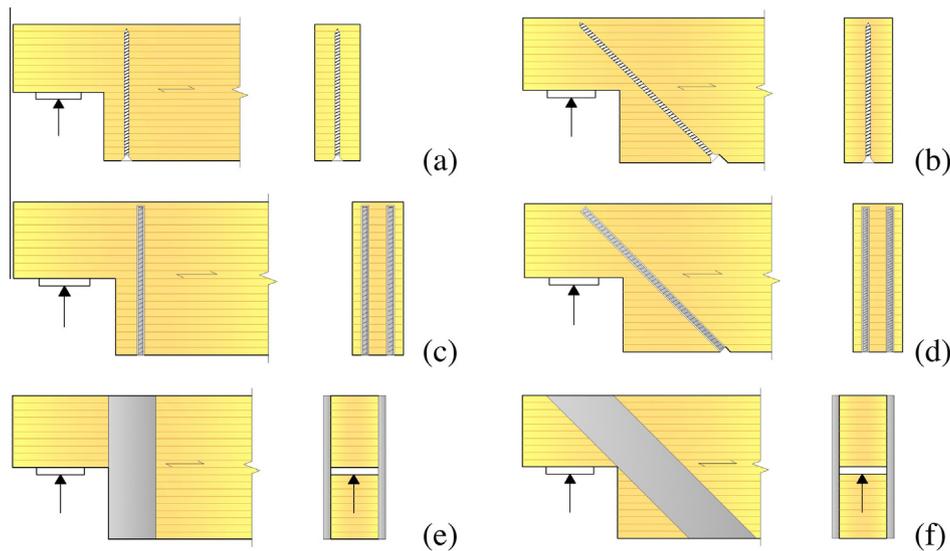


Fig. 24. Typical reinforcement arrangements for notches. (a) & (b) self-tapping screws, (c) & (d) glued-in rods, (e) & (f) EWP or FRP side plates.

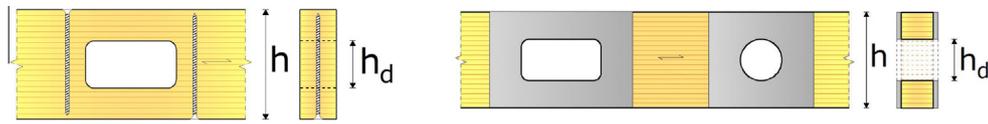


Fig. 25. Reinforcement of openings.

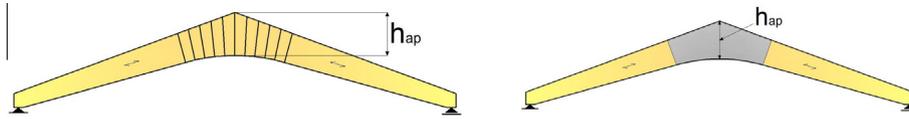


Fig. 26. Reinforcement of pitched camber beams.

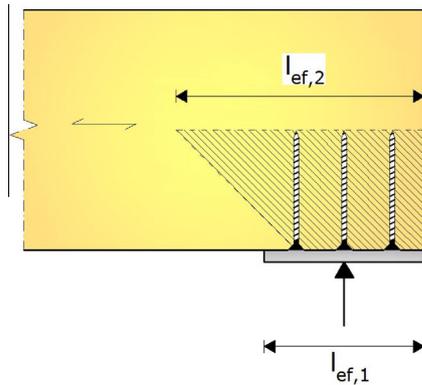


Fig. 27. Reinforcement for compression perpendicular to the grain.



Fig. 29. Span beam repair [18].



Fig. 28. Hip rafter repair [18].



Fig. 30. Long-term monitoring of Sins Bridge, [57].

The cross-beams comprised two solid oak beams placed one on top of the other. The 1 mm thick CFRP laminates were bonded to the top and bottom surfaces using epoxy resin. Two types of CFRP were used: One was a high modulus material ($E = 305$ GPa, tensile strength = 2300 MPa) and the other was a high strength material ($E = 152$ GPa, tensile strength = 2600 MPa).

The reinforced cross-beams and a number of unreinforced crossbeams were instrumented with electrical resistance strain gauges and Demec gauges in order to monitor their long-term performance (Fig. 30). The deflection of the reinforced cross-beams was 20–50% lower than the unreinforced beams. Fourteen years after the original installation, the performance of the reinforced beams continued to be satisfactory [56].

5. Conclusion

Due to the impact of different aspects like moisture changes, fungi and insect attacks, timber beam elements can be damaged and resulting in lower capacity and larger deformations. High stresses exceeding the strength limits can also lead to different types of failure cases, like bending, compression, tension or shear failure. Furthermore, changes in building use can lead to a requirement for increased load bearing capacity in structural timber beam elements. The analyses of several assessment reports showed, most damaged structural timber elements present cracks in the grain direction due to any of the aforementioned cases. In all of these

circumstances, retrofitting and reinforcement of the beams can extend the life of the structure.

The paper describes available reinforcement methods to repair or enhance the structural performance of timber beams. The choice of this reinforcement method will be based not only on the ability of the reinforcement to provide adequate strengthening of the structure but will be constrained by other factors such as aesthetics, need for reversibility, access for repair, and available expertise. Approaches to the design of the reinforcement are presented where available.

While some aspects are well understood, additional research is required particularly related to the long-term behaviour of reinforced beams, and the influence of fluctuating temperature and moisture conditions on the performance. In the latter case, different thermal expansion and moisture absorption properties of the timber element and the reinforcement may lead to additional thermal or moisture induced stresses. Guidance on how these stresses may be quantified for design purposes should form the basis of future research.

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