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Abstract

The research and development of connecting and strengthening timber structural elements with glued-in rods (GiR) has been ongoing since the 1980s. Despite many successful applications in practice, agreement regarding design criteria has not been reached. This state-of-the-art review summarises results from both research and practical applications regarding connections and reinforcement with GiR. The review considers manufacturing methods, mechanisms and parameters governing the performance and strength of GiR, theoretical approaches to estimate their load-bearing capacity and existing design recommendations.

Keywords

Reinforcement, steel rod, FRP rod, design, application, adhesive, Eurocode 5, quality control, linear elastic fracture mechanics, non-linear elastic fracture mechanics

1. Introduction

Glued-in rods (GiR) are an effective way of producing stiff, high-capacity connections in timber structures. In addition GiR have been successfully used for almost 30 years for in-situ repair and strengthening of structures, as well as for new construction works. GiR are used for column foundations, moment-resisting connections in beams and frame corners, as shear connectors and for strengthening structural elements when extensively loaded perpendicular to grain and in shear. Early examples of their use also include the connection of windmill blades made from glued laminated timber (glulam) [1, 2]. Most applications have used the GiR connections/reinforcement with metal bars glued into softwood. In practice, glulam made from softwood in combination with rods with metric threads is the most commonly used combination.

Immense experience exists in the repair and strengthening of beams made of solid timber, both softwood and hardwood, and in connecting concrete slabs to floor beams. For applications where corrosion or weight of the structure could be of concern, the use of pultruded FRP rods is quite common. Some investigations have also aimed at the use of reinforcing bars (rebar), e.g. [3, 4].

All known types of adhesives applicable for wood bonding have been trialled for GiR, but one and two-component epoxies, polyurethane (PUR) and resorcinol types are those most frequently used in practice. Specific adhesive products have been formulated to fulfil the needs of GiR connections/reinforcement with timber which offer much better performance with respect to strength. A large number of parameters impact the strength of GiR [5]. Hence, the challenge is to adequately account for these in design and to provide quality control measures to guarantee a reliable load bearing behaviour of GiR, which are usually assigned high loads by the designer.

2. Reinforcement of structural elements with GiR

Key deficiencies of timber in terms of comparably low tensile and compressive strength perpendicular to the grain as well as moderate shear strength can be overcome by strengthening the timber with GiR in zones subjected to excessive stress. Examples are notched beams or beams with holes, curved or tapered beams and contact zones / supports with high compression stresses perpendicular to the grain (Fig. 1). Due to their availability in different lengths and their high stiffness, GiR are an efficient tool in strengthening of timber structures. Since, however, their application in practice is quite demanding (see chapter 2), self-tapping screws are often preferred by designers [Ref:"Reinforcement with self-tapping screws" by Dietsch P. and Brandner R. in this SI of CONBUILDMAT]), particularly for existing structures..

Reinforcing of timber structures is considered an important topic. Hence, as part of the active development of EN 1995, one working group is exclusively dealing with this topic. Theirwork is based on document CEN/TC 250/ SC 5 N 300 [6] which describes the state-of the art related to reinforcement of timber structures.

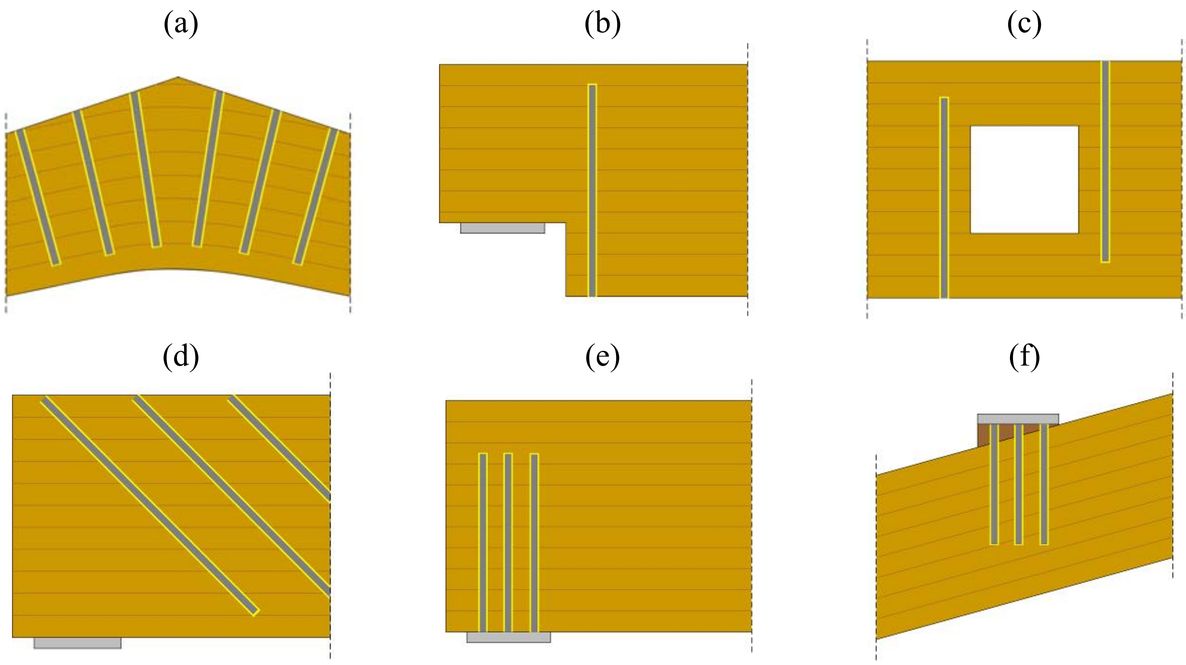


Fig. 1 Application of GiR to strengthen timber structural elements: zones of high tensile stresses perpendicular to the grain in: (a) curved and tapered beams, (b) notched beams, (c) beams with holes, (d) zones of excessive shear stresses, (e, f) compression stresses perpendicular to the grain at supports.

It is important to note that incorporating GiR strengthens elements when overloaded, but will not prevent them from developing cracks due to effects like moisture cycling or non-critical loading!

2.1 Strengthening in tension perpendicular to the grain

Amongst the earliest applications of GiR to strengthen timber structures were members with excessive tension stresses perpendicular to the grain (curved and tapered beams, notched beams, beams with holes) [7], [8], [9]. The GiR reinforcement in these cases prevent the members from early cracking (design of new structures) or stop crack propagation and restore initial load bearing capacity in/of members in existing structures suffering from damage caused by severe cracks [10]. The GiR reinforcement acts like rebar in concrete. Design rules for GiR applied to strengthen members perpendicular to the grain can be found in chapter 6.8 of the German

National Annex to EN 1995-1-1 [11]. According to these rules, glued-in rods with metric thread as well as glued-in profiled rebar can be utilised When designing the reinforcement of notches or holes, tensile strength perpendicular to the grain is not taken into account, i.e. cracking of the structural member is assumed to have taken place already [12].

2.2 Strengthening in shear

The significant impact of crack formation on shear resistance and the desire to prevent the spread of already existing cracks encourages the strengthening of beams. From numerical and experimental studies on shear reinforcement by means of GiR or self-tapping screws [13-18] it can be concluded that GiR (and self-tapping screws) set under an angle of 45° with respect to the beam axis provide an efficient mean of increasing the shear strength of beams. Beams strengthened in shear will reach higher load bearing capacities in bending since early shear failures are prevented. The reinforcing elements also contribute to a considerable increase in flexural stiffness of the beams. For self-tapping screws of types Spax and Würth Assy there are European technical approvals [19, 20] providing a design approach based on research published in [14, 15]. Self-tapping screws provide more ductility and allow for an easy self-setting into the beams compared to GiR, which provide high stiffness but require a higher effort in their application (drilling of holes, centring of rod, gluing).

2.3 Zones of concentrated compression forces perpendicular to the grain

If a designer faces the problem of high compression forces to be transferred to the timber element or from the element to the support, either an adequate area of contact (in order to reduce the compression stresses perpendicular to the grain) or local reinforcement of the timber has to be provided . Such local reinforcement can be achieved by means of self-tapping screws or GiR both of which act similar to pile foundations by transferring the concentrated force along the rod via contact pressure and shear stresses [8, 21].

2.4 Reinforcement in bending

Some researchers successfully applied rods made from steel or from Fibre Reinforced Polymers (FRP) to strengthen beams in zones of excessive bending stresses (e.g. [22-28]). Application of this reinforcement technique in practice may be used in the case of decayed tension face of beams or increased load.

2.5 Moisture induced stresses

When designing reinforcement of timber structures not only the stresses from external loads but also moisture induced stresses (MIS) should be accounted for [29]. MIS can result from changing climatic conditions or from drying of beams with MC higher than that expected on site [30, 31].

3. Application – Gluing-in the rods

3.1 Variants

There are several ways of gluing rods into the wood [32]. Most often, a hole is drilled into the timber member with a diameter that exceeds the nominal diameter of the rod by 1 mm to 4 mm. This results in glue line thicknesses from less than 1 mm up to 2 mm. Thin glue lines are usually preferred over thick glue lines as many adhesives perform better the thinner the glue line is made and the necessary quantity of the expensive adhesive is reduced. In general the holes can be drilled in any direction relative to the grain. An important step after drilling is to clean the hole thoroughly. If pressurised air is used for this purpose it has to be verified that the air is free of oil-dust.

If rods can be set into holes with openings situated at the top of an element an easy variant is to first pour a defined quantity of adhesive into the hole and then to set the rod (Fig. 2(a)). Depending on the viscosity and the open time of the adhesive the rods may sink into the adhesive-filled hole under their own weight or they may have to be pushed into the adhesive filled hole. A disadvantage of this method is that there is no adequate control of the glue line quality in terms of assuring that the adhesive fills all cavities completely and no voids are present in the glue line.

Another often used technique for setting the rod is to drill a second hole, this second hole being drilled perpendicular to the hole drilled for the rod. This hole should lead to the lower end of the rod and thus the adhesive can be injected under pressure from the bottom (Fig. 2(b)). For every rod the injection of adhesive will be continued until it can be observed that the adhesive pours out at the top of the hole that contains the rod or at another hole positioned at the desired location The rod has to be fixed while the adhesive is injected. If the opening between rod and hole is sealed (for example by means of a molded part or super glue), it is also possible to set the rods in a horizontal or overhead configuration as shown in Fig. 2(c) and (d).

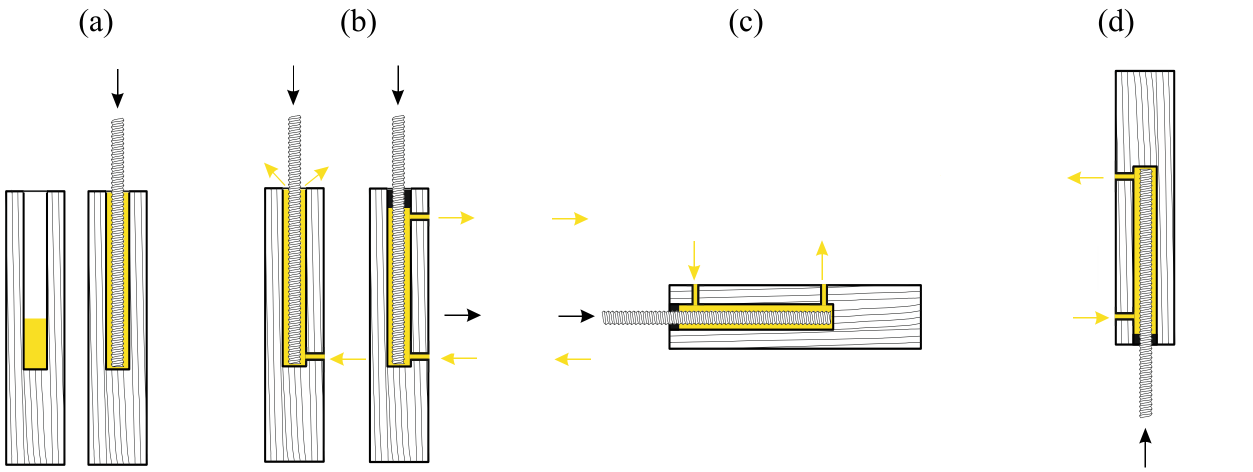


Fig. 2 Variants for the application of GiR.

Other variants of the application of GiR can be found in literature, for exampleusing a concentric continuous hole in the rod for the injection of the adhesive [33] and drilling the rod into an adhesive filled hole with a diameter equal to or smaller than the nominal diameter of the rod. The latter procedure can be regarded as a combination of glued-in and drilled-in rod technology.

However, today these two methods are not of significant importance for practical applications of GiR.

3.2 Quality control

Quality control of the manufacturing process is of great importance. The following parameters have to be checked when GiR connections or reinforcements are applied:

Material

- Timber: strength class, moisture content (MC)
- Adhesive: suitability for gluing in rods, technical specifications, climatic conditions, open time, curing time
- Rod: geometry, type/strength according to design, corrosion resistance, condition of surface (free of oil and/or lubricants)

Application

- Hole: position (including edge and rod distances), diameter, depth, inclination, straightness, cleanliness (Fig. 3a)
- Rod: positioning of rod centrally in the hole (Fig. 3b-d). Depending on glue line thickness the use of spacers and/or centering devices like e.g. plastic or metal rings or a countersink at the bottom of the hole might be required.
- Adhesive: application according to manufacturer specifications, control of filling level, presence of voids (Fig. 3e)

(a) (b) (c) (d) (e)

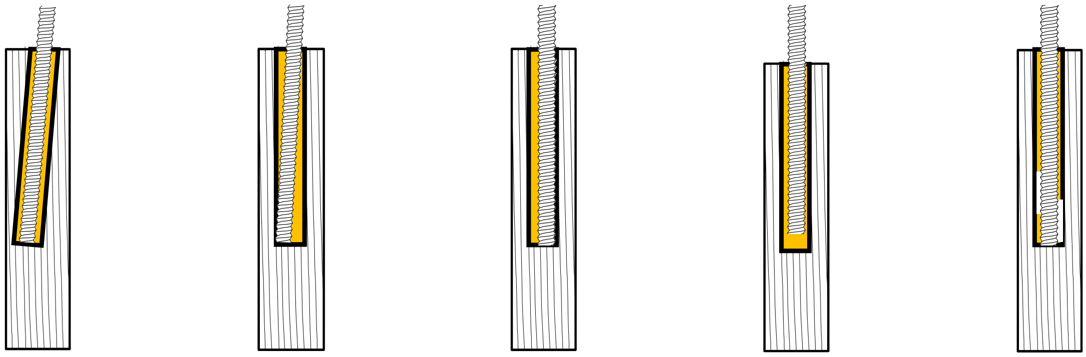


Fig. 3 For optimum performance avoid: (a) unwanted inclination of drilled hole, (b) inclined setting of rod in hole, (c) eccentric position of rod in hole, (d) incomplete insertion of rod in hole, or (e) voids in glue line.

4. Key parameters

Load bearing capacity of GiR connections/reinforcement can be impacted by the following parameters [32] (Fig. 4):

Geometry

- Ratios of area of wood, adhesive area and rod area
- Absolute size of the anchoring zone (represented by hole diameter d_h and anchorage length ℓ)
- Slenderness ratio, which is defined as $\lambda = \ell / d_h$
- Number of rods, edge distances and rod-to-rod distances
- Rod-to-grain angle (including unintentional deviations from planned angle due to production process, definition of a tolerance-range)

Material stiffness

- Moduli of elasticity (MOE) and shear moduli of rod, adhesive and wood

- Ratios of MOE to shear modulus for each material (especially important for the wood material, this being strongly orthotropic)

Material strength

- Strength of the wood (especially shear strength and tensile or compressive strength perpendicular to the grain). Note that the strength of wood is influenced by the density and that solid timber and glulam are usually assigned to strength classes according to EN 338 [34, 35] or EN 14080 [36] respectively. (This also applies to engineered wood products!)
- Cohesive and adhesive strength of the adhesive
- Ultimate strength of the rod material (for steel rods the yield strength is also important)

Fracture mechanical properties of wood and adhesive

- Fracture energy and fracture softening characteristics

Variability of all properties

- Irregularities, i.e. deviation from nominal properties
- Variations in mechanical properties of wood, rod and adhesive

Loading conditions

- Direction of external load on the rod in relation to its axis (pull-out, shearing) and reaction forces on the specimen that counteract the external load in the tests (Fig. 5)
- Load duration (static)
- Number of load cycles, frequency and amplitude (dynamic)

Other parameters

- Wood species

- Special features to reduce stress peaks and/or to guarantee for a ductile failure mode
- Manufacturing practice (curing time and pressure, surface characteristics etc.)
- Quality control.

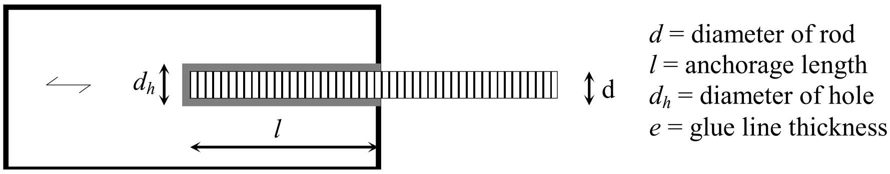


Fig. 4 Parameters in GiR connections / reinforcement.

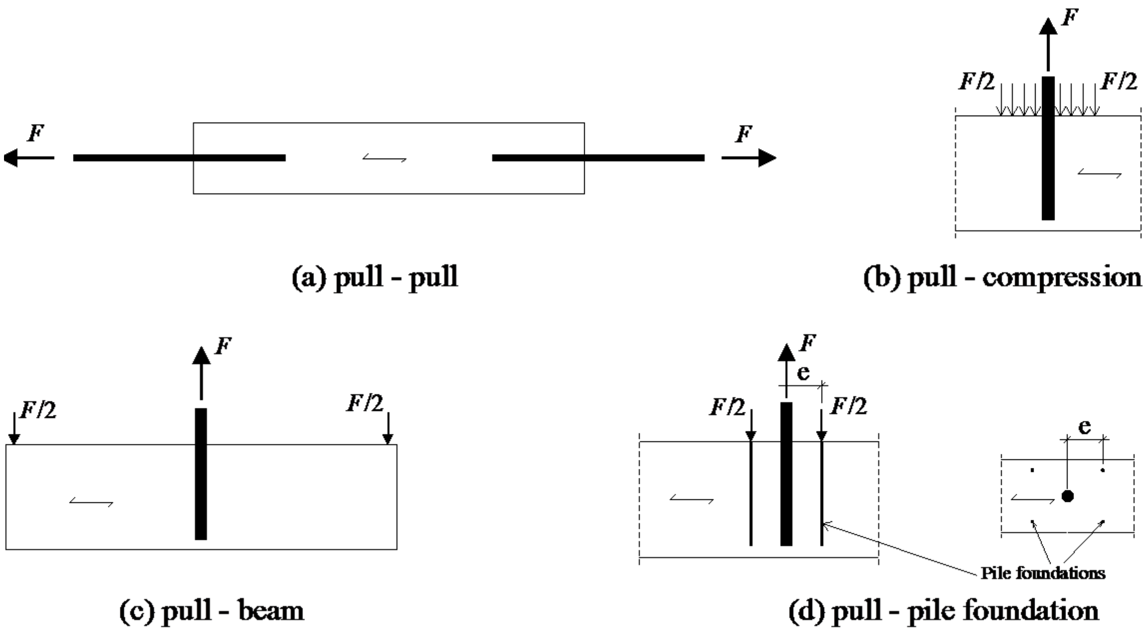


Fig. 5 Different types of loading conditions GiR specimens may be subjected to in tests of axially loaded rods (Figure reproduced from [32, 37]).

5. Adhesives

A variety of adhesives have been tested to glue in rods. In early years, traditional wood adhesives based on phenol-resorcinol (PRF) or epoxies (EPX) were used, while later work has included also the use of polyurethanes (PUR). In 1999, Kemmsies investigated the suitability of 12 different adhesives [38]. In experiments conducted within a large European research project in the late 1990s, (GIROD), three types of adhesives were used and compared [39]: PRF, EPX and PUR. This work concluded that the adhesives revealed increasing strength in pull-out tests in the following order: fibre reinforced PRF, PUR and EPX. EPX adhesives develop a strong bond with both steel and the wood resulting in the wood becoming the weakest link of the connection and thus the fracture properties of the wood or the wood/adhesive interface are decisive for pull-out strength.

Characterising an adhesive only by terms like EPX or PUR is not sufficient. There are many adhesives available of each type and they ‘‘can show all types of constitutive behaviour’’ (regarding EPX: [40]). The pull-out strength of the GiR is obviously related to the adhesive type, but also to the used wood species, since different adherends may develop different bonding strength with different adhesives [41]. Generally speaking, and to a varying degree depending on the specific adhesive used, bond strength can be affected by shrinkage during initial hardening, by the adhesive's sensitivity to elevated temperatures, by its limited gap-filling qualities and by the sensitivity to moisture content changes due to changes in local climatic conditions [41]. These effects have to be taken into account in design [32, 42]. Adhesives for GiR connections must have acceptable creep and creep-rupture properties in addition to good strength and durability. In order to assess these properties tests based on existing methods (e.g. longitudinal shear strength according to EN 302-1) [43] as well as special guidelines (e.g. [44]) have been developed..

The choice of adhesive is not independent of the method used to produce the connections. The main parameters of concern are adhesion to the wood, the mechanical link to the rod (interlocking), the thickness of the glue line and the properties (e.g. viscosity) of the bonding agent [32]. The adhesive should have good gap-filling properties.

For the connections with GiR there are many failure locations and modes which can be critical for load bearing capacity (see 5.3). The adhesive might be chosen during the design of the connection taking into account geometrical properties, requests of application methods and with the aim of avoiding a brittle failure mode to ensure the adhesive bond will not be the weakest link of the connection [45] in order to profit from the full capacity in shear strength that wood offers. In countries like Sweden, UK, Switzerland, Germany [46] and New Zealand [47] the most commonly used adhesives for connections and reinforcement with GiR are 2-component PUR and EPX. When designing connections and reinforcement with GiR it has to be taken into account that most of the adhesives suffer from losing strength at a certain temperature and should allow for curing without additional pressure.

6. Mechanics, failure modes, design philosophy

6.1 Mechanical behaviour of GiR connections

Current knowledge about the mechanical performance of GiR connections is largely based on practical experience and design formulas developed by curve-fitting of empirical data [32]. The majority of studies in this area have focused on axial pull-out strength of a single GiR and its dependency on various material and/or geometrical parameters.

During axial pulling, load transfer between timber and rod is governed by shear of the adhesive. Depending on the strength of the adhesive and the surface characteristics of the rod and its surface treatment, the anchorage between the threaded rod and the adhesive may act as a mechanical connection [48, 49] similar to screws [8, 50]. Some design codes (e.g. [11, 51]) do not allow use of rods lacking a threaded surface since a pure adhesive bond is suspected not to be able to guarantee a reliable and durable force transfer. The force transfer mechanism is also influenced by the ratio of the diameter of the hole to the diameter of the rod, i.e. the bond line thickness. In some sources it is claimed that GiR connections act like a combination of glued and mechanical connections [40, 52, 53]. For rods inserted in undersized holes, it can be expected that the connection strength predominantly results from the mechanical interaction between the wood and the thread of the rod [54].

One major advantage of GiR connections is the transfer of forces directly into the inner part of the members' cross-section [55]. The connection is a hybrid one, made up of three different materials (wood, adhesive, rod) with different stiffness and strength properties [41] which have to work simultaneously under loading. This severely complicates the analysis of the connections and is one of the reasons for today's lack of full understanding of the behaviour of this connection type and agreement on a design model.

6.2 Theoretical approaches to describe the behaviour of the adhesive bond

The adhesive bond line (i.e. the adhesive layer plus the interface between adhesive and adherends) plays a major role in the overall behaviour of the GiR. Different approaches to describe the laws governing the behaviour of adhesive connections can be found in literature: (a) traditional strength analyses, (b) analyses based on linear elastic fracture mechanics (LEFM) and(c) non-linear fracture mechanics (NLFM) analyses [32].

In a traditional strength analysis, stress (and strain) distribution in the GiR for a given loading situation are predicted and then some failure criterion for this distribution are applied. The failure criterion can be based on stress or strain, involving also multi-dimensional criteria. The approach will give a prediction of the load bearing capacity of the GiR, and also a prediction of the stiffness. The stress (and strain) distribution can be determined with analytical or numerical methods, the former e.g. according to the Volkersen theory [56-59].

When using the framework of classical LEFM, the situation of loading a connection with a pre-existing crack is considered. The crack introduces a stress (and strain) singularity, and thus a traditional single point maximum stress criterion is not useful. Instead the crack driving force, also known as the energy release rate, is calculated. The energy release rate is defined as the amount of (elastic) energy released during crack propagation. The critical energy release rate of the connection, G_c, is the amount of energy needed to increase the crack area. By assuming that failure of the connection takes place when the strain energy released is equal to the critical energy release rate of the connection, the load bearing capacity can be calculated [60].

NLFM provides a framework that takes into account not only the strength of the bond line (like in a strength analysis) nor only the fracture energy (like in the LEFM approach), but both ([60]). Consequently, NLFM can be said to include both the framework of traditional strength analysis and LEFM. In traditional strength analysis it is assumed that the strength of the material is limited and that the fracture energy is either zero or infinite, the latter in the case of perfect plasticity. If a crack exists, such traditional strength analyses methods will fail since infinite stress (or strain) will be predicted. The framework of LEFM is, as mentioned above, only be applicable to cases with an assumed pre-existing crack. LEFM assumes finite fracture energy but an infinite strength of the material and a zero size of the fracture process zone. NLFM is one possible way to account for not only a limited strength of the bond line but also a limited fracture energy and a finite size of the fracture process zone. In NLFM this is done by assuming a nonlinear softening behaviour of the bond line. Such bond line behaviour can be implemented in finite element models by the use of e.g. cohesive elements representing the stress-displacement behaviour of the bond line. Thus in what is termed here NLFM, the stress-strain relation used in conventional approaches is exchanged by a non-linear stress-displacement relation. Consequently the bond line, after stress has reached the strength of the material, can still transfer load. This post peak-stress load transferring capacity diminishes with increasing displacement (normal opening or shear slip across the bond line) and will eventually reach zero. Thus, a typical stress versus displacement relation involves both an ascending part (typically the linear elastic response) and a post peak-stress descending part known as strain softening [60]. Such an approach has the benefit of making it possible to perform non-linear analyses without having to assume the existence of a pre-existing crack. Instead, in a single non-linear analysis it is possible to predict the position and load level at which a crack will nucleate and also to predict crack growth accounting for the presence of a fracture process zone of finite size.

The choice of theory to be applied depends on the predicted failure characteristics (brittle or ductile) of the adhesive bond, relative to the properties of the bonding agent, the size and shape of the connection and the stiffness of the adherends [60]. For ductile adhesive bonds stress based approaches can be useful, for very brittle adhesive bonds an approach based on LEFM can be appropriate, and in theory, a NLFM-approach can be used for both these cases and any in-between situation. It must be emphasised that the failure characteristic of the bond line (brittle or ductile) depends on material (strength and stiffness of timber, type and strength of adhesive), geometry (surface and thickness of bond line) and loading conditions.

As regards NLFM, it should be mentioned that apart from rather elaborate nonlinear finite element approaches analytical approaches have also been proposed for analysis of connections with GiR following further developments of the Volkersen theory, and taking into consideration NLFM. A broad description of available theories and the historical development of them are available in [40].

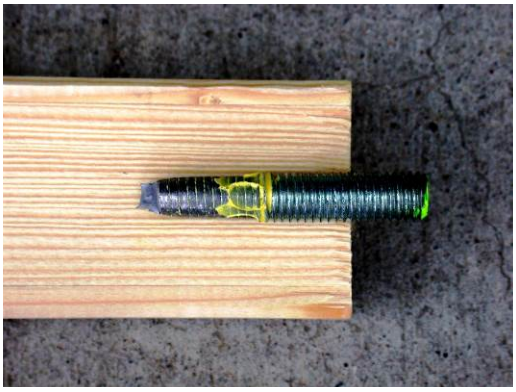
6.3 Failure modes

The GiR connection acts like a chain consisting of the links “rod”, “adhesive” and “wood” [35], the load bearing capacity and failure mode is influenced by the parameters listed in chapter 3. The following failure modes are relevant for a single rod (Fig. 5a-g). Although such connections are of little interest in practice, they form the basis for research and the design of groups of rods.

1. *Failure of the rod due to*
    a. material failure (e. g. yielding of steel)
    b. buckling of the rod in case of compression loading
2. *Pull-out of the rod due to*
    a. adhesive failure at the steel-adhesive interface (in case of lack of rods without profiled surface)
    b. cohesive failure in the adhesive
    c. adhesive failure at the wood-adhesive interface
    d. cohesive failure in the wood close to the bond line
3. *Pull-out of wood plug*
4. *Splitting failure of the wood due to*
    a. short edge distances
    b. the rod being not set perfectly parallel to the grain
    c. excessive perpendicular to the grain loading
5. *Tensile failure in the net or gross wood cross-section*

In addition to these failure modes for single-rod connections, the following are of interest for multiple rod connections:

6. *Splitting failure due to short rod-to-rod distance*
7. *Group pull-out (Fig. 6h)*

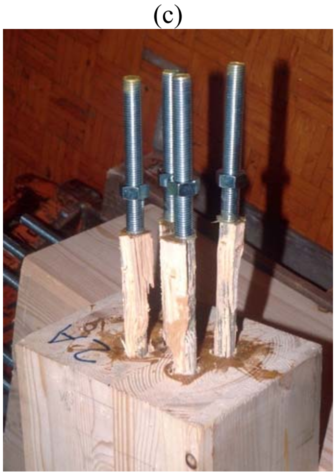


(a)

(b)

(c)

(d)



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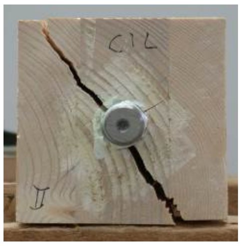
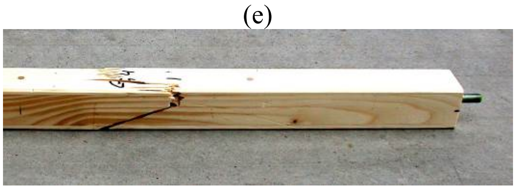


(f)

(h)



(g)

Fig. 6 Different failure modes of GiR: (a) Failure of the rod: (b) pull-out of the rod due to adhesive failure at the steel-adhesive interface, (c) adhesive failure at the wood-adhesive interface, (d)cohesive failure in the wood close to the bond line, (e) pull-out of wood-plug, (f) splitting failure of the wood, (g);tensile failure in the net or gross wood cross-section, (h) group pull-out.

Splitting due to shrinkage or excessive shear stresses and especially due to the stress peaks that are typically formed at the end of the rod [8, 32, 57] can be prevented by transversely reinforcing the connection, e.g. by means of self-tapping screws or threaded steel bars glued into drilled holes [61] crossing potential crack lines, approximately 50 mm from the end of the member [62]. Other possibilities to overcome the peaks in the shear stress distribution are to countersink the drill hole or to widen its diameter at the face end [37]. In references [4, 63] it is suggested to shift the anchorage zone to the inner part of the member (i.e. away from the surface) by either applying no adhesive at the face end of the drill hole or by turning off the thread of the bar over a certain length in order to prevent indentation and shear force transfer there. Successful experiments with widened bottom parts of the drill hole which allow the adhesive to spread in bulbs are reported in [64].

Since moisture induced stresses increase the risk of splitting, the application of GiR is usually restricted to service classes (SC) 1 and 2 (for a definition of SC see: [65]).

6.4 Design philosophy

Dependant on the design philosophy each of the aforementioned links can be considered to be the weakest. Whilst it is straightforward to calculate the tensile strength of the rod in cases where the material quality is clearly defined and is not influenced by excessive variations, the load bearing capacity in the wood, the adhesive and in the interfaces is more difficult to estimate. In practice, the failure load for each of the failure modes must be assessed and the design philosophy set in order that a chosen failure mode can be ensured or prevented respectively. It has to be clearly differentiated between experimental investigations and guidance for safe design in practice. In the first case the GiR are designed such that the wood is the weakest link (in order to identify the maximum load bearing capacity of the GiR being subject of investigation). In the second case assigning the rod to be the weakest link allows for ductility and robustness.

Several design approaches have been suggested [32]. One approach is to ensure that a connection fails in a ductile failure mode, such as by failure in the steel, which must allow large plastic strains to develop with constant or monotonically increasing load capacity until final collapse [63, 66]. Some design codes (e.g. the Swiss design code SIA 265:2012 [49]) prescribe this type of ductile failure, which is favourable for any design case, regardless of materials in use and regardless of the possibility of seismic actions. In case of multiple rod connections it is of even greater importance to aim for a ductile failure mode. Only when the steel rods are the weakest link a uniform distribution of the load among all rods is possible [63]. Plastic deformations in the steel rod can develop only if there is sufficient free length for elongation. To achieve this, a part of the rod near the surface of the timber should be left unbonded [2, 4, 48, 67, 68] and necked down to a slightly smaller diameter by turning off the thread where possible [4, 67]. This helps to prevent mechanical interlocking in this particular part of the anchorage zone and to force plastic deformations to develop in this zone [4, 63, 69]. With respect to ductility there is certainly an advantage in using mild steel with large yield capacity. For GiR connections in high strength timber like beech or ash rods of quality 8.8 may be indicated. This is also the case when (in experimental investigations) pull-out failures are to be achieved in order to derive the optimal anchorage length, to check performance of a specific adhesive or to study the influence of parameters like wood density or shear strength of the wood.

It is worthwhile mentioning that no matter what failure mode is intended the engineer has to be able to assess all of the above failure modes in order to perform the design [32]. The adhesive used, shall not be the weakest link because this would not allow utilisation of the full capacity the glued-in rod connection provides. Therefore there is no contradiction in performing large test series intended to assess the pull-out strength of GiR, even if the practising engineer would rather choose a failure mode based on plastic failure taking place in the rod.

In order to optimise performance of GiR connections: (1) the transfer of stresses should be steady, (2) deviations between force and grain direction should be small, (3) both the rod(s) and the timber should have similar stiffness (i.e. $E_{Timber} \cdot A_{Timber} = E_{Rod} \cdot A_{Rod}$, which in case of steel rods results in $A_{Timber} \approx 16 \text{ to } 20 \cdot A_{Steel}$) and (4) the deformation in rod and timber should be in similar range and not exceed the ultimate deformation capacity (2 to 3 % for Norway spruce) [63, 67].

7. Design of GiR connections

7.1 Background

Despite many national research projects, European projects, COST Actions (e.g. E13, E34) and constant practical application of GiR over the past 25 years there is still no universal standard for their design [70, 71]. This problem originates from the many different design approaches available in the literature for defining the behaviour of the adhesive connections and the fact that a large number of parameters impact the design. The following review of design approaches focuses on work mainly carried out in Europe but also considers New Zealand design guidelines [62] since these are well documented and provide valuable information about specific problems which are not included or missing in European standards (e.g. design rules for multiple rod connections).

An early design approach was published in 1988 by Riberholt [72], who proposed an equation for the estimation of the pull-out strength of an axially loaded single GiR. In the 1990s a considerable amount of experimental work was done resulting in the presentation of several different design methods (see below). Certain design methods were introduced into national design standards and in 1997 a proposal was included in the pre-standard prEN 1995-2 [73]. Although not being exclusively related to the design of timber bridges, the design rules for GiR were included in part 2 of EN 1995 since, at that time work on prEN 1995-1-1 had already been finalised and it was not possible to amend this part of prEN 1995. In 1998, the European GIROD project was launched. The main objective of this project was to establish design rules and the project result was a new calculation model based on the generalized Volkersen theory (GIROD Project Report 2002, [74]). This resulted in a proposal to be implemented in the pre-standard prEN 1995-2, Annex C [75]. During the CEN/TC 250/SC 5 meeting in 2003 it was decided to discard the Annex C. Delegates argued that the proposed code text did not meet the actual status of research (e.g.[76], [77], [78]). Recently both past and current research has been considered with the purpose to propose a design approach that could replace several national design rules. Proposals and design rules developed during the years are shown in Fig. 7.

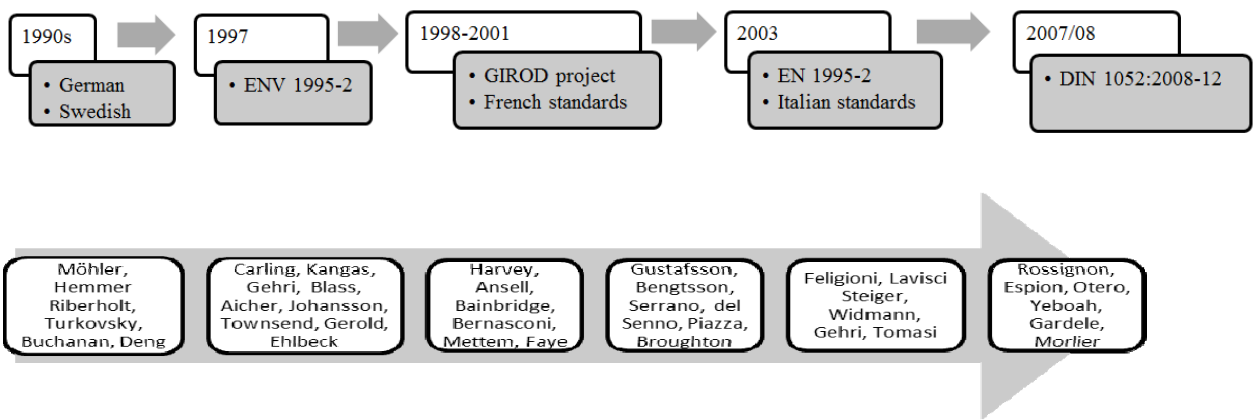


Fig. 7 Standards and proposals containing design rules to estimate the pull-out strength of GiR and researchers involved in the development in the last 25 years.

A calculation model must take into account all relevant parameters that impact the load bearing capacity of glued-in rods (see chapter 4). Although there are numerous studies and calculation methods, and although in an earlier version of EN 1995 design methods exists, the basic problem is still which method to accept and to implement in EN 1995. It is clear that a lack of a common European design approach is a serious obstacle to the widespread uptake of the GiR connection [70].

For more than ten years many research efforts and research programs have contributed to the knowledge about GiR and attempted to provide the information required to prepare design rules which would allow an increased, more advanced and more reliable use of GiR in timber structures [79]. Stepinac et al. [80] carried out a survey on the practical use of GiR and problems the designer faces when designing this connection. Results were as expected: Available design rules were characterised as unreliable and unsatisfying. The most commonly applied design approaches were those in prEN 1995-2, Annex C [75] and in DIN 1052 [51]. Key reservations with the available design rules were found to be [80]:

- Definition of rod spacing and edge distances are not reliable for rods under tension and shear load
- Design rules (and requirements in rod spacing and edge distances) often are too conservative
- Ductility should be treated as a key issue
- There are no reliable rules for multiple rod connections
- The duration of load (DOL) effect is not accounted for
- There are no design rules for the case of interacting axial load and transverse load
- The influence of load-to-grain angle is not addressed
- Some of the available design approaches contain non user-friendly formulae and/or parameters which are difficult to assess

7.2 Comparison of design rules

Since substantial research has been carried out dealing exclusively with pull-out of single rods most of the available design equations are focused only on the pull-out strength of single axially loaded GiR. In sections 6.3 and 6.4 calculation models for rods set perpendicular to the grain and rules for multiple rods are introduced briefly. In this section rules commonly applied for the design of GiR are compared. Diagrams in this Section in general show graphs on characteristic level, except when stated in the caption of the respective Figure.

7.2.1 Axially loaded single GiR parallel to the grain

Tlustochowicz et al. [32] and Stepinac et al. [80] explained in detail proposals and design rules published in the last 25 years. In this manuscript six design rules and methods which are most commonly applied are analysed and explained in detail. Parameters related to geometrical and material properties have been defined in Fig. 4.

Riberholt equation, 1998 [72]: $R_{ax,k} = f_{w1} \cdot \rho_k \cdot d \cdot l_g$ (1)

GIROD equation, 2003 [74]: $P_f = \tau_f \cdot \pi \cdot d \cdot l \cdot (\tan\omega / \omega)$ (2)

prEN 1995-2, 2003 [75]: $R_{ax,k} = \pi \cdot d_{equ} \cdot l_a \cdot f_{ax,k} \cdot (\tan\omega)/\omega$ (3)

Proposal by Gehri, Steiger, Widmann, 2007 [69]: $F_{ax,mean} = f_{v,0,mean} \cdot \pi \cdot d_h \cdot l$ (4)

New Zealand Design Guide, 2007 [62]:

$$Q_k = 6{,}73 \cdot k_b \cdot k_e \cdot k_m \cdot (l/d)^{0{,}86} \cdot (d/20)^{1{,}62} \cdot (h/d)^{0{,}5} \cdot (e/d)^{0{,}5} \quad (5)$$

DIN 1052:2008 [51] and CNR DT 206/2007 [81]: $R_{ax,d} = \pi \cdot d \cdot l_{ad} \cdot f_{k1,d}$ (6)

where:

$R_{ax,k}$ / P_f / Q_k	characteristic value of axial resistance [N], [kN]
$R_{ax,d}$	design value of axial strength [N], [kN]
$F_{ax,mean}$	mean value axial resistance [N], [kN]
l / l_a / l_g / l_{ad}	glued-in length / effective anchorage length [mm]
d	nominal diameter of the rod [mm]

d_h / h diameter of the drill hole [mm]

d_{equ} equivalent diameter [mm]

e edge distance [mm]

k_b / k_m / k_e bar type factor / moisture factor / epoxy factor

ω stiffness ratio of the connection

ρ_k characteristic value of density [kg/m^3]

τ_f local shear strength of the bond line [N/mm^2]

f_{w1} / $f_{v,\alpha,k}$ / $f_{v,k}$ / $f_{ax,k}$ / $f_{k1,d}$ strength parameter / characteristic value of the shear strength of the wood / design value of the shear strength of wood across the grain / characteristic value of the shear strength of the wood at the angle between the rod and grain direction / design value of the bond line strength [N/mm^2]

$f_{v,0,mean}$ nominal shear strength parallel to the grain of a single axially loaded rod [N/mm^2].

Pull-out strength depends primarily on the interfacial layer and shear strength parameter which is influenced by mechanical and geometrical properties of the three component materials. Hence, a simplified calculation model for axial loading could be similar to that for screws:

$$R_{ax,k} = \pi \cdot d \cdot l \cdot f_{v,k} \tag{7}$$

where:

$R_{ax,k}$ characteristic value of pull-out strength

l anchorage length

d diameter

$f_{v,k}$ shear strength parameter.

The mechanics of GiR are complex, so any attempted simplification from the designer's point of view would be helpful in making the design of GiR straightforward but may however result in uneconomic connection design. A closer look at the simplified equation reveals several unanswered questions such as: Which diameter (diameter of rod, diameter of hole or equivalent diameter) and anchorage length (length of bonded rod or equivalent anchorage length) to use? Can the geometry of the hole be described by the slenderness ratio $\lambda = \ell / d$? Which parameters must be included in the shear strength parameter (timber density, MC of timber, MOE of timber, rod and adhesive, rod surface, rod material, type of adhesive, slenderness ratio, geometrical factors, etc.)? These points are among the reasons for present standards and proposals differing significantly (Fig. 8 and Fig. 9).

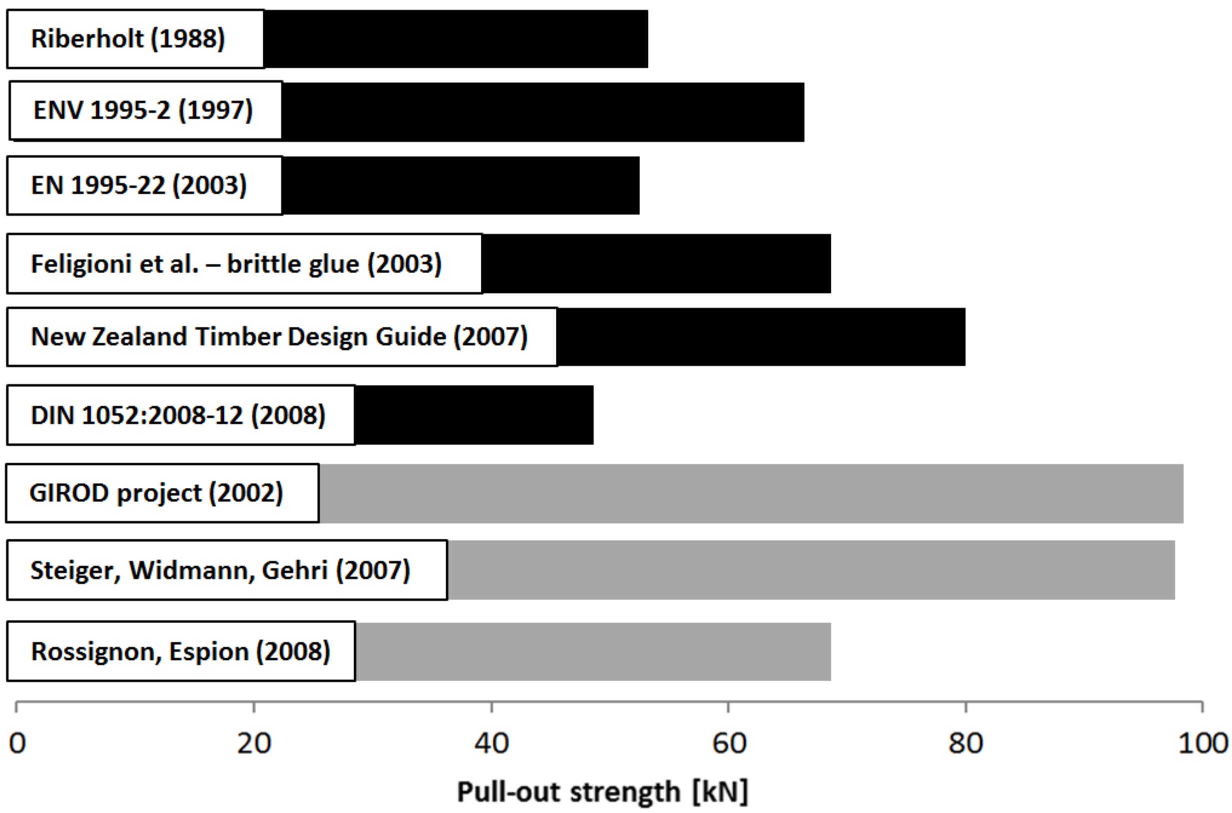


Fig. 8 Comparison of the pull-out strength [kN] derived with different design approaches ([51, 62], [69], [72], [73], [74], [75], [79], [82]), (EPX, l=200 mm, ρ_k=370 kg/m³ (MC<14%),

d=20 mm, e=2 mm). Black bars represent characteristic values; grey bars represent mean values.

From experts discussions it can be concluded that the most common design rules like the ones in prEN 1995-2 [75], the former DIN 1052 [51] are conservative while equations proposed in various scientific papers, in most cases relying on experimental data derived from tests on specific connection systems, deliver much higher values for the pull-out strength. The glue line thickness e is considered only in some formulae. Some standards propose a maximum value of 2 mm [51], [83], [49] but do not provide for design with thinner glue-lines. Differences and the influence on the calculated load bearing capacity are shown in Fig. 9.

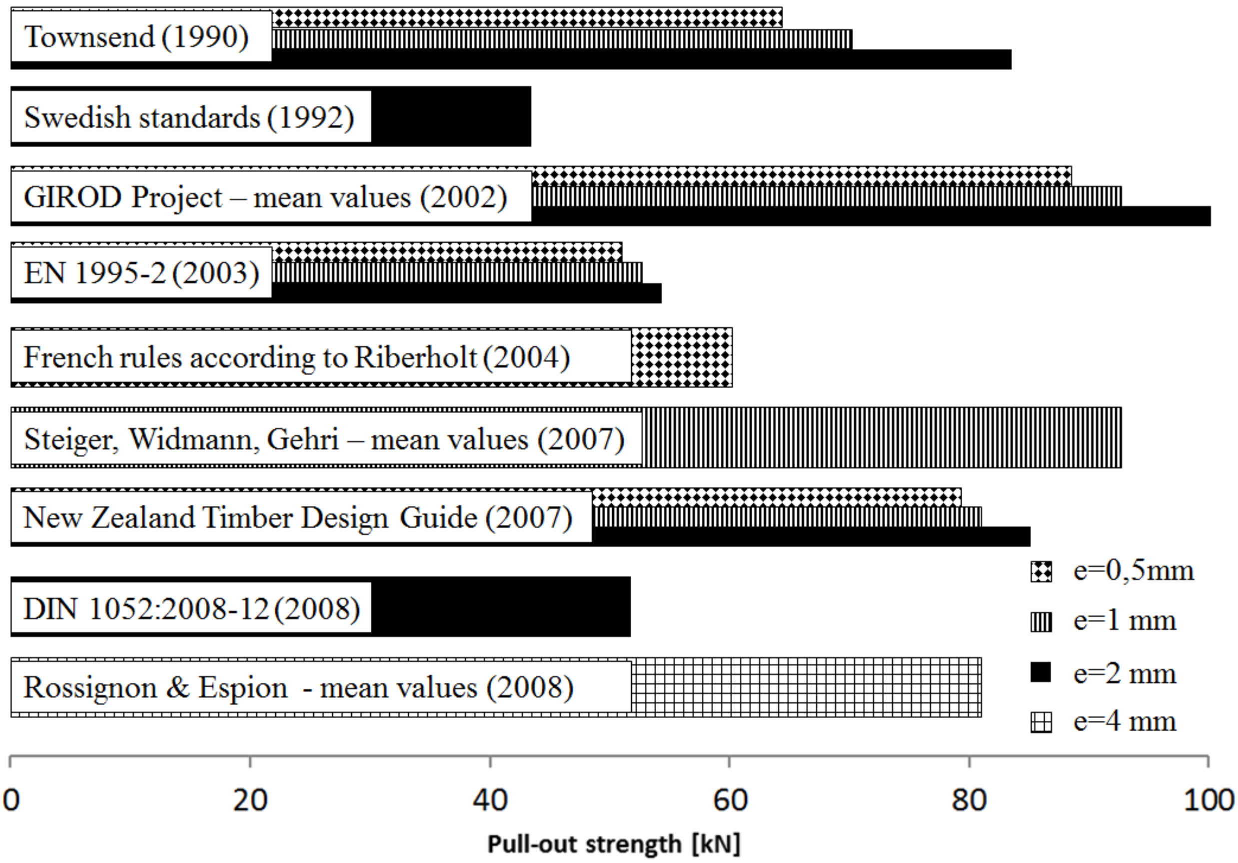


Fig. 9 Influence of glue line thickness on the pull-out strength [kN] (EPX, l=200 mm, ρ_k=370 kg/m^3 (MC<14%), d=20 mm) ([51], [53], [62], [69], [74], [75], [79], [83], [84]).

Fig. 10 and Fig. 11 show the characteristic value of the pull-out strength of one single axially loaded rod estimated with different design rules whereby the diameter of the rod and the anchorage length were varied. Problems occur when defining these two parameters. The diameter *d* is sometimes the diameter of the rod [72], [51], the diameter of the drill hole [69] or an equivalent diameter [85], [82]. A similar problem applies for the definition of the anchorage length. The former prEN 1995-2 equation [75], which was based on the GIROD project findings, included several different parameters. Some of these parameters, e.g. fracture mechanics parameters, cannot be easily determined by engineers in practice.

The influence of wood density has been subject of several studies (e.g. [72], [82], [69], [85]) (Fig. 12). Opinions on the influence of density on the pull-out strength of glued-in rods differ. The recommendations given in [73] for the design of GiR connections indicate that the axial strength of glued-in rods depends on the density of the wooden element. It could be expected that such a relation exists considering that it has been demonstrated that the pull-out strength of nailed and screwed connections is dependent on the density of the wooden member [50, 86-88]. On the other hand, the correlation between density and strength of wood in general is poor [89].

A recent study on the influence of density based on pull-out tests performed on low and high density specimens of Norway spruce glulam [69] demonstrated that the influence of density on pull-out strength of the rods bonded in parallel to grain direction can be quantified by a power function of density ρ^c with the exponent $c_0 = 0{,}55$. The adhesive used in this case was EPX. The further testing of rods glued-in perpendicular to grain [85] revealed less consistent results and therefore it was recommended that the influence of the density of the timber should not be taken into account or to account for it by using an exponent of $c_{90} = 0{,}25$. Bernasconi [90] also reported finding such a relation. However, other studies [91, 92] showed that if such a correlation exists, it is hard to identify.

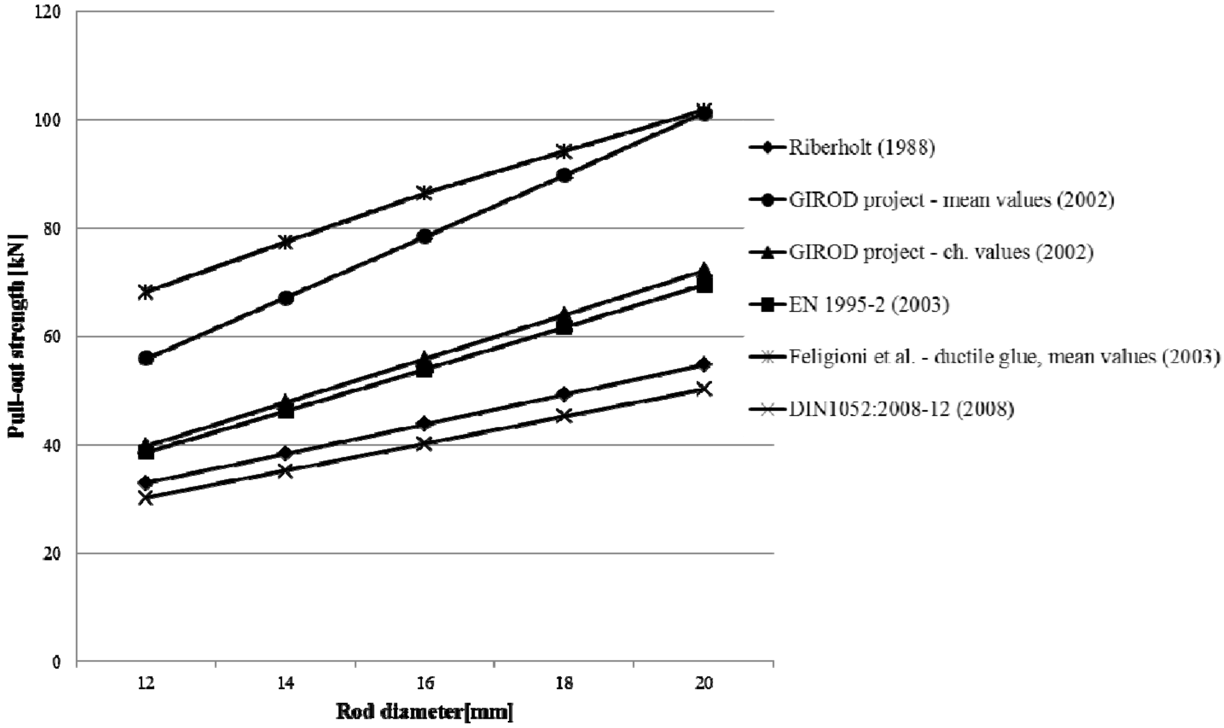


Fig. 10 Comparison of pull-out strength [kN] derived with different design rules ([51], [72], [74], [75], [82]) when varying the diameter of the rod (EPX, l=200 mm, ρ_k=370 kg/m^3, e=2 mm).

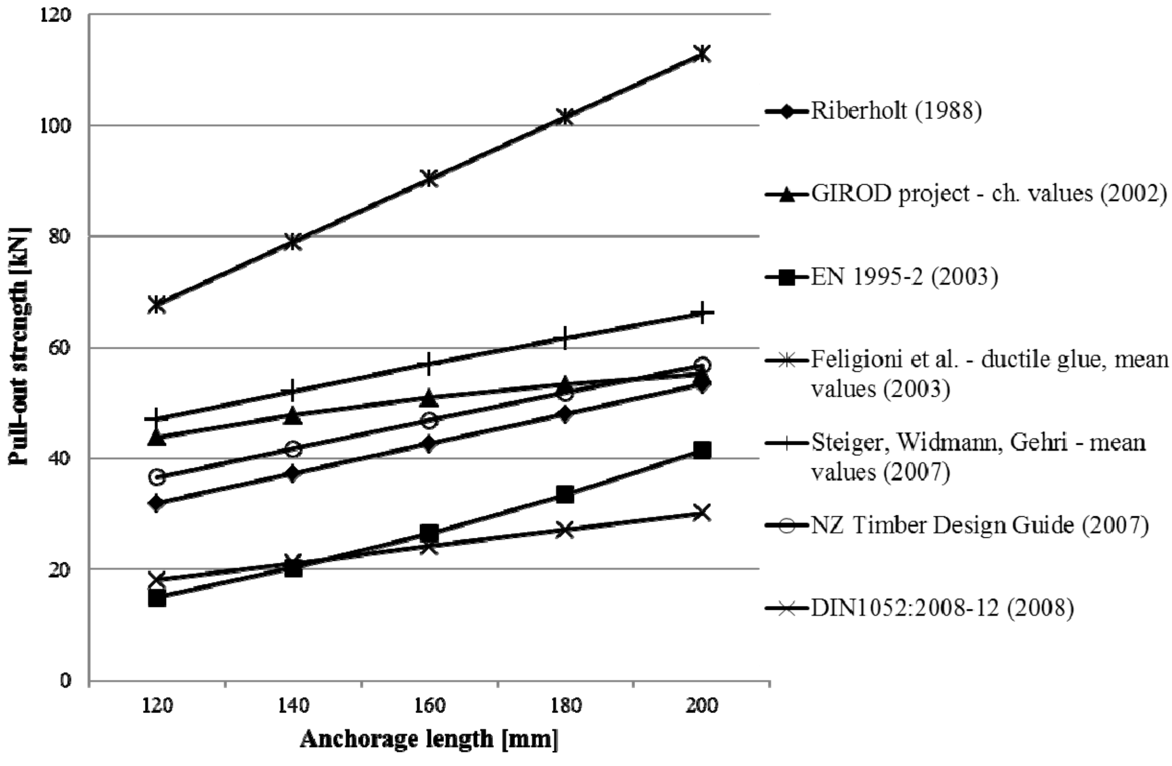


Fig. 11 Comparison of pull-out strength [kN] derived with different design rules when varying the anchorage length ([51], [62], [69], [72], [74], [75], [79]), (EPX, d=12 mm, e=2 mm d=20 mm).

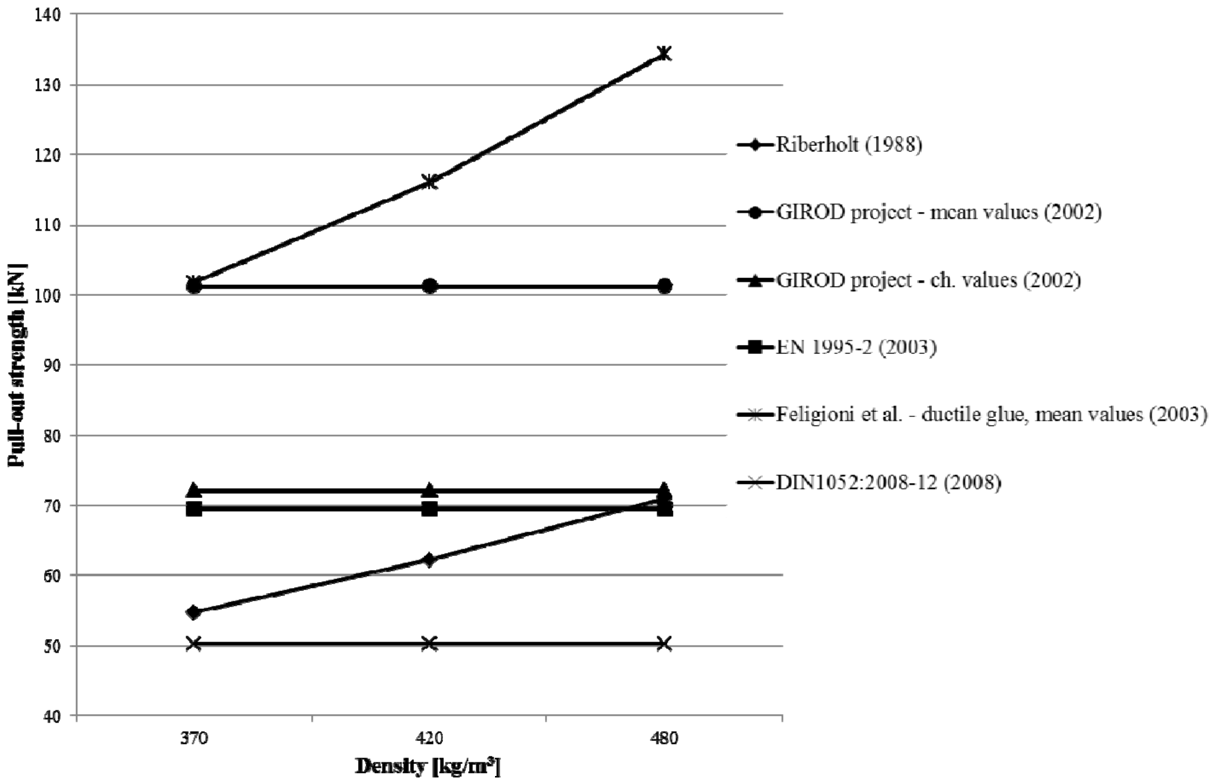


Fig. 12 Comparison of pull-out strength [kN] parallel to the grain derived with different design rules when varying the timber density (EPX, l=200 mm, e=2 mm, d=20 mm) ([51], [72], [74], [75], [82]).

Theoretically, the influence of density is often regarded as a secondary effect, meaning that changing the density changes the value of the parameters in the theoretical expressions for pull-out strength. Thus, an increased density of the wood can influence the load bearing capacity by increased shear strength of the wood, reduced adhesion to the wood, increased stiffness of the wood, etc. Consequently, a number of factors can in part counteract each other. It should be noted that a possible influence of density on the load-bearing capacity of GiR can only be derived from test series where failure occurred in the wood or in the wood/adhesive interface.

7.3 Axially loaded single GiR set in timber perpendicular to the grain

Although most design rules and proposals for pull-out strength of single GiR do not differ whether the rod is set parallel or perpendicular to grain, it is known that the rod-to-grain angle markedly impacts the pull-out strength of GiR. In applications with rods set perpendicular to the grain one of the main parameters is the perpendicular to the grain tensile strength of the timber. Widmann et al. [85], [69] tested and compared specimens set perpendicular and parallel to grain. Rods set perpendicular to the grain achieved higher pull-out strengths than those set parallel to the grain, therefore rod-to-grain angle is regarded as a parameter which cannot be neglected [69]. Blass & Laskewitz [93] proposed a mechanical model of which a simplified version has been implemented in German standards [51]. From their online survey Stepinac et al. [80] concluded that designers are using the same equations for rods set perpendicular and parallel to the grain, or are referring to [85] where the pull-out strength is estimated as follows:

$$F_{ax,mean} = 0{,}045 \cdot A_g^{0,8} \quad \text{with} \quad A_g = l \cdot \pi \cdot d_h \tag{8}$$

l	*anchorage length [mm]*
d_h	*diameter of drill hole [mm]*

7.4 Multiple rod connections

Very little data on the behaviour of multiple GiR connections is available. In a recent study Parida et al. [66] concluded that the use of mild steel as well as more rods of smaller diameter are effective measures to increase the ductility of the connection. In multiple rod connections non-uniform distribution of forces and interference between rods occurs [32]. In prEN 1995-2 [75] there was an equation to estimate the pull-out strength of a group of rods inserted parallel to the grain. This design approach however, was based on failure in the timber element. The characteristic load bearing capacity of one rod $R_{ax,k}$ was taken as:

$$R_{ax,k} = f_{t,0,k} \cdot A_{ef} \quad (9)$$

where: $f_{t,0,k}$ is the characteristic tensile strength of the wood and A_{ef} is the effective timber failure area. This formulation was not accepted as it was characterized as unreliable (e.g. brittleness could lead to progressive failure in multiple rod connections). An easy way to reach a uniform distribution of forces among all rods is to use steel rods and to design the connection such that the steel rods are the weakest link [63].

For multiple rod connections spacing between the rods and edge distances are key issues governing the load bearing capacity of the connection [32]. Blass et al. [94] studied the influence of these parameters for axially GiR and found that load bearing capacity decreased if the edge distance was less than 2.5 times the rod diameter. The results of a study by Broughton et al. [37] also confirmed this, demonstrating how multiple rods spaced too closely do not act individually but instead pull-out as one plug. Edge distances are a crucial factor on the load bearing capacity since insufficient edge distances may cause splitting of the wood [95]. There are some differences in the proposals; more than 2 *d* [72], more than 2.3 *d* [69] however values for minimum edge distances of 2.5 *d* are present in most design equations (Table 1).

Table 1: Edge distances and distances between rods as proposed in different design rules for connections with rods set parallel to the grain.

Design rule	Rods set parallel to the grain: Minimum distances	
	a_1 – between the rods	a_2 – edge distances
Riberholt [72], Deng [48]	1,5*d*	2*d*
prEN 1995-2 [75], CNR DT [81]	4*d*	2,5*d*
GIROD [74], DIN 1052:2008 [51]	5*d*	2,5*d*
French rules [83]	3*d*	2,5*d*
Steiger et al. [69]	4*d*	> 2,3*d*

New Zealand Timber Design Guide [62]	2*d*	1,5*d* (no shear force) 2,5*d*

Rod spacing and edge distances are key parameters regarding not only the prevention of early splitting of the connection or of plug failure in case of multiple rod connections but also the overall performance of a GiR connection. The overall performance is defined in terms of balancing the axial stiffness of the timber and the rods to obtain as uniform stress distribution as possible and in terms of percentage of the load bearing capacity of the timber gross cross-section transferred by the connection. This means that distances between rods as well as edge distances should be fixed such that $E_{Timber} \cdot A_{Timber} = E_{Rod} \cdot A_{Rod}$, which in case of steel rods results in $A_{Timber} \approx 16 \text{ to } 20 \cdot A_{Steel}$ (see 5.4) and such that distances $a_1 = 4d \text{ to } 5d$ and $a_2 = 2.5d$.

According to the provisions in [62] the pull-out strength of a group of GiR must be reduced by a factor k_g for groups of bars (0,8 for 5 or 6 bars in a group, 0,9 for 3 or 4 bars in a group and 1,0 for 1 or 2 bars in a group). European standards provide only information about reduction of pull-out strength of a group of screws, no provision is made for groups of GiR. In Table 2 the respective design equation ($n_{ef} = n^{0,9}$) (from EN 1995-1-1 [65] is compared to the one in the New Zealand Timber Design Guide [62].

Table 2: Effective number of GiR calculated according to the New Zealand Timber Design Guide [62] for GiR and according to EN 1995-1-1 [65] for screws

Number of rods / screws *n*	3	4	5	6
Effective number of rods according to [62] $n_{ef,NZ}$	2,7	3,6	4	4,8
Effective number of screws according to [65] $n_{ef,EN}$	2,7	3,5	4,25	5

7.5 Technical approvals

Neither an EC design approach nor a product standard (EN) for GiR connections is available to date. To account for the specific features incorporated within different systems of GiR, companies offering such systems or adhesives for gluing in rods enabled the practical application of their products/systems by means of technical approvals (TA). Examples include e. g. the WEVO-Spezialharz EP 32 S /B 22 TS [96], the Purbond PUR adhesive CR 421 [97] and the GSA® system [98]. In Germany the Studiengemeinschaft Holzleimbau e.V. holds a technical approval [99] containing general specifications and design rules (referring to the former DIN 1052 standard [51]) for the application of GiR in practice.

Amongst others, the aforementioned product related TAs provide detailed information and relevant data regarding application (service classes, temperatures, type of load), system components (timber, adhesive, rods) and system design (design loads, rod to rod and rod to edge distances). In general the determination of the design loads according to the mentioned TAs is based on the German National Annex to EN 1995 [11] or the preceding standard DIN 1052 [51] (both standards contain identical design approaches). Hence, the basic design equation is similar to equation (7). As a consequence, the design can lead to different results compared to the experimentally derived performance of a connection or reinforcement formed with a particular product or system. The main reason for this is that basic parameters like characteristic values of pull-out strength and/or required rod to rod and rod to edge distances can differ from product to product.

8. Rods made from FRP

8.1 Background

FRPs are composite materials consisting of load bearing fibres held in a polymer matrix that protects the fibres and enables load to be transferred between them. Hence, the strength of an FRP is determined by the strength of the fibrous matrix used. Carbon, glass, aramid or basalt fibres and a thermosetting or thermoplastic polymer such as EPX or perfluoroalkoxy alkane (PFA) [100, 101] can be used.

FRP comes in two forms; unidirectional parallel fibres or layered fabrics. Rods are the former, and are created through a pultrusion process. This is where the fibres are pulled through a resin bath in which they are impregnated with the polymer; they then enter a heated die with a constant cross-section to create the required diameter of rod [102].

Fibre Reinforced Polymers have been used in concrete and masonry structures for many years. The use of FRP in timber dates back to the 1960s where a number of laminated timber structures were reinforced with Glass Fibre Reinforced Polymer (GFRP). The introduction of Carbon Fibre Reinforced Polymer (CFRP) and Aramid Fibre Reinforced Polymer (AFRP) in timber construction [103] first occurred in the 1990s. In the past two decades much work has been done investigating the potential of bonded-in FRP in timber as an alternative to steel rods [42, 103-106].

8.2 Material properties

As Table 3 demonstrates, even the weakest FRP is stronger in tension than steel and they are all of much lower density. Both Basalt Fibre Reinforced Polymer (BFRP) and Glass Fibre Reinforced Polymer (GFRP) have a much lower modulus of elasticity than steel. Therefore when used in timber these FRP should be more compatible with most timbers.

Table 3 Material properties of bar materials [107-112].

Material	**Density** (kg/m^3)	**Tensile strength** (MPa)	**Yield strength** (MPa)	**Elastic modulus** (GPa)	**Cost*** (Euro/m^3)

Steel	7’800	400 – 700	275 – 500	200	6’700
Aramid FRP	1’450	3’000	–	77 – 135	82’000
Basalt FRP	2’700	1’000	–	90	14’000
Carbon FRP	1’500	1’600	–	120 - 300	90’000
Glass FRP	1’800	850	–	46	11’500

* Costs are based on 2008 figures and will vary depending on the bar diameter [108, 112].

The higher strength compared with steel rods allows a lesser equivalent volume to be used to achieve the desired performance. From a cost perspective, both BFRP and GFRP are cost-effective options but BFRP has a higher tensile strength and slightly better corrosion resistance than equivalent GFRP [41, 108, 110].

8.3 Application and design

In GiR using a rods made from FRP, failure will occur in the timber, close to the glue-timber interface, as this is the weakest part in the bond, provided a good bond was achieved in the first place. Adhesives which have good viscosity and gap-filling properties, such as EPX or PFA, should be used to bond rods made from FRP to timber. The timber should be freshly drilled and cleaned out and the FRP abraded and wiped down with a solvent or a peel-ply method used to guarantee a good quality bond.

When designing FRP GiR the orientation of fibres in the FRP should be considered. FRPs are anisotropic materials; they are strong parallel to the direction of their fibres but are weaker perpendicular to them. Therefore load-carrying components should be designed using FRP orientated parallel to the load, and GiR applications that require some flexibility should use fibres perpendicular to loading.

At present there is no guidance for design using FRP in Eurocode 5 however, the Italian design guides [113] have information on using FRP for retrofit and include strengthening in bending, simultaneous bending and axial force, in-plane actions and connections.

8.4 Advantages and disadvantages

Rods made from FRP have a much higher strength-to-weight ratio than steel rods of equivalent diameter; therefore they can be used to produce lightweight structures with equal strength. This also makes them easier to handle and install and reduces transportation costs. FRPs are corrosion resistant and so can be used in harsh environments such as chloride-rich splash zones where steel would be at risk from corrosion. As a result of this corrosion resistance, structures using FRP have a longer service life than when steel is used, with less monitoring and maintenance required and thus reduced expenditure where this is concerned.

The cost of using FRP is higher than steel and this can be a major barrier to their use. As FRPs are not as readily available as steel their manufacturing process is more costly, leading to an overall increase in cost of use. The level of expertise and availability of personnel with such experience and skill is also an issue to be considered. Disposal of waste FRP is another end stage component related to increased costs; as they cannot be separated in to their original components they are very difficult to recycle [114]. However, with time and as more experience is gained about using FRP the cost of using them should decrease and come in to line with those associated with steel. Table 2 also demonstrates that FRP behave in a brittle fashion whereas steel exhibits ductile behaviour, hence FRP not having a yield strength value. However, in cases where a bonded-in rod connection is designed in such a way that failure occurs due to timber shear, the brittle failure mode of the rod is not a critical issue.

9. Conclusions

GiR are an efficient tool in strengthening timber structures suffering from unsufficient strength due to damage or a change in use. There are several GiR systems offering good solutions for the designer. For most of these systems technical approvals containing recommendations for design and application are available. Due to the fact that many parameters impact the performance of GiR connections / reinforcement these have to be regarded as systems, each consisting of unique combinations of timber, rod material, adhesive, geometrical dimensions, setting procedure and quality control. Often connections / reinforcement with GiR are applied where high performance in terms of strength and stiffness is required. In order to provide sufficient robustness to the connection / reinforced structural element subjected to high loads, ductile failure modes are to be preferred and the design strategy should assign the weakest link to an element of the GiR system which provides sufficient ductility.

Despite the timber design codes in some countries (e.g. New Zealand) containing design rules for GiR, such rules still do not exist in the European timber design code EN 1995-1-1. Attempts should be made to develop a design rule for EN 1995 covering all issues and parameters described in the preceding chapters of this state-of-the-art review. Highlighting GiR as an important item in the course of the CEN/TC 250/SC5 work programme for the next five years ("towards a 2nd generation of EN Eurocodes") [115] is a first and critical step in this direction.

One way to untie the "Gordian knot" of conflicting opinions on rules for the design of GiR could be to start from answering the question: "What are the key advantages and what is the potential GiR offers compared to other types of connections/reinforcement and what requirements have to be fulfilled in order to profit best from these advantages/this potential?"

When setting up rules for Europe it has to be recognised that the European system works as a 3-step-pyramid consisting of (1) test standards (containing rules on how to test products), (2) product standards (giving strength and stiffness parameters, boundary conditions and rules for production and quality control) and (3) design codes (providing design equations and formulating specific requirements in e.g. spacing, edge distance, minimum anchorage length, etc.). Since the pyramid will not be complete if one element is missing, drafting rules for GiR connections / reinforcement has to be concentrated on all 3 steps of the pyramid.

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