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Published in:
International Conference on Connections in Timber Engineering - From Research to Standards

Published: 01/01/2017

Document Version
Publisher's PDF, also known as Version of record

Please cite the original version:
Jockwer, R., Fink, G., & Kohler, J. (2017). Assessment of existing safety formats for timber connections - How probabilistic approaches can influence connection design in timber engineering. In R. Brandner, A. Ringhofer, & P. Dietsch (Eds.), *International Conference on Connections in Timber Engineering - From Research to Standards: Proceedings of the Conference of COST Action FP1402, Graz University of Technology, Institute of Timber Engineering and Wood Technology, Graz, Austria, 13.09.2017* (pp. 16-31). Verlag der Technischen Universität Graz.

Assessment of Existing Safety Formats for Timber Connections – How Probabilistic Approaches can Influence Connection Design in Timber Engineering

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Summary

Connections are important details in timber construction, connecting single members and elements to larger structures. The design of connections is regulated by structural standards that in general make use of the so called semi-probabilistic safety concept. This concept contains reliability elements, i.e. a conventional deterministic representation of strength and stiffness related properties and action effects as specified fractile values of the underlying probability distributions, partial factors and load combination factors. Standardisation bodies ascertain the reliability elements in order to provide sufficient reliability for the design solutions that result from the application of the code.

This is also done for the semi-probabilistic design basis for timber connections. However, despite of the fundamental differences in mechanical and material behaviour, in general the same reliability elements as for the design of timber structural components are used.

The present article takes a critical appraisal of the existing safety format for timber connections as implemented in the Eurocode.

1.1 Introduction

Due to the natural origin of the wood the dimensions of timber elements are limited. In order to be able to build larger structures, individual timber elements are connected by means of different types of connections. The types of connections most commonly used in modern timber engineering are amongst others: glued-connections, dowelled, bolted, nailed or stapled connections, connections with screws or glued-in rods. The performance of the above-mentioned connections depends on their applications; e.g. used as shear or tensile connector, type of connecting materials like timber or engineered wood products.

The structural performance of a timber structure is considerably influenced by the performance of the connections between the individual structural members. These connections are often the cause of failure of timber structures [1,2]. Despite their importance timber connection design frameworks are not based on a consistent basis compared to the design regulations of timber structural components.

1.2 Design of Timber Members

For the determination of the load-carrying capacity and for the design of individual timber members their behaviour is characterized by the principal mechanical properties, e.g. the tensile and compression strength of the timber loaded parallel and perpendicular to the grain, respectively, the shear strength and rolling shear strength. The design can be performed by comparison of the acting stresses and the corresponding strength of the members.

1.3 Design of Connections in Timber Structures

The structural performance of single connections depends on different elements with individual material strength and stiffness and individual geometrical properties. Due to this complexity a straight forward comparison of acting stresses and corresponding strength as compared to timber members is hardly possible for the design of connections.

Mechanical models have been developed in order to explain the structural behaviour of connections and in order to handle the variety of possible arrangement of connections in timber structures. Certain material related parameters and system properties are used in the mechanical models that represent a specific performance of the material. These material related parameters or system properties can be determined in material tests or in simplified tests on representative connections or parts of it, respectively. An example of a material related parameter is the tensile strength of steel determined according to EN ISO 6892-1 [3]. An example of a system property is the embedment strength of the timber determined according to EN 383 [4].

One of the challenges for the implementation of mechanical models and provisions for the design of connections in codes is to account for the different characteristic properties of the elements and the different failure modes of a connection. For a reliable design of connections the entire system of the individual members of the connection has to be assessed.

1.4 Ductility Aspects for Design of Timber Structures

The performance of a structure depends not only on its resistance but also on its deformation capacity. Besides elastic deformations of the structure especially the non-linear behaviour of connections is of interest. Especially ductile behaviour of connections offers the potential for redistribution of loads in the structure as shown by [5]. Different design codes like DIN 1052 [6] or SIA 265 [7] set the ductile failure mode of connections as the basis for the design. A detailed discussion of the importance of ductile failure modes in connections can be found in [8,9].

Due to e.g. geometrical constraints it can be necessary to reduce the dimensions of the connections necessary to achieve ductile failures. This seems adequate especially if the desired load-carrying capacity can be obtained, however, the consequences of brittle failures should be minimized by implementing additional measures for guaranteeing sufficient robustness.

2. Safety Concept of Eurocode

2.1 Load and Resistance Factor Design Format

Both, the loads and the resistances are subject to uncertainties. In order to ensure an adequate level of reliability almost all design codes, including the Eurocodes, introduced design values for resistances and actions in the design equations; the so-called load and resistance factor design (LRFD) format.

The optimal partial factors for achieving the desired failure probability can be determined in dependency of the loading situation and the relevant material parameters as discussed in [10]. The calibration of these partial factors for timber structures is based mainly on loading situations of members in pure bending [11].

2.2 Load-Carrying Capacity and Resistance in the Eurocodes

Different formats for representing the design value of the load-carrying capacity can be set up. The design resistance is defined as:

$$R_\mathrm{d} = \frac{R\{X_\mathrm{d}\}}{\gamma_\mathrm{Rd}} \tag{1}$$

where:

X_d design value of the relevant material property;

γ_Rd partial factor accounting for uncertainty in the resistance model; and

$R\{\ \}$ outcome of the resistance model.

The design values of the material property (X_d) that is used in Eq. (1) to verify ultimate limit states should be calculated from:

$$X_\mathrm{d} = \frac{\eta\{X_\mathrm{k}\}}{\gamma_\mathrm{m}} \tag{2}$$

where:

X_k characteristic value of the relevant material property;

η conversion factor that takes account of volume and scale effects, effects of moisture and temperature, and any other relevant parameters; and

γ_m partial factor accounting for uncertainty in the material property.

Besides the separate consideration of the partial factors either on the resistance level or on the material property level a joint consideration of the partial factors related to the material property and to the resistance model could be set up. In this regard the following two equations are considered.

The following formulation is known as the 'material factor approach' (MFA) as given in Equation 6.6a in EC 0.

$$R_d = R\left\{\frac{\eta X_k}{\gamma_M}\right\} \tag{3}$$

where:

$\gamma_M = \gamma_m \cdot \gamma_{Rd}$

Alternatively to Eq. (3), the design resistance may be obtained directly from the characteristic value of a resistance, without explicit determination of design values for the individual material property. This formulation is known as the 'resistance factor approach' (RFA):

$$R_d = \frac{R\{\eta X_k\}}{\gamma_R} \tag{4}$$

where:

$\gamma_R = \gamma_M = \gamma_m \cdot \gamma_{Rd}$

2.3 Application in EN 1995-1-1 [12]

EN 1995-1-1 (Eurocode 5, EC 5) [12] specifies design rules for the use of timber and timber based products in structural design. Timber and most of the derivated building products are complex inhomogeneous materials and it cannot be referred to material properties without reference to the corresponding test conditions in terms of loading mode, size, time, surrounding climate, etc. However, in EC5 [12] the term "material property" is used for simplicity (as in the entire timber engineering profession) as a proxy for the more correct term "properties of standardized test specimen examined under standardized test conditions".

2.3.1 General Definition of the Design Material Property

The design material property as defined in Eq. (2) is applied in EC 5 [12] as follows.

$$X_d = k_{mod} \frac{X_k}{\gamma_M} \tag{5}$$

where:

X_k characteristic value of strength property;

γ_M partial factor for a material property;

k_{mod} modification factor taking into account the effect of the duration of load and moisture content.

The factor k_{mod} is a conversion factor from standardized test load duration and moisture conditions to the anticipated conditions in the structure.

2.3.2 General Definition of the Design Resistance

The design resistance or load-carrying capacity is defined as:

$$R_d = k_{mod} \frac{R_k}{\gamma_M} \tag{6}$$

where:

R_k characteristic value of load-carrying capacity.

The conversion factor is here directly multiplied to the resistance. The partial factor for the partial material property is also used directly on the resistance. The universal use of the partial material factor is illustrated in Tab. 1, where different partial factors are suggested for both, material properties and resistances.

Tab. 1 Recommended partial factors γ_M for material properties and resistances [12].

Fundamental combinations	
Solid Timber	1.30
Glued laminated timber	1.25
LVL, plywood, OSB	1.20
Connections	1.30
Punched metal plate fasteners	1.25
Accidental combinations	1.30

2.4 Design of Connections in Eurocode 5 [12]

As an example the design value of the load-carrying capacity can be derived as follows:

$$F_{v,Rd} = k_{mod} \frac{F_{v,Rk}\left\{f_{h,i,k}; M_{y,Rk}; F_{ax,Rk}\right\}}{\gamma_M} \tag{7}$$

where:

$f_{h,i,k}$ characteristic value of embedment strength in the timber member i;

$M_{y,Rk}$ characteristic value of the yield moment of the fastener;

$F_{ax,Rk}$ characteristic value of the axial withdrawal capacity of the fastener.

An additional factor k_γ is used for certain failure modes in order to account for the different partial factors for the material of timber and steel. The factor can be derived e.g. as follows for the failure modes with plastic hinges in the fastener:

$$k_\gamma = \sqrt{\frac{\gamma_M}{\gamma_{M,steel} \cdot k_{mod}}} = \sqrt{\frac{1.3}{1.1 \cdot 0.9}} = 1.15 \tag{8}$$

2.5 Discussion of the Implementation of the EC 0 Safety Format to the Design of Connections in EC 5 [12]

The design value of the resistance of a connection is calculated in Eurocode 5 [12] by applying the general values of the partial factor γ_M and the modification factor k_{mod} to the characteristic value of the resistance of a connection. This procedure is correct only if (a) the coefficient of variation and (b) the distribution function of the resistance of the connection are the same as assumed for the determination of the general values γ_M. Existing differences in the variation of the resistance are considered currently by the factor k_γ. However, the differences of the distribution functions are not accounted for in the current design format for connections in EC 5 [12].

3. Structural Behaviour of Connections with Metal Dowel-Type Fasteners

The structural behaviour of connections is discussed e.g. in [13,14]. The estimation of the resistance of connections is based on extensive mechanical models that include several material properties. The load-carrying capacity of dowel type fasteners is governed by four main characteristics:

- The embedment strength of the timber f_h. The embedment strength is the system property that is associated to the resistance of solid timber against the lateral penetration of a stiff fastener. Additional properties like dowel geometry or surface roughness have an important impact on the embedment strength.
- The bending moment capacity of the dowel M_y. The bending moment capacity is mainly influenced by the dowel diameter and the yield strength of the dowel material. A plastic deformation capacity is necessary to provide bending moment capacity even after considerable deformation of the dowel.
- The pulling out resistance of the dowel F_{ax}. Under special circumstances the so called pulling out resistance of dowel type fasteners can be activated even in lateral loading. In that case a large bending deformation of the fastener is required. This effect is also referred to as the rope effect. For smooth dowels the rope effect is commonly neglected.
- The resistance against splitting, block or plug shear failure. This resistance is mainly governed by a fracture mechanical phenomena and depends on the spacing, edge and end-distances as well as the member thickness and penetration depth of the fasteners.

In addition to those four main characteristics, also effects such as the effective number of fasteners or the impact of friction between the members due to the rope effect influence the load-carrying capacity. However, they are not considered in the present study.

3.1 Mechanical Models

3.1.1 Fastener Failure: European Yield Model

The resistance of laterally loaded dowel type timber connections is commonly determined as the minimum of the capacities according to the so called European Yield model (EYM) that is based on the studies by Johansen [15]. These failure modes describe the embedment failure of the timber and/or the plastic failure of the dowel in dependency of the thickness t_i of the timber members i (failure modes $R_{\mathrm{I,i}}$ to $R_{\mathrm{III,i}}$ in Fig. 1). The load-carrying capacities of the different failure modes according to the EYM for a single shear plane in a wood-steel-wood connection are given in Eqs. (9)-(11).

Failure mode I: Embedment failure

$$R_{\mathrm{I,i}} = f_{\mathrm{h,i}} t_\mathrm{i} d \tag{9}$$

Failure mode II: Mixed failure with plastic deformation of the dowel in the steel plate

$$R_{\mathrm{II,i}} = t_\mathrm{i} f_{\mathrm{h,i}} d \left[\sqrt{2 + \frac{4M_\mathrm{y}}{f_{\mathrm{h,i}}\, d\, t_\mathrm{i}^2}} - 1 \right] \tag{10}$$

Failure mode III: Failure with plastic deformation of the dowel in the timber member

$$R_{\mathrm{III,i}} = \sqrt{4 M_\mathrm{y} f_{\mathrm{h,i}}\, d} \tag{11}$$

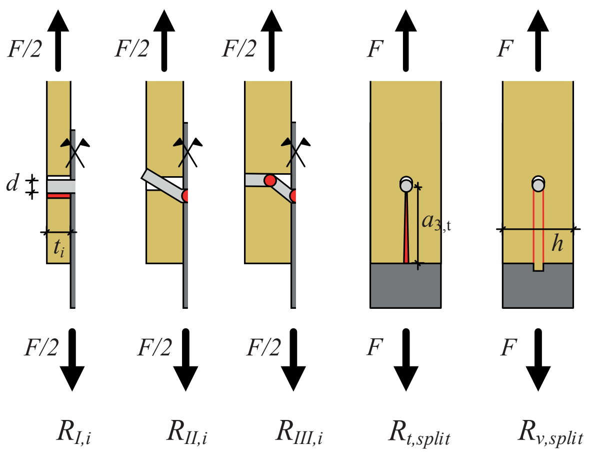

Fig. 1 Simplification of failure modes of the EYM for the symmetric half of a dowelled timber-steel-timber connection and splitting and block shear failure modes.

3.1.2 Timber Failure: Splitting and Block Shear Failure

Failure modes in the timber members are often characterized by brittle failure mechanisms in shear and tension perpendicular to the grain. So far only a design equation for the situation of block shear failure of laterally loaded groups of fasteners is given in the Appendix A of EC 5 [12]. Additional failure modes with tension perpendicular to the grain splitting and shear fracture of the connection as shown for the cases $R_{t,split,i}$ and $R_{v,split,i}$ in Fig. 1 are not accounted for in EC 5 [12]. Brittle failure modes are relevant especially for thin side members of double shear connections and small spacing or end-grain distances.

A very simple model for considering impact of the end-grain distance $a_{3,t}$ can be based on a verification of tension perpendicular to grain strength $f_{t,90}$ (Eq. (12)). The relation between force F_{90} acting perpendicular to the grain induced by a dowel loaded parallel to the grain by force F_0 is $F_{90} \approx 0.3\ F_0$ according to [16].

$$R_{t,split,i} = \frac{1}{0.3} t_i a_{3,t} f_{t,90} \tag{12}$$

The model in Eq. (12) can be used in analogy for describing the impact of spacing a_1 on the fracture in tension perpendicular to the grain.

[17] presented a fracture mechanics based design approach for brittle failure of a connection (Eq. (13)). Due to the complex stress state the fracture process is described by mixed mode fracture with $G_{f,mixed}$. An angle of friction $\varphi = 30°$ between dowel and timber is used by Jorissen.

$$R_{v,split,i} = 2t_i \sqrt{\frac{G_{f,mixed,i} E_{0,i} d \sin\varphi \left(h - d\sin\varphi\right)}{h}} \tag{13}$$

A conservative estimate can be made by assuming the mixed mode fracture energy to be equal to the mode 1 fracture energy with crack opening: $G_{f,mixed} = G_I$. Other more sophisticated fracture mechanics based approaches can be found e.g. in [18].

3.2 Material Properties

The determination of different material property values and their impact on the load-carrying capacity of connections with dowel type fasteners was discussed by [19].

The distribution characteristics of the relevant material property values and a probabilistic assessment of the load-carrying capacity of shear connections with dowels was presented by [13]. In the following the most important characteristics of the material property values are summarized.

3.2.1 Embedment Strength f_h

The distribution characteristics of embedment strength were determined by [20] as summarized in Eq. (14) and Tab. 2.

$$f_h = A\rho^B d^C \varepsilon \tag{14}$$

Tab. 2 Regression parameters for Eq. (14) from [20].

Parameter	Distribution function	Mean value	*stDev*
A	Lognormal	0.097	0.23
B	Normal	1.07	0.04
C	Normal	–0.25	0.012
ε	Lognormal	1	0.11

3.2.2 Yield Moment M_y

The relevant resistance of a fastener in bending is between the elastic and full plastic bending capacity (e.g. [21]). The empirically derived Eq. (15) is given in EC 5 and is based on studies by [22].

$$M_{\mathrm{y}} = 0.3 f_{\mathrm{u}} d^{2.6} \tag{15}$$

The variation of material properties of the steel within one batch is rather small. [23] proposes $CoV \approx 4\%$. In Tab. 3 the yield and tensile strength of common steel grades are summarized. Recent studies by [24] show that there can be a considerable difference between steel qualities of different batches and overstrength is a common issue.

Tab. 3 Yield strength f_y and tensile strength f_u in dependency of steel grades for a CoV = 4% and lognormal distribution properties.

Grade	$f_{y,k}$ [N/mm²]	$f_{u,k}$ [N/mm²]	$f_{u,mean}$ [N/mm²]
S235	≈ 190 – 360	≈ 360 – 510	≈ 385 – 545
5.6	300	500	534
8.8	640	800	854
ETG 100	> 865	≈ 960 – 1100	≈ 1025 – 1175

3.2.3 Additional Material Properties and Correlations

The distribution characteristics of density ρ, modulus of elasticity parallel to the grain E_0 and tension perpendicular to grain strength $f_{t,90}$ can be found in [25]. The mode 1 fracture energy G_I is based on studies by [26]. All distribution characteristics used in this study are summarized in Tab. 4.

Tab. 4 Distribution characteristics of material parameters.

	Distribution function	Mean value	*CoV*
ρ	Lognormal	420	10%
E_0	Lognormal	11500	23%
$f_{t,90}$	Weibull	2.0	30%
G_I	Lognormal	0.3	20%

The correlations between the material property values is based on JCSS [25] (Tab. 5) and [20] (Tab. 6). No correlation is assumed between G_I and the other material properties as discussed in [27] which leads to a larger impact of $R_{v,split}$.

Tab. 5 Correlation between material properties values [25].

	E_0	$f_{t,90}$
ρ	0.6	0.4
E_0	–	0.4

Tab. 6 Correlation between embedment strength parameters [20].

	B	C	ε
A	–0.99	–0.24	0
B	–	0.11	0
C	–	–	0

4. Load-Carrying Capacity of Connections

4.1 Impact of Varying Material Properties on the Load-Carrying Capacity of Connections

The geometrical parameters of relevance for the load-carrying capacity according to EYM are the thickness of the timber member(s) t_i and the dowel diameter d. These geometrical parameters can be expressed by the slenderness $\lambda = t/d$. The steel quality has an impact only on the load-carrying capacity in failure mode II and III. At the transition between the failure modes II to III the critical slenderness $\lambda_{II/III} = t/d$ for achieving ductile failure can be defined. The end-grain distance $a_{3,t}$ of a connection with a single fastener has an impact on the failure mode. For small end-grain distance the splitting failure modes cause a reduction of load-carrying capacity. In the example shown in Fig. 2 in addition to the values specified in Tab. 4 the following material and geometric properties have been chosen: steel quality 5.6, $d = 12$ mm, $h = 10\ d$, $a_{3,t} = 7.5\ d$.

In Fig. 2 the impact of varying material properties on the variability of the relevant load-carrying capacity is shown in dependency of the relative thickness of the side members $\lambda = t_i/d$. For each thickness $n = 10^5$ simulations were performed. It is obvious that with increasing λ the load-carrying capacity is increasing. However, a closer look also indicates that the variability decreases and the shape of the distribution function changes, in particular the lower and most important tale of the distribution function. This is a result of the different types of failure (see also Fig. 1) with the different corresponding strength parameters: For small relative thickness of the side members (approx. $\lambda < 2.5$) about 3/4 of the simulated connections failed in R_I and 1/4 in $R_{t,split}$. For larger relative thickness λ failure mode R_{II} (approx. $2.5 < \lambda < 5.5$) and failure mode R_{III} (approx. $\lambda > 5.5$) become dominant.

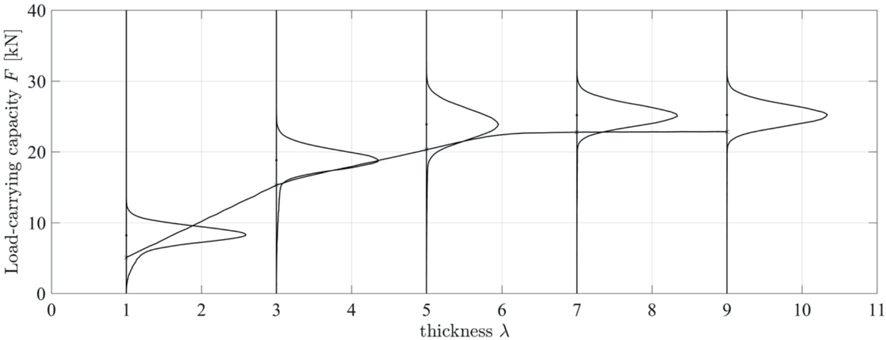

Fig. 2 Load-carrying capacity according to EYM and timber failure modes in dependency of the relative side member thickness λ = t_i/d.

In Fig. 3 the load-carrying capacity normalized with the mean value is illustrated. Comparing the black lines (5 % and 1 % fractile) and the grey lines (95 % and 99 % fractile) the skewness of the distribution becomes obvious. Furthermore, the change of the leading failure modes, for different relative thicknesses, is visible. This change in skewness has to be accounted for when defining and specifying a partial factor for the failure mode. A solution would be the consideration of individual partial factors for the different strength properties.

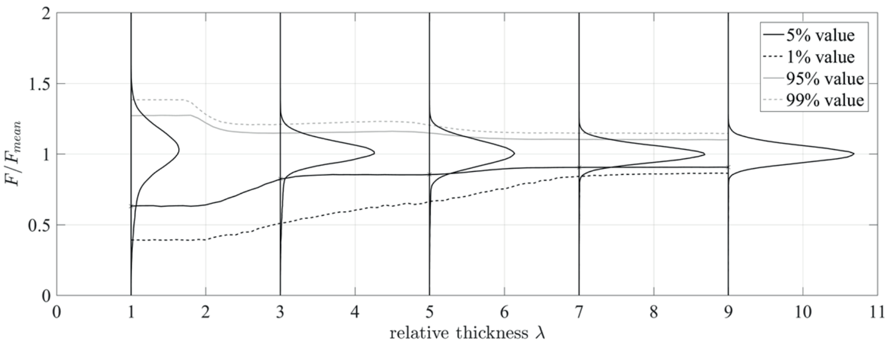

Fig. 3 Normalized load-carrying capacity in dependency of the relative side member thickness λ = t_i/d.

4.2 Variation of Load-Carrying Capacity of Connections in Tests by [17]

[17] reports a large number of tests with various configurations. The tests were carried out as bolted shear connection in timber-to-timber double lap joints. Teflon sheets in the contact areas were used to reduce the impact of friction induced by the

rope effect. [23] confirmed the validity of the fracture mechanics design approach derived by [17] (Eq. (13)) for the load-carrying capacity of a single dowel of small slenderness $\lambda = d/t$.

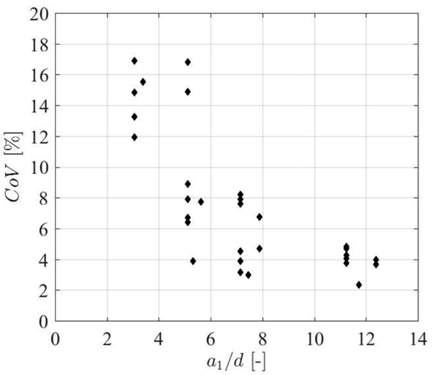

Fig. 4 CoV of the load-carrying capacity at density ρ_{mean} = 420 kg/m^3 for different test series from [19].

In this paper the impact of spacing a_1 on the variation of load-carrying capacity shall be studied. In Fig. 4 the coefficient of variation of the load-carrying capacity at a reference density is shown in dependency of the spacing a_1 of the dowels. A considerable increase of variation with decreasing spacing can be observed. The reason for the increase of variation with decreasing spacing between the dowels can be explained by the change of the failure mode: for small spacing the material properties of the timber ($f_{t,90}$, G_I) featuring high variation govern the failure whereas for large spacing the steel properties (M_y) are decisive.

5. Discussion

5.1 Reliability of Connections with Dowel Type Fasteners

The dimensions and properties of shear connections with dowel type fasteners should be designed in a way to achieve the target reliability level. Most beneficial are failure modes that cause a low variability of the load-carrying capacity as e.g. plastic failure of the metal fasteners. As already stated by [17] for the different failure modes of connections with different level of ductility different partial factors might be necessary. For the ductile failure mode EYM III the variability is in the range of $CoV \approx 5$ %. For other, brittle failure modes not only the reduction in resistance but also the increased variability should be accounted for.

5.2 Other Types of Connections

In the framework of this paper only dowel type connections were discussed in detail. However, similar considerations can be made for all kind of connections. In the following an overview about other selected connections, in respect to the reliability analysis are presented:

5.2.1 Glued-in Rods

For glued-in rods so far no homogenous design standard exists; however, the different failure modes are well-known (see e.g. [28,29]): bondline failure along the rod, tensile failure of the net cross section, block shear, splitting, and yielding of the rod. As for dowel type connections, the occurring failure type depends on different parameters such as the number of rods, the spacing and end and edge distances as well as the relative slenderness of the rods. Based on the different failure modes with the associated strength properties of the timber, steel and adhesive, the resistance of the glued-in rod connection will have different variability in dependency of the geometric properties of the system. The ductile failure mode of the rod loaded in axial tension shows commonly the highest predictability and lowest variability and, hence, should be targeted.

5.2.2 Axially Loaded Screws

The design of axially loaded screws is standardized in EC 5 [12]; accordingly the following failure types should be considered: withdrawal of the threaded part of the screw, pull-through and tear-off failure of the screw head, tensile failure of the core cross-section of the screw as well as group effects, such as pull-out of a block. The occurring type of failure depends on similar material strength parameters as described above. Screws made of high grade steel wire and hardened screws show commonly a reduced ability for ductile deformation. This may lead to limited redistribution of forces in connections with multiple screws and may result in premature brittle failure due to unequal loading of single screws.

5.2.3 Finger Joint Connections

Typical failure types of finger joint connections are a failure of the timber net cross-section, shear failure along the fingers in the timber or in the bondline. Due to inhomogeneity of timber also a timber failure outside the finger joint connections can occur. In particular, for lower strength grades the failure often occurs outside the finger joint connections. However, the quality of the finger joint connections have to be guaranteed by the producer and the resulting uncertainties are already considered in the safety factors of the corresponding engineered wood product. The same applies for other glued connections that are used for the fabrication of engineered wood products and as well as for universal finger joint connections.

6. Conclusions

From the study presented in this paper, the following conclusions can be drawn:

- the current implementation of the safety format in the design rules for timber structures is based mainly on individual member design
- connections are complex compounds of different parts and materials exhibiting a wide range of possible failure modes
- the different failure modes are governed by different geometrical parameters and material properties
- depending on the failure mode these different material properties cause different variability of the resistance of a connection

- the reliability and the resulting optimal partial factor depend on the failure mode of the connection and the variability of its resistance
- failure modes with a plastic failure of the steel allow for a low partial factor and, hence, an economic design
- brittle failure modes require a larger safety margin

The following recommendations for an optimal design can be given:

- In order to allow for an economic and reliable design the geometry and configuration of a connection should be chosen in a way to obtain high load-carrying capacity with only a small variability. This can be achieved by sufficiently large spacing, end and edge distances and timber member thickness (large dowel slenderness λ) in order to reach a failure mode with ductile deformation of the fasteners. This allows benefiting from the small variability of these ductile failure modes and the consequent small partial factors.
- The unfavourable brittle failure modes due to splitting or plug-shear should be accounted for in the design but charged with sufficient safety margin in order to account for the higher variability and reduced reliability compared to ductile failure modes.
- Reinforcement by means of e.g. self-tapping screws can be a good solution to reduce the risk of brittle failure of dowelled connections due to splitting failure [30]. It can be used to reduce the variability of load-carrying capacity also for small spacing and end and edge distances and sustain an adequate level of reliability for this type of connection geometries. Nevertheless, possible restrain in case of moisture variation might lead to negative consequences.

7. Acknowledgement

The work presented in this paper was written in the framework of the COST Action FP 1402 (www.costfp1402.tum.de).

8. References

[1] Foliente G.C., "Design of timber structures subjected to extreme loads", *Progress in Structural Engineering and Materials*, Vol. 1, 1998, pp. 236–244.

[2] Frühwald E., Serrano E., Toratti T., Emilsson A., and Thelandersson S., *Design of safe timber structures - how can we learn from structural failures in concrete, steel and timber?*, 2007, Division of Structural Engineering, Lund University, Sweden.

[3] EN ISO 6892-1, *Metallic materials - Tensile testing - Part 1: Method of test at room temperature*, European Committee for Standardization (CEN), 2016, Brussels Belgium.

[4] EN 383, *Timber structures - Test methods - Determination of embedment strength and foundation values for dowel type fasteners*, European Committee for Standardization (CEN), 2007, Brussels, Belgium.

[5] Dietsch P., "Robustness of large-span timber roof structures - Structural aspects", *Engineering Structures*. Vol. 33, 2011, pp. 3106–3112.

[6] DIN 1052, *Entwurf, Berechnung und Bemessung von Holzbauwerken - Allgemeine Bemessungsregeln und Bemessungsregeln für den Hochbau*, DIN Deutsches Institut für Normung e.V., 2008, Berlin, Germany (in German).

[7] SIA 265:2012, *Holzbau*, SIA - Schweizerischer Ingenieur- und Architekten-verein, 2012, Zurich, Switzerland (in German).

[8] Mischler A., "Influence of ductility on the load-carrying capacity of joints with dowel-type fasteners", *Proceedings of the 30th CIB-W18 Meeting*, 1997, p. CIB-W18/30-7-6, Vancouver, BC, Canada.

[9] Mischler A., "Design of joints with laterally loaded dowels", *Proceedings of the 31st CIB-W18 Meeting*, 1998, p. CIB-W18/31-7-2, Savonlinna, Finland.

[10] Kohler J., Steiger R., Fink G., and Jockwer R., "Assessment of selected Eurocode based design equations in regard to structural reliability", *Proceedings of the 45th CIB-W18 Meeting*, 2012, p. CIB-W18/45-102-1, Växjö, Sweden.

[11] Sørensen J.D., "Calibration of partial safety factors in Danish structural codes", *Proceedings of the JCSS Workshop on Reliability Based Code Calibration*, 2002, Zurich, Switzerland.

[12] EN 1995-1-1, *Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings*, European Committee for Standardization (CEN), 2004, Brussels, Belgium.

[13] Köhler J., *Reliability of Timber Structures*, 2007, Institute of Structural Engineering, ETH, vdf, Zurich, Switzerland.

[14] Jockwer R., "Impact of varying material properties and geometrical parameters on the reliability of shear connections with dowel type fasteners", *Proceedings of the 49th INTER Meeting*, 2016, p. INTER/49-7-1, Graz, Austria.

[15] Johansen K.W., "Theory of Timber Connections", *IABSE Publications*, Vol. 9, 1949, pp. 249–262.

[16] Schmid M., *Anwendung der Bruchmechanik auf Verbindungen mit Holz*, 2002, Universität Karlsruhe, Germany (in German).

[17] Jorissen A., *Double shear timber connections with dowel type fasteners*, 1998, University Press Delft, The Netherlands.

[18] Schmid M., Blaß H.-J., and Frasson R.P.M., "Effect of distances, spacing, and number of dowels in a row on the load carrying capacity of connections with

dowels failing by splitting", *Proceedings of the 35th CIB-W18 Meeting*, 2002, pp. CIB-W18/35-7-5, Kyoto, Japan.

[19] Werner H., "Tragfähigkeit von Holz-Verbindungen mit stiftförmigen Verbindungsmitteln unter Beräuksichtigung streuender Einflussgrössen", PhD Thesis, 1993, Technical University Karlsruhe, Germany (in German).

[20] Leijten A.J.M., Köhler J., and Jorissen A., "Review of probability data for timber connections with dowel-type fasteners", *Proceedings of the 37th CIB-W18 Meeting*, 2004, p. CIB-W18/37-7-12, Edinburgh, UK.

[21] Jorissen A., and Leijten A.J.M., "The yield capacity of dowel type fasteners", *Proceedings of the 38th CIB-W18 Meeting*, 2005, p. CIB-W18/38-7-5, Kyoto, Japan.

[22] Blaß H.-J., Bienhaus A., and Kramer V., "Effective bending capacity of dowel-type fasteners", *Proceedings of the 33th CIB-W18 Meeting*, 2000, p. CIB-W18/33-7-5, Delft, The Netherlands.

[23] Kohler J., "A probabilistic framework for the reliability assessment of connections with dowel type fasteners", *Proceedings of the 38th CIB-W18 Meeting,* 2005, p. CIB-W18/38-7-2, Karlsruhe, Germany.

[24] Blaß H.-J., and Colling F., "Load-carrying capacity of dowelled connections", *Proceedings of the 48th INTER Meeting*, 2015, p. INTER/48-7-3, Ŝibenik, Croatia.

[25] JCSS, *Probabilistic Model Code*, Joint Committee of Structural Safety, 2001.

[26] Jockwer R., *Structural behaviour of glued laminated timber beams with unreinforced and reinforced notches*, PhD Thesis Nr. 21825, 2014, IBK ETH Zurich, Switzerland.

[27] Jockwer R., Steiger R., Frangi A., and Kohler J., "Impact of material properties on the fracture mechanics design approach for notched beams in Eurocode 5", *Proceedings of the 44th CIB-W18 Meeting*, 2011, p. CIB-W18/44-6-1, Alghero, Italy.

[28] Stepinac M., Hunger F., Tomasi R., Serrano E., Rajcic V., and Van de Kuilen J.W.G., "Comparison of design rules for glued-in rods and design rule proposal for implementation in European standards", *Proceedings of the 46th CIB-W18 Meeting*, 2013, p. CIB-W18/46-7-10, Vancouver, BC, Canada.

[29] Tlustochowicz G., Serrano E., and Steiger R., "State-of-the-art review on timber connections with glued-in steel rods", *Materials and Structures*. Vol. 44, 2011, pp. 997–1020.

[30] Bejtka I., *Verstärkung von Bauteilen aus Holz mit Vollgewindeschrauben*, PhD Thesis, 2005, Technical University Karlsruhe, Germany.