Assessment of the failure behaviour and reliability of timber connections with multiple dowel-type fasteners

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Abstract

Connections with metal dowel-type fasteners are important details in timber construction, connecting single members and elements to larger structures. The load-carrying capacity can be described by different failure modes of the fasteners and in the surrounding timber. These failure modes show a dependency on different dimensions and material properties. The failure can be classified into brittle and ductile failure modes based on the deformation capacity. The limited deformation capacity of the brittle failure modes has an impact on the load-carrying capacity of the entire connection with multiple fasteners. The present study takes a critical appraisal of load-carrying capacity and deformation capacity of timber connections and the implementation of their design in the Eurocode 5. By aiming for the ductile failure modes with plastic deformation of the fasteners in the design of the connection, high load-carrying capacities and high reliability can be achieved. For brittle failure modes the reduced resistance and the reduced reliability should be accounted for, especially for connections with multiple fasteners.

Keywords: dowel-type fasteners, connections, European Yield model, Monte Carlo simulation, deformation capacity, ductility

1. Introduction

In order to be able to build larger structures, individual timber elements are connected by means of different types of connections. The structural performance of the overall structure depends to a considerable part on the connections between different timber structural members. Connections not only can govern the overall strength and resistance but also the serviceability, durability and fire resistance. The performance of these connections depends on their applications; i.e. type of load (e.g. tension, shear), connecting materials, geometry, climate exposure etc.

Assessments of damaged timber structures shows that connections are responsible for a large portion of failure events [1]. Despite their importance, timber connection design frameworks are not based on a consistent basis compared to the design regulations of timber structural components. Explanations for this difference in progress of design provisions for members and connections can be found in the relative simplicity of characterising mechanical behaviour of members, as compared to connections.

1.1. 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 connections with fasteners can be divided into two groups depending on how the forces are transferred between the connected members. The main group corresponds to the connections with dowel-type fasteners such as dowels, bolts, nails, screws and staples. The load-carrying behaviour is characterized by bending deformation of the slender fasteners. The second group includes connections with stiff fasteners such as split-rings, shear-plates and punched metal plates. The load is transferred primarily by a large bearing area at the surface of the members.

The diversity of connections types is used in practice and these types have infinite variety in arrangement. This usually precludes the option of testing large numbers of replicas for a reliable quantification and verification of statistical and mechanical models.

1.2. Design of connections in timber structures

The structural performance of single connections depends on different elements with individual material and individual geometrical properties. Due to this complexity, a straightforward comparison of acting stresses and corresponding strength as done with 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.

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

Connections consisting of components of different materials, such as timber and metal fasteners, may benefit from the much smaller variability of the properties of the metal elements and, hence, from the considerably lower safety factors for the metallic fasteners when evaluating the reliability [2]. In the design equations in the current European design code for timber structures EN 1995 (Eurocode 5, EC5 [3]), this benefit amounts to about 15% [4]. The reliability based design concept offers a high potential for further enhancement of the currently applied procedures in order to benefit from the full potential of timber and hybrid structures.

1.3. Some aspects on ductility for design of timber structures

Connections are important structural details and are responsible for a large portion of failure events. Inadequate connections were found by Foliente [5] to be the primary cause of damage after extreme events such as storms or earthquakes. Ductility of the connections offers the potential for redistribution of loads in the structure as a measure for robustness [6]. A detailed discussion of the importance of ductile failure modes in connections was done by Mischler [7, 8]. In order to achieve the desired level of ductility, minimum dimensions, spacing and edge- and end-distances have to be satisfied. In practice, geometrical constraints may lead to dimensions of the connections lower than necessary to achieve ductile failure and desired high load-carrying capacities may require higher number of fasteners and smaller spacing and distances. This seems adequate especially if the desired load-carrying capacity can be obtained, however the resulting brittle failure modes may result in different consequences of failure. The ductility demonstrated based on a single fastener may not necessarily be achieved if multiple fasteners are applied in the connection. In addition also the change in variability of the load-carrying capacity has to be accounted for.

1.4. Content of this study

In this study the impact of ductile and brittle failure modes on the load-carrying capacity and failure behaviour of connections with multiple fasteners is discussed based on experimental and theoretical studies. It is not intended to evaluate and validate the different design models that exist for ductile and brittle failure modes of connections. This study deals with laterally loaded timber-steel-timber connections with metal dowel-type fasteners only.

2. Load-carrying capacity of connections

The load-carrying capacity of dowel-type fasteners is governed by the following characteristics:

- *Embedding strength* f_{h}
  The embedment strength of timber f_{h} is the system property that is associated to the resistance of solid timber against the lateral penetration of a stiff fastener. Properties such as dowel geometry, surface roughness or load to grain direction have an important impact on the embedment strength. The load-deformation behaviour of the dowel in lateral penetration in the timber is strongly non-linear. Nevertheless, a linear elastic - perfectly plastic load-deformation behaviour is assumed for the design. According to the test standard EN 383 [9] the embedment strength is determined as the maximum load within a penetration of the fastener in the timber of 5 mm.

- *Bending moment capacity of the dowel* M_{y}
  The bending moment capacity of the dowel in bending depends on the diameter and the yield strength of the dowel material. A distinct plasticity is necessary in order to achieve sufficient deformation capacity of the dowel. For simplification a linear elastic perfectly plastic material behaviour is assumed. The bending angle at which the yield moment is reached is $\leq 45/d^{0.7}$ degrees (d in mm) according to EN 14592 [10]. Small diameter fasteners show a higher deformation capacity whereas large diameter fastener reach the yield moment already at small bending angles. Overstrength or high carbon content of the steel may diminish the plastic deformation capacity of the dowel.

- *Axial resistance of the dowel* F_{ax}
  In the case of a failure mode where the fastener is inclined to the shear plane, the axial resistance of the dowel-type fastener can be activated. This so called rope effect causes an additional force component and can be used to mobilise the friction between the members of the connection. The axial resistance can be limited by the tensile, pulling out or head pull through resistance of the fastener. For smooth dowels the rope effect is commonly neglected due to their negligible pulling out resistance.

- *Timber failure*
  The resistance against splitting, block or plug shear failure is mainly governed by 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, effects such as the effective number of fasteners or the friction between the timber members also influence the load-carrying capacity.

Connections with dowel-type fasteners usually contain more than one fastener. Modelling of the load-carrying capacity of multiple fastener connections is, however, always based on the mechanics and calculations of a single fastener. This simplification might be for practical reasons: since the mechanical behaviour of single fastener connections is rather complex, the behaviour is even more complicated for multiple fastener connections, due to the large variety of configurations which could be considered amongst other factors.

2.1. Mechanical models

2.1.1. 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 Meyer [11], who included the plastic section modulus in the models by Johansen [12]. Johansen used the elastic section modulus in his studies and analysis. These failure modes describe the embedment failure of the timber and/or the ductile failure of the dowel in dependency of the thickness t_i of the timber member i (failure modes $R_{\mathrm{I},i}$ to $R_{\mathrm{III},i}$ in Figure 1). The relevant material properties are the embedment strength $f_{\mathrm{h},i}$ of the timber members and the yield moment M_y of the fastener. Geometrical parameters are the thickness t_i of the timber members and the diameter d of the fastener. The load-carrying capacities of the different failure modes applicable for a connection with a single internal steel plate (Figure 1) according to the EYM are:

Failure mode I: Embedment failure

$$R_{\mathrm{I},i} = f_{\mathrm{h},i}\ d\ t_i \tag{1}$$

Failure mode II: Failure with one plastic hinge

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

Failure mode III: Failure with two plastic hinges

$$R_{\mathrm{III},i} = \sqrt{4M_\mathrm{y}\ f_{\mathrm{h},i}\ d} \tag{3}$$

In connections with multiple fasteners, additional effects such as the unequal distribution of load between the fasteners or the accumulation of splitting forces have to be accounted for. In EC5 this is accounted for using the effective number of fasteners $n_\mathrm{ef} \leq n$.

Comparison between estimated values according to EYM and test results can exhibit considerable difference [13]. Meyer [11] proposed an additional portion of resistance from friction between the timber elements induced by the deformation and relative shorting of the fastener: the rope effect. This rope effect is limited by the axial load-carrying capacity of the fasteners and is neglected in general for dowels. Svensson and Munch-Andersen [14] discussed the impact of friction between the fastener and the timber, increasing the load-carrying capacity by an axial force component inducing the rope effect.

2.1.2. Timber failure: Splitting and block shear failure

Failure modes in the timber members are often characterized by brittle failure mechanisms in shear and tension

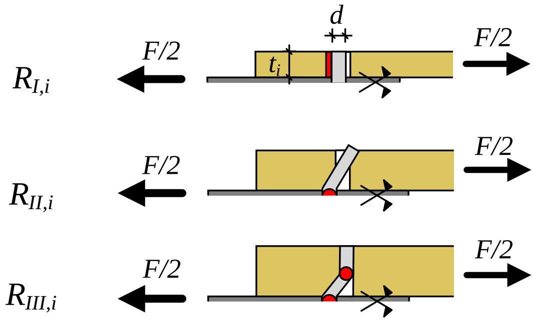

Figure 1: Simplification of failure modes of the EYM for a symmetric half of a dowelled timber-steel-timber connection.

perpendicular to the grain. A comprehensive review and assessment of different design approaches for timber failure modes is given in [15]. A design equation for the situation of block shear failure of laterally loaded groups of fasteners in steel-timber connections is given in the Appendix A of EC5. Additional failure modes with tension perpendicular to the grain splitting and shear fracture of the connection are not accounted for in detail. The Canadian standard CSA O.86 [16] considers different brittle failure modes for the design of connections [17].

Geometrical parameters with an impact on the brittle failure of connections are spacing between fasteners a_1, end-grain distance a_3, edge distances a_4, member thickness t. The material parameters with an impact are shear strength f_v, tension perpendicular to grain strength $f_{\mathrm{t},90}$, stiffness properties (E_0 and G_v) and fracture energies in tension perpendicular to grain $G_{\mathrm{f,I}}$ and shear $G_{\mathrm{f,II}}$.

A very simplified model for considering impact of the end-grain distance a_3 is presented in Eq. (4) where $R_{\mathrm{t,split},i}$ is the load-carrying capacity parallel to the grain. The model is based on a verification of tension perpendicular to grain strength $f_{\mathrm{t},90}$. 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 $F_{90} \approx 0.3F_0$ according to [18].

$$R_{\mathrm{t,split},i} = \frac{1}{0.3} t_i\ a_3\ f_{\mathrm{t},90} \tag{4}$$

The model in Eq. (4) can be used in analogy for describing the impact of spacing a_1 on the fracture in tension perpendicular to the grain.

Jorissen [19] presented a fracture mechanics-based design approach for brittle failure of a connection (Eq. (5)). Due to the complex stress state, the fracture process is described by mixed mode fracture with $G_{\mathrm{f,mixed}}$. An angle of friction $\phi = 30°$ between dowel and timber is used by Jorissen.

$$R_{\mathrm{f,split},i} = 2t_i \sqrt{\frac{G_{\mathrm{f,mixed},i}\ E_{0,i}\ d\ \sin\phi\,(h - d\ \sin\phi)}{h}} \tag{5}$$

A conservative estimate can be made by assuming the mixed mode fracture energy to be equal to the mode I fracture energy with crack opening: $G_{\mathrm{f,mixed}} = G_{\mathrm{f,I}}$.

Other more sophisticated fracture mechanics-based approaches can be found e.g. in [20]. They state that mode I splitting is most common for $m = 1$ row of fasteners whereas for $m \geq 2$ rows plug shear or group tear out failure is more common due to the change in energy release rate in the model of a beam on elastic foundation.

2.2. 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 Werner [21]. 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 Köhler [22]. In the following, the most important characteristics of the material property values are summarized.

2.2.1. Embedment strength f_h

The equation in EC5 for the determination of embedment strength for dowels in predrilled holes loaded parallel to the grain was proposed by Whale and Smith [23] as follows:

$$f_{\mathrm{h,k}} = 0.082\rho_{\mathrm{k}}\,(1 - 0.01d) \tag{6}$$

Table 1: Regression parameters from [25]

Parameter	Type	Mean	stDev
A	Lognormal	0.097	0.23
B	Normal	1.07	0.04
C	Normal	-0.25	0.012
ϵ	Lognormal	1	0.11

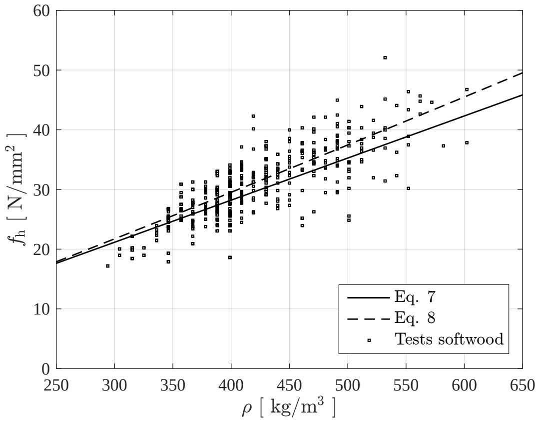

Figure 2: Individual test results of embedment strength for softwood from [25] together with mean values of Eq. (7) and (8).

The mean value of embedment strength was proposed by Whale and Smith [23] using the mean value of the density:

$$f_{\mathrm{h,mean}} = 0.082\rho_{\mathrm{mean}}\,(1 - 0.01d) \tag{7}$$

Additional impacts on the embedment strength such as the surface roughness of the dowel or the moisture content of the timber are discussed in e.g. [24].

The test data from the basis for the development of Eq. (6) was analysed more in detail by Leijten et al. [25] and the distribution characteristics given in Table 1 were determined for the proposed equation for embedment strength:

$$f_{\mathrm{h}} = A\rho^B d^C \epsilon \tag{8}$$

A comparison between Eq. (6) and Eq. (8) together with the test data is given in Figure 2. The embedment strength in Eq. (8) yields for GL24h with $\rho_{\mathrm{mean}} = 420$ kg/m^3 ($CoV = 10\%$) and $f_{\mathrm{h,mean}} = 32.6$ N/mm^2 ($CoV = 16\%$). The mean embedment strength according to Eq. (7) is $f_{\mathrm{h,mean}} = 30.3$ N/mm^2.

Table 2: Yield strength f_y and tensile strength f_u in dependency of steel grades for a $CoV = 4\%$ and lognormal distr. properties

Grade	$f_{y,k}$ [N/mm^2]	$f_{u,k}$ [N/mm^2]	$f_{u,mean}$ [N/mm^2]
S235	$\approx 190 - 360$	$\approx 360 - 510$	$\approx 385 - 545$
4.6	240	400	427
6.6	360	600	641
8.8	640	800	854
ETG 100	> 865	$\approx 960 - 1100$	$\approx 1025 - 1175$

Table 3: Distribution characteristics of material parameters.

Property	Unit	Distribution function	mean	CoV
ρ	kg/m^3	Lognormal	420	10%
f_u	N/mm^2	Lognormal	437	4%
E_0	N/mm^2	Lognormal	11500	23%
$G_{f,I}$	N/mm	Lognormal	0.3	20%
$G_{f,II}$	N/mm	Lognormal	1.05	30%
f_v	N/mm^2	Lognormal	5	25%
$f_{t,90}$	N/mm^2	Weibul	2	30%

Table 4: Correlation between material properties values [31].

	E_0	f_v	$f_{t,90}$
ρ	0.6	0.6	0.4
E_0	-	0.4	0.4
f_v		-	0.6

Table 5: Correlation between embedment strength parameters according to [25].

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

2.2.2. Yield moment M_y

The relevant resistance of a fastener in bending is between the elastic and full plastic bending capacity [26]. Depending on the failure mode of the EYM and the diameter of the fastener, the relevant resisting moment of the fasteners is reached at different bending angles. The resisting moment of the fastener can be determined in four-point bending tests e.g. by means of the test equipment presented by Werner [21] and Ehlbeck and Werner [27]. The connection between yield moment of the dowel M_y and yield and tensile strength of the steel is discussed in literature; e.g. [28].

Eq. (9) is given in EC5 and is based on studies by Blaß et al. [29]. Recent studies by Blaß and Colling [30] show that there can be a considerable difference between steel qualities of different batches. The variation of material properties of the steel within one batch is rather small. Kohler [2] proposes $CoV \approx 4\%$.

$$M_y = 0.3 f_u \, d^{2.6} \tag{9}$$

2.2.3. Additional material properties and correlations

All distribution characteristics used in this study are summarized in Table 3. The distribution characteristics of density ρ, Modulus of elasticity parallel to the grain E_0, shear strength f_v, and tension perpendicular to grain strength $f_{t,90}$, are taken from the Probabilistic Mode Code of the JCSS [31] (see also Köhler et al. [32]). The tension perpendicular to grain strength and shear strength show a pronounced volume effect. The stressed volume in a connection is rather small compared to other situations in timber structures, e.g. curved or pitch-cambered beams. Hence, a rather high value $f_{t,90}$ compared to the values specified in EN 338 [33] is suggested in Table 3. Aicher et al. [34] give similar values of tension perpendicular to grain strength for small sized specimen of $V \approx 0.1$ dm^3. The mode I fracture energy G_I is based on studies by Jockwer [35].

The correlations between the material property values is based on [31] (Table 4) and [25] (Table 5). No correlation between $G_{f,I}$ and the other material properties is suggested by Jockwer et al. [36] for softwoods commonly used in practice. In contrast, Larsen and Gustafsson [37] presented an equation for fracture energy in dependency of timber density based on test results from a wide range of timber densities and species.

2.2.4. Model uncertainties

The mismatch of test results and predicted load-carrying capacities has been known for a long time. Larsen [13] reports the load-carrying capacity observed in tests on nailed connections was approximately 20% higher compared to the predicted values according to the EYM. Advanced models such as the one proposed by Svensson and Munch-Andersen [14] may help to achieve a better estimate of the load-carrying capacity. Köhler [22] evaluated the model uncertainties for different mechanical and empirical models based on the test results given in [19]. He accounted for the fracture mechanics based model in Eq. (5) and additional parameters in the evaluation. The predicted capacities according to EYM increased by 20% and 30% for the failure modes with one and two plastic hinges in the fasteners, respectively. As a result Köhler [22] was able to minimize the bias of the model uncertainty and to reduce the coefficient of variation to $CoV \approx 15\%$.

The present study is focused on the interaction of different failure modes and on their impact on the variability of the load-carrying capacity and type of failure. The absolute value of the individual load-carrying capacity is not validated in more detail. An increase of the load-carrying capacity in Eqs. (2) and (3) by approximately 20% and 30% respectively, as suggested by Köhler [22], would increase the relative impact of the timber failure modes represented by the simplified models in Eqs. (4) and (5).

2.3. Impact of varying material properties on the load-carrying capacity of connections

The load-carrying capacity of a wood-steel-wood connection with a single dowel-type fastener is the minimum of Eqs. (1), (2), & (3) and limited by the timber failure represented by the simplified models in Eqs. (4) & (5). The impact of varying material properties on the load-carrying capacity was studied by random generation of individual load-carrying capacities with $n_s = 10^5$ simulations per step. In the example shown in Figure 3 & 4 in addition to the values specified in Table 3, the following material and geometric properties have been chosen: $\rho_{\text{mean}} = 420\ \text{kg/m}^2$, $f_{\text{u,k}} = 400\ \text{N/mm}^2$ (steel quality 4.6), $d = 12$ mm, $h = 10d$, $a_3 = 7d$.

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 represented by the relative thickness $\lambda = t/d$. The material properties of relevance for the load-carrying capacity according to EYM are the embedment strength of the timber and the yield moment of the steel. The yield moment of the steel only impacts the load-carrying capacity in failure modes II and III. The end-grain distance a_3 of a connection with a single fastener has an impact on the failure mode for small λ. For small end-grain distance the splitting failure modes become relevant.

In Figure 3 (left) the different percentile levels of the load-carrying capacity are shown together with the coefficient of variation (CoV) in dependency of the relative thickness of the side members $\lambda = t/d$. With increasing λ the load-carrying capacity is increasing. In addition, the variability decreases and the shape of the distribution function changes, in particular the lower and most important tail of the distribution function. This can be recognized by the relative distance of the 95% and 99% fractile values. The CoV is highest of the brittle failure modes for small λ. In Figure 3 (right) the relative portion of the corresponding failure modes are shown. For small relative thickness of the side members (approx. $\lambda < 2.5$), more than 90% of the simulated connections failed in the brittle mode $R_{\text{f,split}}$ (Eq. 5). For larger relative thickness λ, the ductile failure modes R_{II} (approx. $3 < \lambda < 5$) and R_{III} (approx. $\lambda > 5.5$) become dominant.

In Figure 4 the different percentile levels of the load-carrying capacity are shown in dependency of the end-grain distance a_3/d. In Figure 4 (right) the relative portion of the corresponding failure modes are shown. For small relative end-grain distances a_3/d the splitting failure modes cause a reduction of load-carrying capacity and an increase of the CoV. For large a_3/d the impact of the splitting failure modes decreases to such an extent that for $a_3/d > 6$ the 1% fracture value of load-carrying capacity is almost constant.

3. Failure behaviour of connections

3.1. Tests on dowelled connections with slotted-in metal steel plates

In tests carried out at ETH Zurich, the impacts of geometrical and material parameters on the load-carrying capacity of dowelled connections with slotted in steel plates was evaluated. The specimens were wood-steel-wood connections with two individual side members. The tests were carried out as pull-pull tests, but only one connection with $d = 12$ mm was tested until failure since the opposite connection with $d = 25$ mm was considerably stronger and exhibited little deformation. The interaction between both connections was neglected due to the large distance of ≈ 200 mm between the last rows of fasteners. The steel plate had a thickness 10 mm.

The side members with a thickness $t = 50$ mm and a width $h = 150$ mm were made of solid timber and were selected in order to achieve similar density. Three dowels in a row ($n = 3,\ m = 1$) with different spacing and end-distances were tested as illustrated in Figure 5; the configurations and load-carrying capacities are summarized in Table 6.

The tests were carried out by displacement control and the deformation of the two side members with respect to the central steel plate was measured by means of LVDT. For further evaluation, the mean value of the deformation w of the two sides of the specimen was used. Failure was reached within approx. 5 min.

The timber for the specimens was selected from a sample of boards with a wide range of densities. It was aimed at an equal density of the two side members of the connection. The resulting range of timber density of the specimens is between $\rho = 360 - 520$ kg/m^3.

The properties of the steel of the dowels was controlled in four point bending tests. The resulting tensile strength back-calculated from Eq. (9) is $f_{u,mean} \approx 581$ N/mm^2 for S235 and $f_{u,mean} \approx 969$ N/mm^2 for ETG 100. Especially the tensile strength of the low grade steel S235 is much higher than expected by the specification of the steel quality. In total 7 bending tests have been carried out for S235 and 8 for ETG 100. The resulting coefficients of variation of tensile strength are $CoV = 5.8\%$ for S235 and $CoV = 5.1\%$ for ETG 100.

In Figure 6 examples of the load-deformation behaviour of different configurations are shown. For the specimens with small spacing a_1, early failure before larger plastic deformation can be seen. Tension perpendicular to grain splitting and/or plug shear failure were the main reasons for this early and brittle failure. For larger spacing a_1 larger plastic deformations were achieved. Nevertheless splitting and/or plug shear failure occurred at larger deformation as the final failure also for large spacing and end-distances. The deformation capacity of the connection can be considered as sufficient if all fasteners in the connection are able to develop a failure mode according to the EYM (Eqs. (1-3)) before timber failure modes in tension perpendicular to grain splitting and/or plug shear (Eqs. (4) & (5)) occur.

In order to allow for a comparison of load-carrying capacities and variation between specimens of different density the results are normalized to a density of $\rho = 420$ kg/m^3 as follows:

$$R_{u,420} = R_{u,i} \left(\frac{420 \text{ kg/m}^3}{\rho_i} \right)^k \tag{10}$$

The parameters k were determined by means of least squares fit for each test series. The resulting mean load-carrying capacity $R_{\mathrm{u,mean,420}}$ for a density of $\rho = 420$ kg/m^3 together with the parameters k are given in

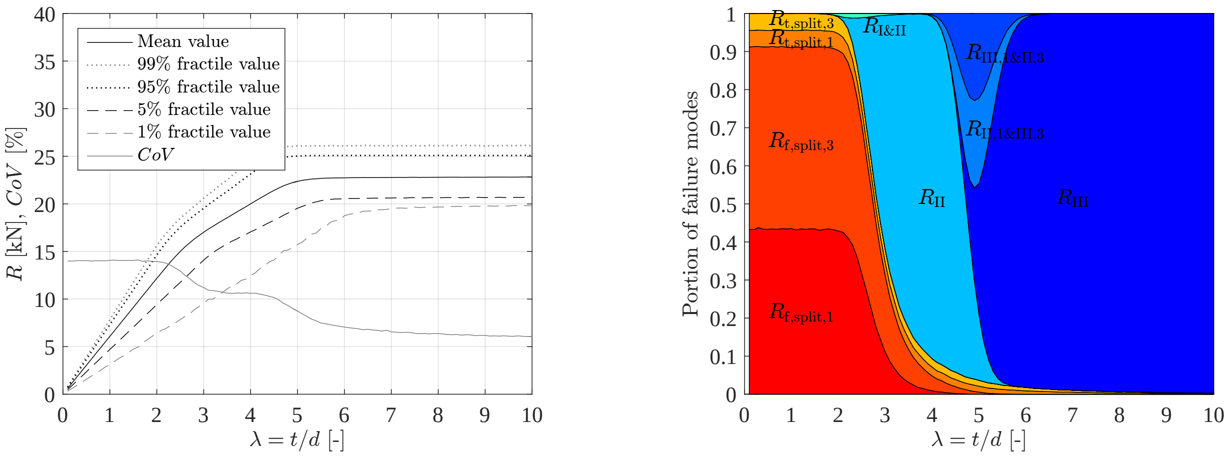

Figure 3: Relevant load-carrying capacity at mean and different fractile levels and corresponding coefficient of variation (CoV) in dependency of the thickness of the side members $\lambda = t/d$ ($a_3/d = 7$) and portion of the respective failure mode.

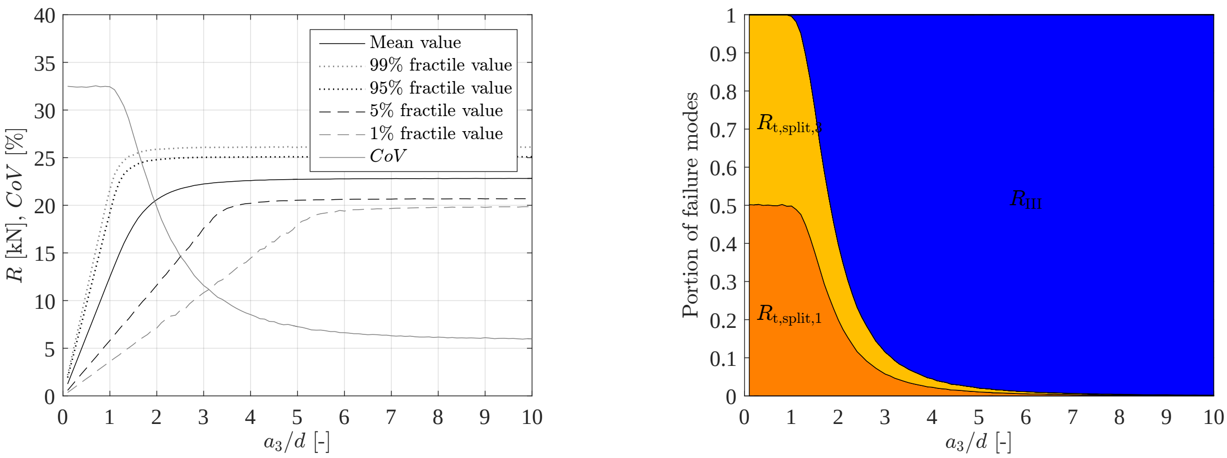

Figure 4: Relevant load-carrying capacity at mean and different fractile levels and corresponding coefficient of variation (CoV) in dependency of the end-grain distance a_3/d ($\lambda = t/d = 8$) and portion of the respective failure mode.

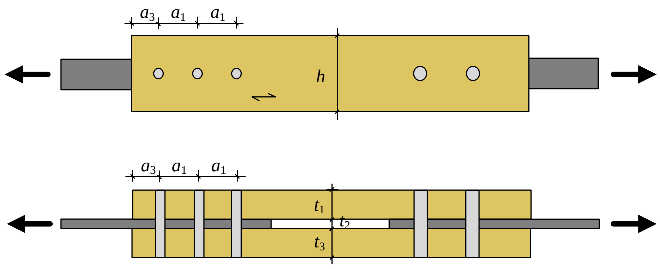

Figure 5: Geometry of the dowelled connections with slotted in metal steel plates.

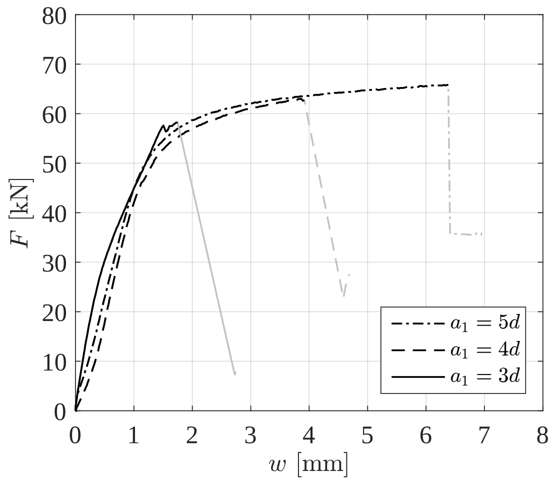

Figure 6: Impact of spacing a_1 on load-deformation behaviour for $a_3 = 5d$.

Table 6. The ductility ratio $D_s = w_y/w_u$ is calculated according to SIA 265 [38] for $w_u = w_{max}$. The ultimate deformation w_u was chosen to be equal to the deformation at maximum load w_{max} since all tests showed a sudden load drop. From the results and observations the following conclusions can be drawn with regard to the impact of the geometrical and material parameters on the load-carrying capacity and variation:

- The load-carrying capacity decreases with decreasing spacing a_1 or end-grain distance a_3.
- The load-carrying capacity increases with increasing tensile strength of the steel dowels.
- The test series with the smallest end-grain distance shows the highest variation of load-carrying capacity.
- The ductility ratio D_s increases with increasing spacing and end-distances.
- Higher load-carrying capacities are achieved for the test series with higher D_s.

Table 6: Results of the test series. Load-carrying capacity $R_{\mathrm{u,mean,420}}$ is normalized for a density of $\rho = 420\mathrm{kg/m}^2$.

a_1 [-]	a_3 [-]	Steel grade	# [-]	$R_{\mathrm{u,mean,420}}$ (CoV) [kN] (%)	k [-]	D_s [-]
$5d$	$3d$	Low	6	44.0 (12.4%)	0.77	1.4
$3d$	$5d$	Low	10	53.6 (7.4%)	-0.48	2
$4d$	$5d$	Low	10	65.5 (3.0%)	0.10	2.7
$5d$	$5d$	Low	8	67.9 (7.0%)	0.46	3.7
$3d$	$7d$	Low	12	52.9 (8.9%)	0.52	2.0
$4d$	$7d$	Low	13	64.1 (5.0%)	0.51	3.0
$5d$	$7d$	Low	12	65.9 (4.8%)	0.59	4.6
$3d$	$7d$	High	7	67.0 (6.2%)	0.17	2.0
$5d$	$7d$	High	8	84.8 (2.9%)	0.56	3.4

3.2. Consequences of brittle and ductile failures

3.2.1. General

Ductile deformations of fasteners and connections offer the potential for redistribution of loads in the connections and in the structure. The deformation capacity of the splitting and shear failure modes is generally low. Hence different design codes, such as DIN 1052 [39] or SIA 265 [38], set a ductile failure mode with $D_s > 3$ as the basis for the design of connections. It might seem adequate to chose a brittle failure mode if the load-carrying capacity of the individual fasteners is considered. However, the consequences of failure have to be accounted for if not ductile but brittle failure modes become relevant.

The low deformation capacity of the brittle failure modes does not allow for any redistribution of forces and cause an immediate failure of the entire connection. This brittle failure behaviour can be modelled by a serial assembly of single resistance elements. With increasing number of elements the overall resistance decreases, as described by the weakest link theory according to Weibull [40].

Ductile failure modes allow for a redistribution of loads within the connections which can be modelled by a parallel assembly of single elements. The load-carrying capacity of the entire connection is the sum of the capacities of the individual elements.

3.2.2. *Effect of multiple fasteners in a row*

In connections with multiple fasteners it can be observed that the load-carrying capacity of the entire connections is smaller than the sum of the load-carrying capacities of each individual fastener. The distribution of forces in each fastener depends, amongst others, on the stiffness of the fasteners and the timber members. The unequal distribution of forces in connections with multiple fasteners was discussed e.g. by Volkersen [41] and Blaß [42].

Jorissen [19] performed a large number of tests with various configurations and different numbers of fasteners in a row. The tests carried out were bolted shear connections in wood-wood-wood. Jorissen observed a reduction of load-carrying capacity with decreasing spacing due to premature splitting of the connection. The evaluation of the test results shows an increase of variation of load-carrying capacities for these brittle failure mechanisms for small spacing. Jorissen proposed a reduction factor for the effective number of fasteners in dependency of the number of fasteners, their spacing a_1 and a reference spacing $a_{1,\mathrm{ref}}$:

$$n_{\mathrm{ef}} = \min \begin{cases} n \\ n^{0.9} \cdot \sqrt[4]{\frac{a_1}{a_{1,\mathrm{ref}}}} \end{cases} \tag{11}$$

The reference spacing $a_{1,\mathrm{ref}} = 13d$ according to the Jorissen [19] was chosen for the implementation of Eq. (11) in EC5. In contrast, DIN 1052 and SIA 265 use a smaller value $a_{1,\mathrm{ref}} = 10d$.

Eq. (11) considers a perfect load redistribution with the effective number of fasteners equal to the number of fasteners $n_{\mathrm{ef}} = n$ for large spacing a_1. This is the case if the ductile failure modes of the European Yield Model with large deformations of single fasteners at constant load is reached. In order to achieve this beneficial load redistribution between single fasteners, premature brittle failure modes have to be avoided. This behaviour was discussed e.g. by Gehri [43] for glued-in rod connections in order to overcome detrimental effects of production inaccuracies.

3.2.3. *Modelling the failure behaviour of multiple fastener connections*

Based on the observations from tests and on the models for serial and parallel connections, the load-carrying capacity of a multiple fastener connection can be modelled as follows:

$$R_{\mathrm{total}} = \min \left\{ n \cdot \min\left\{R_{\mathrm{brittle},i}\right\}, \sum_{i=1}^{n} R_{\mathrm{ductile},i} \right\} \tag{12}$$

The resulting load-carrying capacity can be evaluated with regard to the number of fasteners failing in a ductile mode. The maximum load-carrying capacity of the entire connection can be achieved only if all fasteners reach a ductile failure mode. The occurrence of brittle failure will always cause premature failure at lower load levels.

In Figure 7 (left) the number of fasteners failing in a ductile mode in dependency of the spacing a_1/d are determined for a connection with $n = 6$ fasteners in a row and with a side member thickness of $\lambda = t/d = 8$. A number of $n_s = 10^5$ simulations per spacing were performed

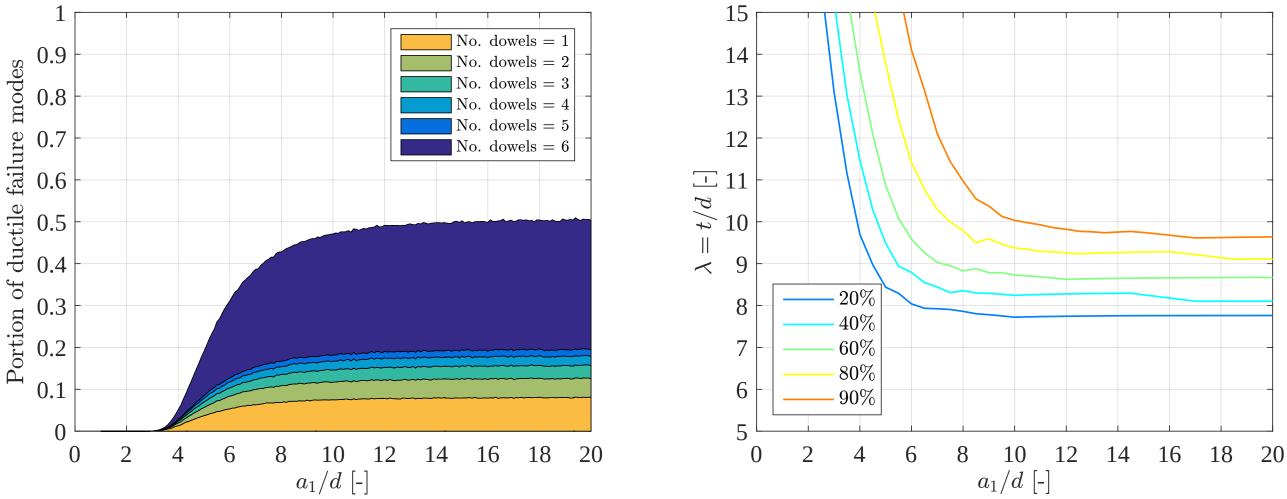

Figure 7: Portion of ductile failure modes in dependency of the spacing a_1/d for $n = 6$ and $\lambda = t/d = 8$ (left) and required configuration of spacing a_1/d and relative timber thickness $\lambda = t/d$ for achieving the respective percentage of ductile failure of all $n = 6$ fasteners.

as random generation of individual load-carrying capacity of connections. Even for large relative spacing a_1/d, only approximately 50% of the simulated connections fail with ductile failure in all fasteners. In Figure 7 (right) the required side member thickness for achieving the respective percentage of ductile failure of all $n = 6$ fasteners is derived in dependency of the spacing a_1/d. For small spacings, brittle failure occurs also for large side member thickness. For large spacings of approximately $a_1/d > 12d$ and large relative side member thicknesses $\lambda > 10$, the percentage of the ductile failure in all fasteners of the simulated connections is well above 90%. This dominating ductile failure is predicted by Eq. (11) with $a_{1,\text{ref}} = 13d$.

From the example shown in Figure 7 the following conclusions can be drawn:

- Brittle failure occurs for the majority of the connections for small spacing a_1
- Sufficient spacing is needed in order to achieve ductile failure of a larger number fasteners in a connection
- The required member thickness t/d for achieving a certain percentage of ductile failures increases with decreasing spacing a_1/d
- The member thickness for achieving the failure mode with two plastic hinges in the fasteners is not sufficient for guaranteeing ductile failure of a connection with multiple fasteners.

4. Discussion

The properties and dimensions of connections with dowel-type fasteners should be chosen and designed in a way to achieve desired reliability of the structure. According to EC5, a constant partial safety factor is applied for the design of connections irrespective of the dimensions. Therefore, what is most beneficial for achieving a high reliability is to aim for failure modes that cause a low variability of the load-carrying capacity as e.g. ductile failure of the metal fasteners. As already stated by Jorissen [19], for an optimized design different partial safety factors might be necessary for the different failure modes of connections with different level of ductility.

Brittle failure modes cause an immediate failure without the possibility of redistribution of load within a connection with multiple fasteners or between different connections. This deficiency of deformation capacity shows no potential for robustness. The ductile failure modes allow for a redistribution of loads and an activation of the load-carrying capacity of all fasteners. Failure occurs due to excessive deformations, which can be associated with low consequences of failure. Especially in case of connections with multiple fasteners, the ductility is essential in order to avoid weakest link effects.

It is not sufficient to reward the failure modes in dependency of their variability of load-carrying capacities as currently done by respective factors (1.15 according to EC5 for failure mode III): brittle failure modes leading to severe consequences in case of failure have to also be charged.

The following recommendations with regard to a more robust and reliable design of connections can be made:

- Brittle failure modes should be avoided in general.
- Ductile failure modes are essential for connections with multiple fasteners in order to achieve high load-carrying capacities, high reliability and adequate robustness.
- Measures for avoiding brittle failure modes are to require sufficient minimum spacing and distances and to recommend larger relative side member thickness $\lambda = t/d$.
- Simplified design procedures as suggested by Blaß and Ehlbeck [44] or as established in DIN 1052 or SIA 265, with a conservative reduction of load-carrying capacity for the failure modes with less than two plastic hinges in the fasteners should be preferred with regard to robustness, especially for connections with multiple fasteners.
- Reinforcement by means of e.g. self-tapping screws can be a good measure to reduce the risk of brittle failure of dowelled connections due to splitting failure [45]. It can be used to reduce the variability of load-carrying capacity also for small spacing and end-distances and sustain an adequate level of reliability for this type of connection geometries. Hence, reinforcement of dowel-type connections should be accounted for in future version of EC5.

5. Conclusions

In this study, lateral timber-steel-timber connections with metal dowel-type fasteners were evaluated with regard to load-carrying capacity, deformation capacity and reliability. The following conclusions can be drawn:

- The different failure modes of connections with dowel-type fasteners depend on the material and geometrical properties of the timber members and the fasteners. The variability of the load-carrying capacity depends on the different variability of the material properties of the respective failure mode. In general, failure modes with a brittle failure mechanism lead to a higher variability of the load-carrying capacity, whereas failure modes with a ductile failure mechanism lead to a lower variability.
- The dimensions and properties of connections should be designed in a way to achieve the desired target reliability level. Due to the absence of design rules for brittle failure of connections in EC5, the reliability of connections failing in these brittle failure modes must be critically assessed, especially for small fastener spacings. Furthermore the introduction of reduction factors for multiple fastener connections blur the impact of these brittle failure modes on the load-carrying capacity. Most beneficial are failure modes that cause a low variability of the load-carrying capacity as e.g. plastic failure of the metal fasteners. The brittle failure modes show not only a reduction in resistance but may also require a larger safety margin.
- 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. In addition,

adequate deformation capacity is necessary for connections with multiple fasteners. This can be achieved by sufficiently large spacing, end-distances and timber member thickness (large dowel slenderness λ) in order to reach a failure mode with ductile deformation of the fasteners.

- The unfavourable brittle failure modes due to splitting or plug-shear failure should be accounted for in the design and charged with sufficient safety margin in order to account for the higher variability and reduced reliability compared to ductile failure modes.

- Similar considerations can be made for all kind of connections, such as glued-in rods, axially loaded screws, and glued connections.

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