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Assessment of existing safety formats for timber connections - How probabilistic approaches can influence connection design in timber engineering

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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:

\[
R_d = \frac{R \{ X_d \}}{\gamma_{nd}} \tag{1}
\]

where:
- \(X_d\) design value of the relevant material property;
- \(\gamma_{nd}\) 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:

\[
X_d = \eta \{ X \}`\frac{1}{\gamma_m} \tag{2}
\]

where:
- \(\eta\) partial factor accounting for uncertainty in the material model;
- \(\gamma_m\) partial factor accounting for uncertainty in the material model; and
- \(X\) outcome of the material model.
where:

\( X_k \) characteristic value of the relevant material property;

\( \eta \) conversion factor that takes account of volume and scale effects, effects of moisture and temperature, and any other relevant parameters; and

\( \gamma_m \) 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.

\[
R_d = R \left( \frac{\eta X_k}{\gamma_m} \right)
\]  

(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):

\[
R_u = \frac{R \{ \eta X_k \}}{\gamma_n}
\]  

(4)

where:

\( \gamma_n = \gamma_M = \gamma_m \cdot \gamma_{Rd} \)

### 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 derived 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.
where:

* $X_k$ characteristic value of strength property;
* $\gamma_M$ partial factor for a material property;
* $k_{\text{mod}}$ modification factor taking into account the effect of the duration of load and moisture content.

The factor $k_{\text{mod}}$ 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:

$$ R_k = k_{\text{mod}} \frac{R}{\gamma_M} $$

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 $\gamma_M$ for material properties and resistances [12].

<table>
<thead>
<tr>
<th>Fundamental combinations</th>
<th>1.30</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid Timber</td>
<td></td>
</tr>
<tr>
<td>Glued laminated timber</td>
<td>1.25</td>
</tr>
<tr>
<td>LVL, plywood, OSB</td>
<td>1.20</td>
</tr>
<tr>
<td>Connections</td>
<td>1.30</td>
</tr>
<tr>
<td>Punched metal plate fasteners</td>
<td>1.25</td>
</tr>
<tr>
<td>Accidental combinations</td>
<td>1.30</td>
</tr>
</tbody>
</table>

### 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:

$$ F_{v,Rk} = k_{\text{mod}} \frac{f_{h,i,k}}{\gamma_M} $$

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;
* $F_{aL,Rk}$ characteristic value of the axial withdrawal capacity of the fastener.
An additional factor $k_i$ 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:

$$k_i = \sqrt[\gamma_M]{\frac{\gamma_{M,\text{steel}}}{\gamma_{M,\text{mod}}}} \cdot \frac{1.3}{1.1} = 1.15$$  \hspace{1cm} (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 $\gamma_M$ and the modification factor $k_{\text{mod}}$ 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 $\gamma_M$. Existing differences in the variation of the resistance are considered currently by the factor $k_{\gamma}$. 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 \( R_{1,i} \) to \( R_{III,i} \) 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

\[
R_{1,i} = f_{s,i,d}
\]  

(9)

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

\[
R_{II,i} = f_{s,i,d} \left[ 2 + \frac{4M_p}{f_{s,i,d}^2 t_i^2} - 1 \right]
\]  

(10)

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

\[
R_{III,i} = \sqrt{4M_p f_{s,i,d}}
\]  

(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 $R_{v,split,i}$ 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 $F_{90} = 0.3 F_0$ according to [16].

\[ R_{s,t},split,i = \frac{1}{0.3} a_{3,t} f_{t,90} \]  

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 $G_{mixed}$. An angle of friction $\varphi = 30^\circ$ between dowel and timber is used by Jorissen.

\[ R_{s,split,i} = 2l \frac{G_{mixed} F_{0} d \sin \varphi (h - d \sin \varphi)}{h} \]  

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: $G_{mixed} \approx G_{1}$. 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.

\[ f_h = Ap^d d^e \epsilon \]  

23
Tab. 2 Regression parameters for Eq. (14) from [20].

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Distribution function</th>
<th>Mean value</th>
<th>stdDev</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Lognormal</td>
<td>0.097</td>
<td>0.23</td>
</tr>
<tr>
<td>B</td>
<td>Normal</td>
<td>1.07</td>
<td>0.04</td>
</tr>
<tr>
<td>C</td>
<td>Normal</td>
<td>-0.25</td>
<td>0.012</td>
</tr>
<tr>
<td>ε</td>
<td>Lognormal</td>
<td>1</td>
<td>0.11</td>
</tr>
</tbody>
</table>

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].

$$M_y = 0.3 f_d d^2$$  \(\text{(15)}\)

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.

<table>
<thead>
<tr>
<th>Grade</th>
<th>$f_{y,k}$ [N/mm$^2$]</th>
<th>$f_{u,k}$ [N/mm$^2$]</th>
<th>$f_{u,\text{mean}}$ [N/mm$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>S235</td>
<td>$390$ – $360$</td>
<td>$510$ – $545$</td>
<td>$385$ – $545$</td>
</tr>
<tr>
<td>5.6</td>
<td>300</td>
<td>500</td>
<td>534</td>
</tr>
<tr>
<td>8.8</td>
<td>640</td>
<td>800</td>
<td>854</td>
</tr>
<tr>
<td>ETG 100</td>
<td>$&gt; 865$</td>
<td>$960$ – $1100$</td>
<td>$1025$ – $1175$</td>
</tr>
</tbody>
</table>

3.2.3 Additional Material Properties and Correlations

The distribution characteristics of density $\rho$, 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 $G_1$ 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.

<table>
<thead>
<tr>
<th>Distribution function</th>
<th>Mean value</th>
<th>$CoV$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho$</td>
<td>Lognormal</td>
<td>420</td>
</tr>
<tr>
<td>$E_0$</td>
<td>Lognormal</td>
<td>11500</td>
</tr>
<tr>
<td>$f_{t,90}$</td>
<td>Weibull</td>
<td>2.0</td>
</tr>
<tr>
<td>$G_1$</td>
<td>Lognormal</td>
<td>0.3</td>
</tr>
</tbody>
</table>
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 $R_{\text{v,split}}$.

| Tab. 5 Correlation between material properties values [25]. |
|-----------------|-----------------|-----------------|
| $E_0$           | $f_{\text{t,90}}$ | $\rho$          |
| $E_0$           | 0.6             | 0.4             |

| Tab. 6 Correlation between embedment strength parameters [20]. |
|------------------|------------------|------------------|
| $B$              | $C$              | $A$              |
| $-0.99$          | $-0.24$          | 0                |
| $B$              | –                | 0.11             |
| $C$              | –                | 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 $\lambda = t_i/d$. 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 $\lambda_{\text{II/III}} = t_i/d$ 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 $\lambda$ 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 $R_{\text{v,split}}$. For larger relative thickness $\lambda$, failure mode $R_{\text{II}}$ (approx. $2.5 < \lambda < 5.5$) and failure mode $R_{\text{III}}$ (approx. $\lambda > 5.5$) become dominant.
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.

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 $\lambda = d/t$.

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 ($f_{t,90}, G_{l}$) featuring high variation govern the failure whereas for large spacing the steel properties ($M_y$) are decisive.

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

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 $\text{CoV} \approx 5\%$. 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 \( \lambda \)) 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


[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


