COST Action FP1402 "Basis of Structural Timber Design" - from research to standards

Short Term Scientific Mission Report

by

Robert Jockwer
ETH Zurich,
Institute of Structural Engineering
Zurich, Switzerland

This report covers research activities carried out
during a stay from 8th to 26th February 2016
Host: Dr. Philipp Dietsch,
Technical University of Munich
Munich, Germany

Review of design recommendations for connections loaded perpendicular to the grain

Robert Jockwer
ETH Zurich, Institute of Structural Engineering IBK, Zurich, Switzerland

1 Introduction

1.1 Background

Timber exhibits good strength and stiffness properties parallel to the grain but only very low strength perpendicular to the grain. Structural details like connections, where tensile forces perpendicular to the grain are introduced in the timber, exhibit a high risk of fracture due to the low strength and brittle failure mechanism of timber in tension perpendicular to grain. Careful design of such lateral connections is required in order to reach the level of structural safety required by design codes. Design procedures for such details can be found in literature exhibiting different degree in complexity. Compared to test results they show different degree of correlation. The failure mechanisms of unreinforced connections perpendicular to the grain and the structural capacity of reinforced connections have to be studied more in detail in order to give reliable recommendations for the design of such connections.

1.2 Research plan for the STSM

It was intended to benefit within the STSM from the experience in experiments and theoretical studies carried out at TU Munich and ETH Zurich. The results of these studies should be evaluated with regard to their relevancy for updates of design codes and design recommendations. The impact of different joint geometries and boundary conditions should be accounted for. It was aimed at proposing recommendations for an updating of design approaches and recommendations for Eurocode 5.

1.3 Content of this report

After a short introduction the geometrical properties and types of connections loaded perpendicular to the grain are discussed in this report. Design approaches from scientific literature and from design standards are summarized and compared. The impact of geometrical parameters on the load-carrying capacity is analysed by evaluation of test series from literature. Selected design approaches are evaluated with regard to their capability to describe the load-carrying capacity of various configurations of connections perpendicular to the grain. Based on these evaluations suggestions for a revision and updating of the existing design equations in Eurocode 5 are given.

2 General

2.1 Geometry of connections loaded perpendicular to the grain

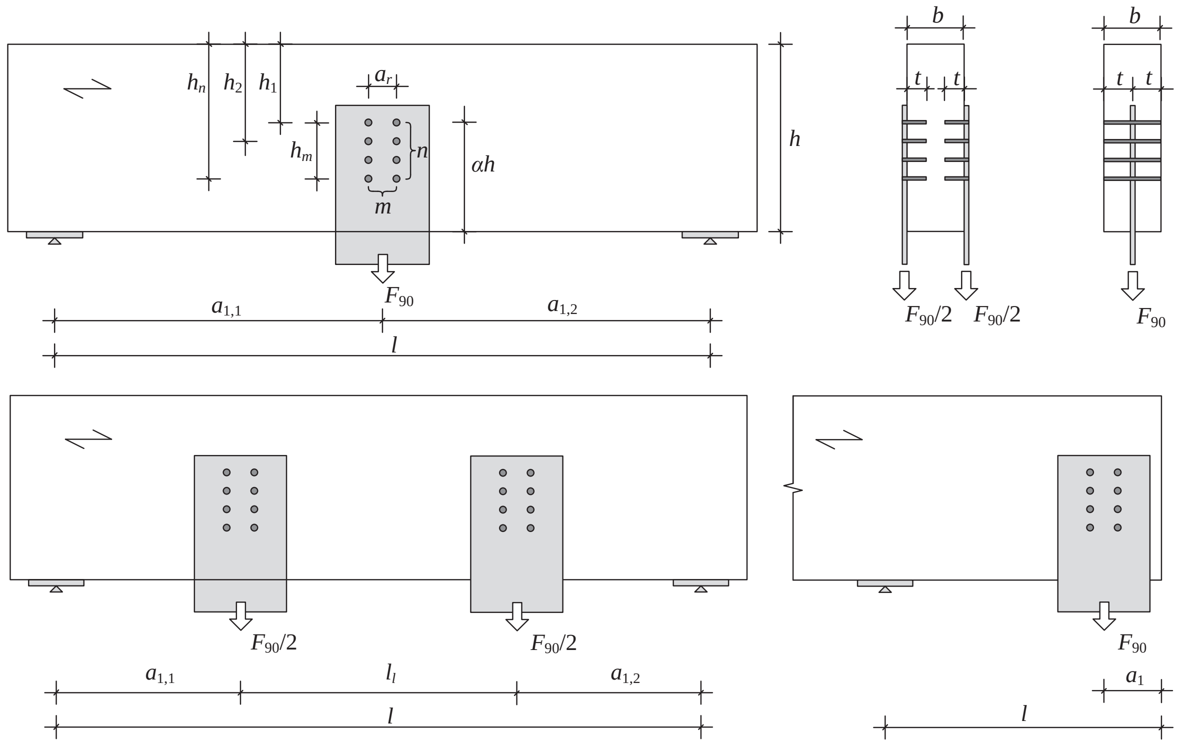


Fig. 1: Definition at connections loaded perpendicular to grain.

The geometrical properties and denotations of a connection loaded perpendicular to the grain are illustrated in Fig. 1. The relevant geometrical properties are the dimensions of the beam ($b \cdot h$), relative connection height α, connection width a_r and height h_m. In addition the geometry of the connection can be described by the number of columns m and rows n of fasteners. The height of the connection and the position of specific fasteners can be specified by h_n the distance between the n^{th}-row of fasteners to the loaded edge. In case of multiple connections the distance between each others is l_l. The distance to the end grain is denoted by a_1.

2.2 Types of connections loaded perpendicular to the grain

Connections loaded perpendicular to the grain are often made by means of e.g. nails, dowels, bolts, (self-tapping) screws, glued-in rods or shear connectors. The number of fasteners in a connection depends on the type of fastener used. Small diameter fasteners like nails or rivets are often used with a larger quantity within one connection whereas large diameter fasteners like bolts, glued-in rods or shear connectors are also used individually.

Connections can be either made as timber/timber connections like in the case of many shear connectors, or can be made in combination with steel plates like for (3D) nailing plates or dowelled slotted in metal steel plates. Glued-in rods or self-tapping screws can be directly loaded in tension and do not need additional elements for hanging loads.

3 Design approaches

The existing design approaches for connections loaded perpendicular to grain can be separated into approaches based on stress criteria or fracture mechanics theory. A good review on existing approaches can be found in e.g. Schoenmakers (2010) and Thiede (2014).

An overview of the background and relation of the existing design approaches is illustrated in Fig. 2. In this chapter all the illustrated design approaches will be presented and discussed more in detail.

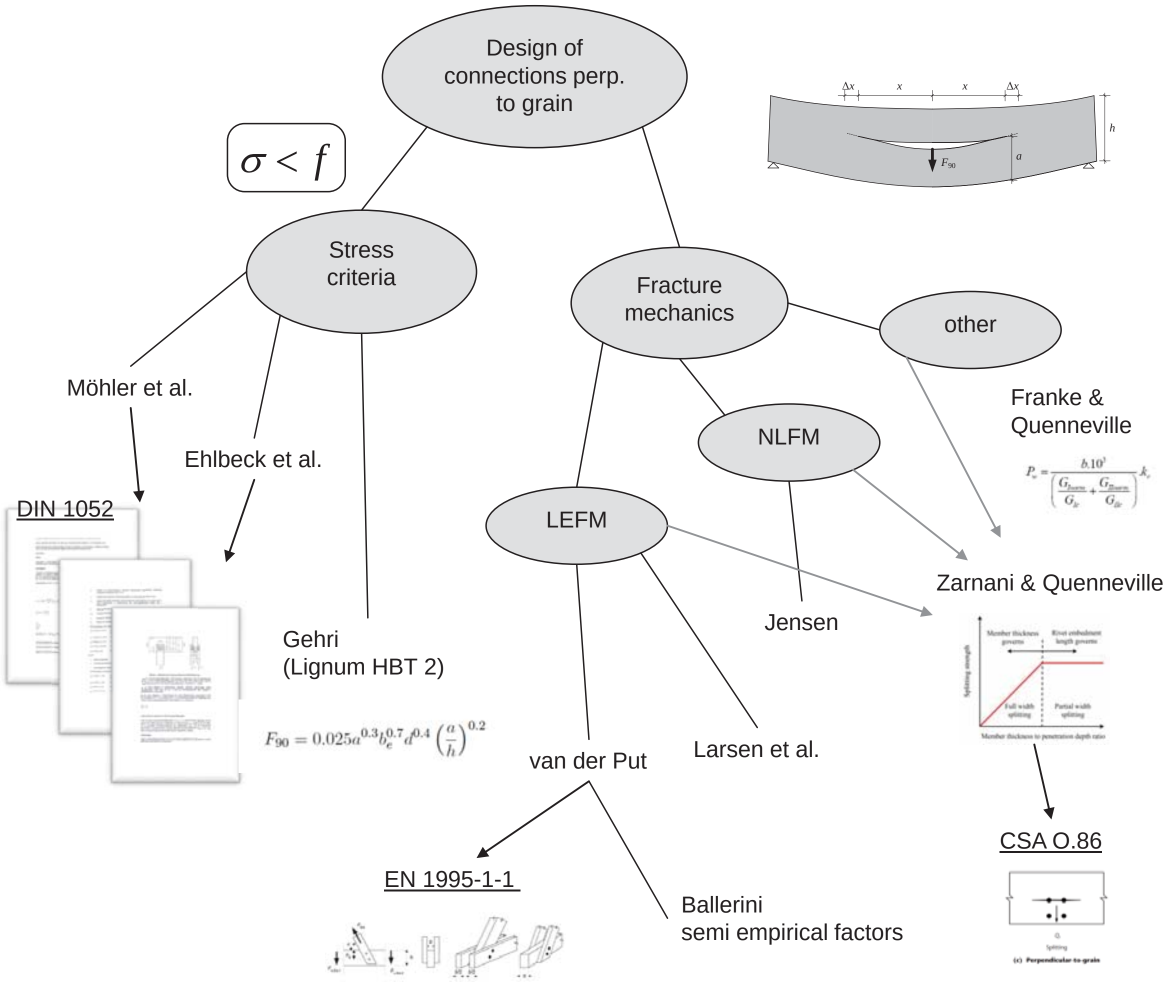


Fig. 2: Illustration of the background of design approaches for connections loaded in tension perpendicular to the grain.

3.1 Approaches based on stress criteria

3.1.1 Möhler and Siebert (1980)

A first approach based on stress criteria was presented by (Möhler and Siebert 1980, 1981). The approach is based on the test series reported in (Möhler and Lautenschläger 1978, Möhler and Siebert 1980). The volume loaded in tension perpendicular to the grain is based on the studies on volume effect by Barrett et al. (1975).

$$F_{90,mean} = \frac{f(a,h)}{10} \frac{(b_{eff} W' h_1)^{-0.2} \, b_{eff} W'}{s} \quad [\text{kN}] \tag{1}$$

with b_{eff}, W' and h_1 in [cm].

$$\frac{f(a,h)}{10} = 0.68 + 1.37 \frac{h}{100} + 0.2 \frac{a}{h} + 0.4 \frac{a}{100} \tag{2}$$

with a and h in [cm].

Factor s for consideration of the connection height h_m is based on the assumption of stress distribution above of the fasteners according to Fig. 3. The distance h_i of the fastener i from the unloaded beam edge.

$$s = \frac{1}{n} \sum_{i=1}^{n} \left(\frac{h_1}{h_i} \right)^2 \tag{3}$$

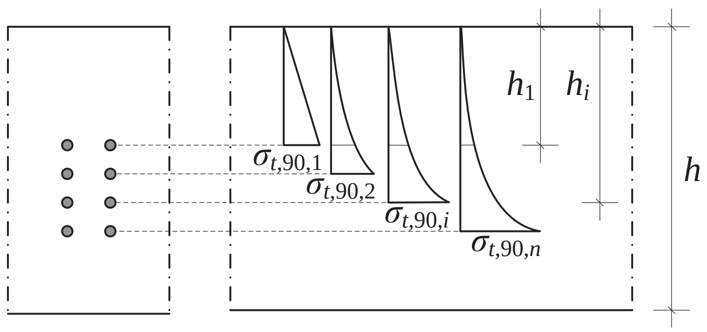


Fig. 3: Distribution of tension perpendicular to grain stresses in different rows n of the connection according to Möhler and Siebert (1980)

The effective beam width b_{eff} was equal to the beam width b. If the beam is loaded only partially over the beam width due to e.g. a small penetration depths of the fasteners the effective beam width should be reduced $b_{eff} \leq b$. The connection width W' are specified as follows:

$W' = m \cdot 2.15d$	for shear connectors of type Appel
$W' = m \cdot 5d$	for dowels
$W' = m \cdot 5d$	for nails

3.1.2 Ehlbeck et al. (1989)

The approach in Eq. 1 was further developed by (Ehlbeck et al. 1989, Ehlbeck and Görlacher 1991). An additional parameter accounting for the connection width was included. The approach was limited to relative connection heights $\alpha \leq 70\%$ based on the observation from tests (Möhler and Lautenschläger 1978, Möhler and Siebert 1980, Ehlbeck and Görlacher 1983). In the test it was observed that connections with stiffer fasteners reach higher load-carrying capacities.

The basic equation is based on a verification of tension perpendicular to grain stresses:

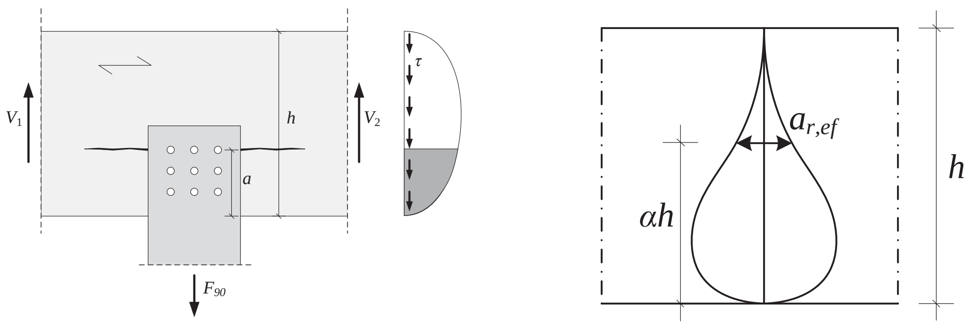
$$\sigma_{t,90,d} = \eta k_r \frac{F_{90,d}}{A_{ef}} \leq 15 A_{ef}^{-0.2} f_{t,90,d} \tag{4}$$

The factor η describes the amount of tension perpendicular to the grain stresses that result from the portion of shear stresses according to beam theory.

$$\eta = 1 - 3\left(\alpha\right)^2 + 2\left(\alpha\right)^3 \tag{5}$$

The factor k_r is similar to Equation 3 by (Möhler and Siebert 1980).

$$k_r = \frac{1}{n}\sum_{i=1}^{n}\left(\frac{h_1}{h_i}\right)^2 \tag{6}$$



(a) (b)

Fig. 4: Impact of the relative connection height α on tension perpendicular to grain stresses (a) and impact of the width of the connection (b) according to Ehlbeck et al. (1989)

The width of the effective width is accounted for by the area loaded in tension perpendicular to grain $A_{ef} = a_r \cdot b$ and an additional factor ch.

$$a_{r,ef} = \sqrt{a_r^2 + \left(ch\right)^2} \tag{7}$$

For connections with only one column of fasteners the theoretical width $a_r = 0$ is increased by the empirically determined value ch for the effective width $a_{r,ef}$ as follows:

$$c = \frac{4}{3}\sqrt{\alpha\left(1-\alpha\right)^3} \tag{8}$$

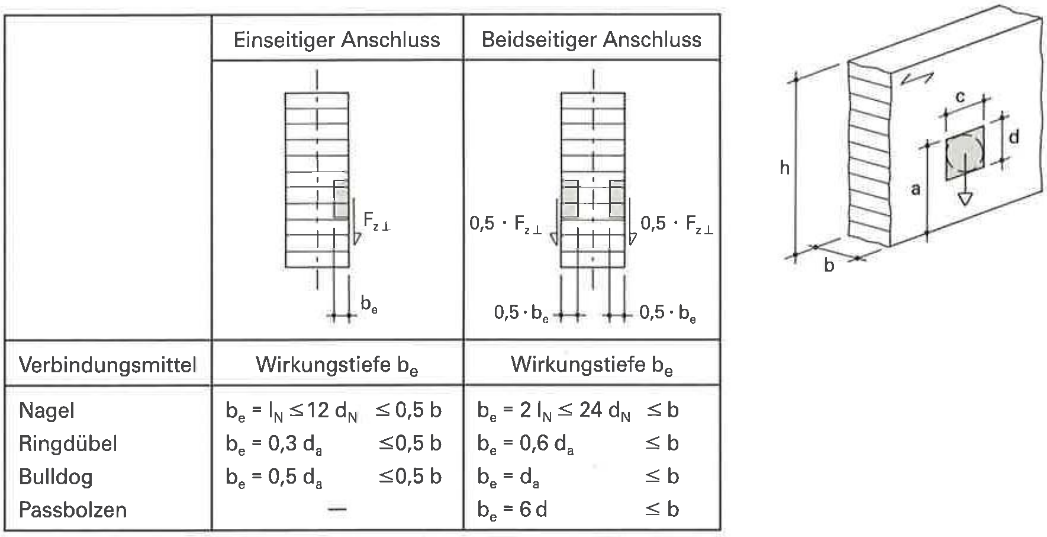
Factor c is derived from the assumed stress distribution according to Fig. 4 (b). For two single connections with a distance l_l between each other the effective total width can be derived as follows.

$$a_{r,ef,total} = a_{ef,r}\left(1 + \frac{l_l}{l_l + a_r}\right) \tag{9}$$

For connections at a cantilever only half of the effective width of the connection is accounted for in case the connection has a distance less than half the beam height from the beam end.

3.1.3 Lignum (1990)

A fully empirical approach was given in Lignum Holzbautabellen 2 (Lignum 1990), that was calibrated from experiments from (Möhler and Siebert 1981, 1983, Ehlbeck and Görlacher 1985) as discussed in Gehri (1988). The approach was developed mainly for the purpose to give information for the design of 3D nail plates like hold-down or joist hangers and not for larger connection loaded perpendicular to the grain. The approach accounts for different types of fasteners (Fig. 5).



	Einseitiger Anschluss	Beidseitiger Anschluss
Verbindungsmittel	Wirkungstiefe b_e	Wirkungstiefe b_e
Nagel	$b_e = l_N \leq 12\, d_N \quad \leq 0{,}5\, b$	$b_e = 2\, l_N \leq 24\, d_N \quad \leq b$
Ringdübel	$b_e = 0{,}3\, d_a \quad \leq 0{,}5\, b$	$b_e = 0{,}6\, d_a \quad \leq b$
Bulldog	$b_e = 0{,}5\, d_a \quad \leq 0{,}5\, b$	$b_e = d_a \quad \leq b$
Passbolzen	—	$b_e = 6\, d \quad \leq b$

Fig. 5: Effective width b_e in dependency of the diameter and type of the fastener and denotations of the geometrical parameters at a connection loaded perpendicular to the grain according to (Lignum 1990).

The strength parameter 0.025 is based on a safety margin of 4 compared to test results.

$$F_{90} = 0.025 a^{0.3} b_e^{0.7} d^{0.4} \left(\frac{a}{h}\right)^{0.2} \tag{10}$$

with
a, b_e, d, h in [mm]
F_{90} in [kN]

The impact of the width of the connection is stated to be low and could be accounted for by the factor $(1 + c/a)^{0.1}$ in case more detailed design is required. For connections at cantilever beams only half the load-carrying capacity is accounted for.

3.2 Fracture mechancics based approaches

3.2.1 van der Put (1990)

A design approach for connections loaded perpendicular to the grain was presented by (van der Put 1990, van der Put and Leijten 2000). The model is based on the crack formation and propagation starting at a single dowel as shown in Fig. 6. Hence, the geometry of the connection is not accounted for. The approach is based on a 2D model and assumes a linear dependency from the beam width.

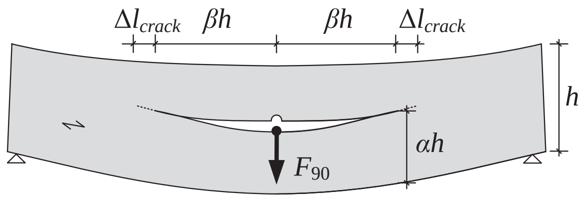


Fig. 6: Model of a beam with a connection perpendicular to the grain and a crack propagating at a single fasteners.

The critical load for crack progression was derived by van der Put (1990) from the equilibrium of energies during infinitesimal crack growth by Δl_{crack} for a crack length $\beta h = 0$.

$$F_{90} = 2b\sqrt{\frac{GG_c\alpha h}{\frac{3}{5}(1-\alpha)}} \tag{11}$$

A more generalized approach was presented by Jensen et al. (2003) for crack length $\beta h > 0$. In addition Jensen et al. proposed design equations for adjacent connections with different distance, see also Fig. 7.

$$F_{90} = 2b\sqrt{\frac{GG_c\alpha h}{\frac{3}{5}(1-\alpha)+\frac{3}{2}\left(\frac{\beta}{\alpha}\right)^2\frac{G}{E}(1-\alpha^3)}} \tag{12}$$

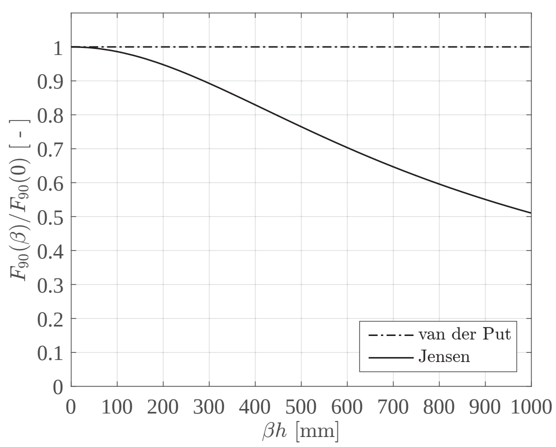


Fig. 7: Relative reduction of crack propagation load with increasing crack length βh for $\alpha = 0.6$, $E = 11500$ N/mm^2 and $G = 650$ N/mm^2.

The calibration of the parameter $\sqrt{GG_c}$ is matter of ongoing discussion. Values in the range of $C_{1,c} \approx 10$ N/mm$^{1.5}$ are discussed in (van der Put and Leijten 2000, Leijten and Jorissen 2001).

The value currently given in EN 1995-1-1 (CEN 2004) is considered to overestimate the load-carrying capacity of connection loaded perpendicular to the grain (e.g. Schoenmakers (2010), Jensen and Quenneville (2011), Jockwer et al. (2015)).

3.2.2 Larsen and Gustafsson (2001)

The duration of load (DOL) effect on the material parameter $C_{1,c}$ was studied in a test series by Larsen and Gustafsson (2001) on tension specimen loaded by dowel connections perpendicular to the grain. The parameter was calculated from the tests according to Equation 13.

$$F_{ult} = 2bC_{Larsen}\sqrt{\alpha h} \tag{13}$$

where

$$C_{Larsen} = \sqrt{\frac{2}{\beta_s} GG_f}$$

The shear correction factor β_s varies between $\beta_s = 1$ for the tests on single rows of fasteners ($m = 1$) in the tests by Larsen and Gustafsson (2001) and the common values according to beam theory $\beta_s = 6/5$ for the evaluation of test results from literature yielding to $C_{Larsen} = C_1$.

3.2.3 Jensen et al. (2012)

Jensen did extensive studies on various problems related to fracture due to loading perpendicular to the grain. A summary of different models for connections loaded perpendicular to the grain based on quasi-nonlinear fracture mechanics (QNLFM) theory is given in (Jensen et al. 2012). The theory can be illustrated as the theory of a beam on elastic foundation. The softening behaviour during fracture is accounted for by this approach, which yields to more complex equations.

$$P_{90} = \lambda P_{90,LEFM} \tag{14}$$

$$\lambda = \frac{\sqrt{2\zeta + 1}}{\zeta + 1} \tag{15}$$

$$P_{90,LEFM} = 2bC_1\sqrt{\frac{\alpha h}{1-\alpha}} \tag{16}$$

and

$$C_1 = \sqrt{\frac{5}{3} GG_c} \tag{17}$$

and

$$\zeta = \frac{C_1}{f_{t,90}}\sqrt{\frac{10G}{\alpha h E}} \tag{18}$$

For a connection at a cantilever beam a modification of the approach given in Jensen (2005) is proposed:

$$P_{90,w} = P_{90,LEFM} min \begin{cases} \frac{1}{2\sqrt{2\zeta+1}} + \frac{bf_t l_e}{P_{90,LEFM}} \\ \frac{2\sqrt{2\zeta+1}}{\zeta+1} \end{cases} \tag{19}$$

This equation was verified for moment connections in spans between two beam parts.

3.2.4 Ballerini (2004)

Ballerini (2004) included parameters f_w and f_r accouting for the width (a_r) and the height (h_m), respectively, in modified versions of the design approach by van der Put (1990) and Jensen et al. (2003). A different power of the relative connection height was based on a better fit with experiments.

$$F_{ult} = 2bC_1\sqrt{\frac{\alpha h}{(1-\alpha^3)}} f_w f_r \tag{20}$$

where:

$$f_w\left(a_r, l_l, h\right) = min \begin{cases} 1 + 0.75\left(\frac{a_r + l_l}{h}\right) \\ 2.2 \end{cases} \tag{21}$$

$$f_r\left(n, h_m\right) = 1 + 1.75\frac{\kappa}{1+\kappa} \tag{22}$$

$$\kappa = \frac{nh_m}{1000} \tag{23}$$

3.2.5 Franke and Quenneville (2011)

The distinction between mode 1 and mode 2 fracture modes was accounted for in the design approach proposed by Franke and Quenneville (2011), which is based on numerical models.

$$F_{90} = \frac{b}{\frac{G^I_{norm}}{G^I_c} + \frac{G^{II}_{norm}}{G^{II}_c}} k_r \tag{24}$$

G^I_c and G^{II}_c are the critical energy release rates of mode 1 (tension perpendicualr to grain) and mode 2 (shear) failure. The normalized fracture energies G^i_{norm} are developed based on numerical studies and calibrated by tests.

$$G^I_{norm} = e^{\left(h^{-1}\left(200-10(\alpha h)h^{-0.25}-a_r\right)\right)} \quad \text{for solid timber and glulam} \tag{25}$$

$$G^I_{norm} = e^{\left(0.8-1.6(\alpha h)h^{-1}-1\cdot 10^{-3}a_r\right)} \quad \text{for LVL} \tag{26}$$

$$G^{II}_{norm} = \left(0.05 + 0.12\alpha + 1\cdot 10^{-3}a_r\right) \quad \text{for solid timber, glulam and LVL} \tag{27}$$

The impact of the number of rows n of fasteners is accounted for by factor k_r:

$$k_\mathrm{r} = \begin{cases} 1 & \text{for} \quad n = 1 \\ 0.1 + \left(arctan\left(n\right)\right)^{0.6} & \text{for} \quad n > 1 \end{cases} \tag{28}$$

3.2.6 Zarnani and Quenneville (2013)

Zarnani and Quenneville (2013) presented an approach that considered the crack length along the fiber direction. Depending on the slenderness of the fasteners, full (corresponding to $P_{s,b}$) or partial cracking over the width (corresponding to $P_{s,tef}$) of the beam can be assumed. For very stout dowels embedment failure can be expected. The effective width of the connection is denoted $w_{net} = a_r - m \cdot d$ and the distance of the connection to the unloaded endgrain to the left or right is denoted as $a_{3,c,L}$ or $a_{3,c,R}$, respectively.

$$P_w = n_p min \left\{ P_{s,tef}; P_{s,b} \right\} \tag{29}$$

$$P_{s,tef} = C_t f_{tp} t_{ef} \left[w_{net} + min\left(\beta\alpha h, a_{3c,L}\right) + min\left(\beta\alpha h, a_{3c,R}\right) \right] \tag{30}$$

where

$$C_{\mathrm{t}} = \begin{cases} 1.264\zeta^{-0.37} & \text{, für} \quad \zeta < 1.9 \\ 1 & \text{, für} \quad \zeta \geq 1.9 \end{cases} \text{, mit} \quad \zeta = \tfrac{a_{4c}}{a_2(n_c-1)} \tag{31}$$

$$P_{s,b} = \eta b C_{fp} \sqrt{\frac{\alpha h}{1-\alpha}} \tag{32}$$

where

$$\eta = \frac{min\left(w_{net} + \gamma\alpha h, a_{3c,L}\right) + min\left(\gamma\alpha h, a_{3c,R}\right)}{2\gamma\alpha h} \tag{33}$$

The parameter η amounts $\eta = 1$ for a single fastener in midspan. A reduction of load-carrying capacity can be observed at a distance to the beam end in cantilever beams of $a_{3c}/(\alpha h) = 4$ (for LVL) and $a_{3c}/(\alpha h) = 2.7$ (for glulam). Therefore, the effective crack length coefficient γ for full separation is $\gamma = 4$ for LVL and $\gamma = 2.7$ for glulam.

3.2.7 Additional design approaches

Additional, mostly empirical design approach can be found e.g. in (Quenneville and Mohammad 2001, Lehoux and Quenneville 2004). These approaches are quite specific with regard to the type of connection used and the timber properties. They are not considered in the further analysis.

3.3 Design approaches in standards

3.3.1 DIN 1052

The approach in DIN 1052 (DIN 2008) is based on the studies by Möhler and Lautenschläger (1978), Möhler and Siebert (1980) and Ehlbeck et al. (1989).

$$F_{90,d} \leq R_{90,d} = k_s k_r \left(6.5 + 18\alpha^2\right) \left(t_{ef} h\right)^{0.8} f_{t,90,d} \tag{34}$$

where

$$k_s = max \begin{cases} 1 \\ 0.7 + \frac{1.6 a_r}{h} \end{cases} \tag{35}$$

and

$$k_r = \frac{n}{\sum_{i=1}^{n} \left(\frac{h_1}{h_i}\right)^2} \tag{36}$$

In addition the following specifications are made:

- Connections with small relative connection height $\alpha < 0.2$ are only allowed for short duration of load (e.g. uplift by wind actions)
- Highly loaded connections with very large connection width $a_r/h > 1$ and $F_{90,d} > 0.5 R_{90,d}$ have to be reinforced.
- For multiple connections along the direction of the beam axis with a distance $l_l \geq 2h$ the individual resistance $R_{90,d}$ can be assumed for each connection.
- For multiple connections along the direction of the beam axis with a distance $0.5h \geq l_l$ the total resistance of the group of connections must not exceed $R_{90,d}$.
- For two connections with a distance along the direction of the beam axis of $0.5h < l_l < 2h$ the individual resistance $R_{90,d}$ of each connection has to be reduced by the factor k_g according to Equation 37.

$$k_g = \frac{l_l}{4h} + 0.5 \tag{37}$$

- Two or more adjacent connections with a distance along the direction of the beam axis of $l_l < 2h$ have to be reinforced if $F_{90,d} > 0.5 k_g R_{90,d}$.
- Connections at cantilever beams with an end grain distance $a_1 < h$ have to be reinforced if $F_{90,d} > 0.5 \cdot R_{90,d}$.

The effective penetration depth t_{ef} of the fasteners is for double sided connections:

$t_{ef} = min\,\{b; 2t; 24d\}$	Timber/Timber connections with nails or screws
$t_{ef} = min\,\{b; 2t; 30d\}$	Steel/Timber connection with nails
$t_{ef} = min\,\{b; 2t; 12d\}$	Dowelled or bolted connections
$t_{ef} = min\,\{b; 100 \text{ mm}\}$	Connections with shear or split ring connections etc.
$t_{ef} = min\,\{b; 6d\}$	Connections with glued-in rods

The effective penetration depth t_{ef} of the fasteners is for single sided connections:

$t_{ef} = min\,\{b; t; 12d\}$	Timber/Timber connections with nails or screws
$t_{ef} = min\,\{b; t; 15d\}$	Steel/Timber connection with nails
$t_{ef} = min\,\{b; t; 6d\}$	Dowelled or bolted connections
$t_{ef} = min\,\{b; 50 \text{ mm}\}$	Connections with shear or split ring connections etc.

3.3.2 EN 1995-1-1

The approach given in EN 1995-1-1 (CEN 2004) is based on the studies by van der Put (1990). The approach is based on a verification of shear stresses in the beam cross-section as shown in Fig. 8. Hence, for connections outside midspan with an unsymmetric spread of shear force the higher of the shear force values on the sides ($F_{v,Ed,1}$) or ($F_{v,Ed,2}$) is decisive for the load carrying capacity of the connection. For connections at a cantilever the force transmitted to the support is taken into account directly.

$$F_{v,Ed} \leq F_{90,Rd} \tag{38}$$

where

$$F_{v,Ed} = max \begin{cases} F_{v,Ed,1} \\ F_{v,Ed,2} \end{cases} \tag{39}$$

and

$$F_{90,Rk} = 14bw\sqrt{\frac{\alpha h}{1-\alpha}} \tag{40}$$

For punched metal plate fasteners an increase of load-carrying capacity can be accounted for.

$$w = \max\left\{\left(\frac{w_{pl}}{100}\right)^{0.35}; 1\right\} \quad \text{for punched metal plate fasteners,}$$

where w_{pl} width of the nailing plate

$$w = 1 \qquad \text{for all other fasteners}$$

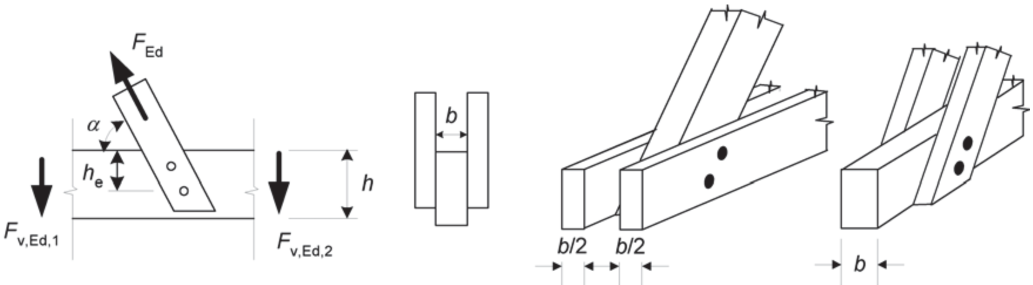


Fig. 8: Connection loaded at an angle to the grain according to EC5 (CEN 2004).

4 Comparison between approaches

4.1 Background

The approaches presented in the previous chapters can be distinguished e.g. according to the underlying theory as shown in Table 1.

Tab. 1: Experimental data from literature used for the evaluation of design approaches for connections loaded perpendicular to the grain

Type	Strength Theory	Fracture Mechancis
Research papers	Möhler and Siebert (1980)	van der Put (1990)
	Ehlbeck et al. (1989)	Larsen and Gustafsson (2001)
	Gehri (1988)	Jensen (2003)
		Jensen et al. (2012)
		Ballerini (2004)
		Franke and Quenneville (2011)
		Zarnani and Quenneville (2013)
Standards	DIN (2008)	CEN (2004)

The approaches show different degree of detail with regard to geometrical and material parameters. The beam height h, the relative connection height α and the beam width b are accounted for by all approaches but not the geometry of the connections. The geometry of the connection can be described by the height h_m and width a_r of the connection and the number of rows n and number of columns m of fasteners. In addition the location of the connection along the span of the beam is of relevance. A summary of the parameters taken into account by the approaches is given in Table 2.

4.2 Material properties used in the approaches

Different material properties are used in the approaches due to their different underlying theory. The approaches by Möhler and Siebert (1981) and Gehri (1988) are based on strength theory and were fit to experimental data. The material parameters in these approaches are related to tensile strength perpendicular to the grain, however, a direct connection to $f_{t,90}$-values determined according to e.g. EN 408 (CEN 2010) is not possible. In contrast, the approach by Ehlbeck et al. (1989) uses the general value of tensile strength perpendicular to the grain. The definition of this value accounts for different impacts like e.g. size effects (Mistler 1998, Aicher et al. 2002) or effects from variations of moisture content and duration of load (Aicher and Dill-Langer 1997, Aicher et al. 1998). Hence, the use of the general value of $f_{t,90,k}$ given in the product standards EN 338 CEN (2009) and EN 14080 (CEN 2013) should be treated with caution.

The approaches based on fracture mechanics by e.g. van der Put (1990), Ballerini (2004) use the material property value based on fracture energy G_I of failure mode 1 in tension perpendicular to the grain No values are specified in EC5 and the related product standards. A comprehensive study for the determination of G_I was performed by Larsen and Gustafsson (1990). Mixed mode fracture and failure in shearing mode 2 requires knowledge of G_{II}, which can be determined e.g. according to (Aicher et al. 1997).

Tab. 2: Overview of parameters accounted for by the approaches for connections loaded in tension perpendicular to the grain.

Approach	Parameter							
	α	h	b	a_r	h_m	l_i	Cantilever	multiple con.
Strength Theory								
Möhler and Siebert (1980)	✓	$\sim h^{0.8}$	$b^{0.8}$	$(a_r+5d)^{0.8}$ for nails & dowels	$\frac{1}{n}\sum_{i=1}^{n}\left(\frac{h_1}{h_i}\right)^2$	✗	✗	✗
Ehlbeck et al. (1989)	$\left(1-3\alpha^2+2\alpha^3\right)$	$h^{0.8}$	$b^{0.8}$	$\sqrt{a_r^2+(\alpha-\alpha^2)\left(\frac{4}{3}h\right)^2}$	$\frac{1}{n}\sum_{i=1}^{n}\left(\frac{h_1}{h_i}\right)^2$	✗	✗	$1+\frac{l_1}{l_1+a_r}$
Gehri (1988)	$\sqrt{\alpha}$	$h^{0.3}$	$b^{0.7}$	$\left(1+\frac{a_r}{\alpha h}\right)^{0.1}$	$h_m^{0.4}$	✗	✗	✗
Fracture Mechanics								
van der Put (1990)	$\sqrt{\frac{\alpha}{1-\alpha}}$	$h^{0.5}$	b	✗	✗	✗	✗	✗
Larsen and Gustafsson (2001)	$\sqrt{\alpha}$	$h^{0.5}$	b	✗	✗	✗	✗	✗
Jensen et al. (2012)	$\sim\sqrt{\frac{\alpha}{1-\alpha}}$	$\sim h^{0.5}$	b	✗	✗	✗	✗	✗
Ballerini (2004)	$\sqrt{\frac{\alpha}{1-\alpha^3}}$	$h^{0.5}$	b	$\min\left\{1+0.75\frac{a_r+l_l}{h};2.2\right\}$	$\left(1+1.75\frac{\kappa}{1+\kappa}\right)$ with $\kappa=\frac{nh_m}{1000}$	✗	✗	$\min\left\{1+0.75\frac{a_r+l_l}{h};2.2\right\}$
Franke and Quenneville (2011)	✓	✓	b	$\sim\frac{a_r}{1000}$	$0.1+(\arctan(n))^{0.6}$ for $n>1$	✗	✗	✗
Zarnani and Quenneville (2013)	$\sqrt{\frac{\alpha}{1-\alpha}}$	$h^{0.5}$	b	✓	✓	✓	✓	✗
Standards								
DIN (2008)	$\left(6.5+18\alpha^2\right)$	$h^{0.8}$	$b^{0.8}$	$\max\left\{1;0.7+\frac{1.4a_r}{h}\right\}$	$\frac{1}{n}\sum_{i=1}^{n}\left(\frac{h_1}{h_i}\right)^2$	✓	✓	$k_g=\frac{l_1}{4h}+0.5$ for $0.5h\leq l_1\leq 2h$
CEN (2004)	$\sqrt{\frac{\alpha}{1-\alpha}}$	$h^{0.5}$	b	✗	✗	✓	✓	✓

✓indicates that the respective parameter is accounted for by the corresponding approach, however a specification would be too complex.

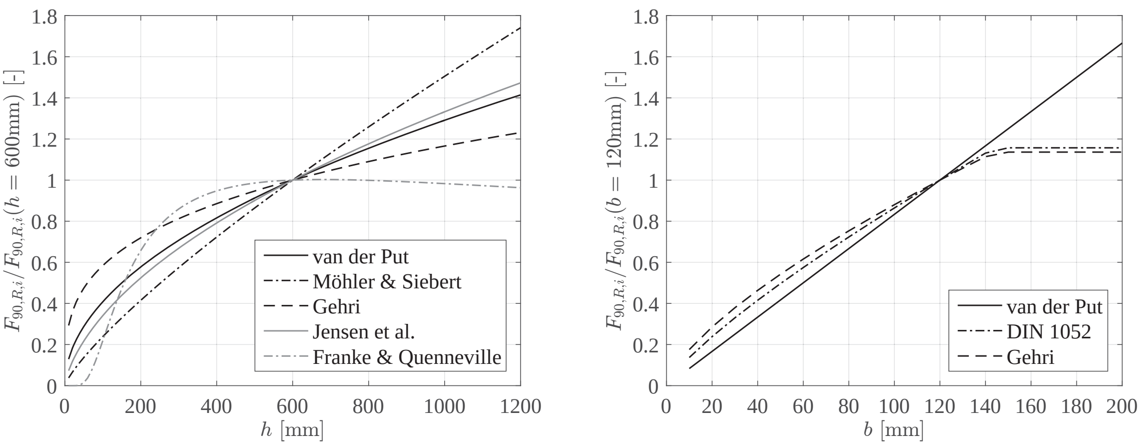
✗indicates that the respective parameter is not accounted for by the corresponding approach.

4.3 Impact of geometric parameters

4.3.1 Beam dimensions h and b

According to fracture mechanics theory the dimensions of a member have a square root impact on its strength. This impact of beam height $h^{1/2}$ is accounted for by the approaches based on fracture mechanics like e.g. (van der Put 1990, Ballerini 2004). Due to the 2 dimensional basis of the approaches only the nonlinear impact of height is considered. The approaches based on non-linear fracture mechanics by (Jensen et al. 2012) and the approach considering mixed mode fracture by Franke and Quenneville (2011) use slightly different height effects. The approaches by Möhler and Siebert (1980), Ehlbeck et al. (1989) consider a volume effect on the tensile strength perpendicular to the grain with $(V/V_{ref})^{0.2}$. This results in a height effect $h^{0.8}$ for the load-carrying capacity of connections loaded perpendicular to the grain.

A comparison of the impact of height on the different approaches is shown in Fig. 9 (a). The relative load-carrying capacity is normalized with regard to a reference height $h = 600$ mm.



(a) (b)

Fig. 9: Impact of the beam height h (a) and beam width b (b) on the relative load-carrying capacity in the approaches.

A linear impact of the beam width is accounted for by all fracture mechanics based design approaches due to the 2 dimensional basis of the problem description. The strength based design approaches by Möhler and Siebert (1980), Ehlbeck et al. (1989) use a non-linear impact of beam width with $b^{0.8}$ due to the considered volume effect. The empirically based design approach by Gehri (1988) uses even a smaller impact of beam width with $b^{0.7}$. The approaches given in DIN 1052 and proposed by Gehri (1988) use an effective beam width in dependency of the type and slenderness of the fastener. The reduced, effective beam width allows amongst others for the consideration of crack initiation due to ductile deformation of the fasteners. This early splitting was studied in detail by Schoenmakers (2010) and accounted for by a reduced embedment strength compared to EC5 values. In the approach by Zarnani and Quenneville (2013) it is distinguished between full and partial splitting of the beam due to failure of the fasteners or splitting failure of the timber. The impact of beam width in the different approaches is shown in Fig. 9 (b). The relative load-carrying capacity is normalized with regard to a reference width $b = 120$ mm. The effective beam width with $b_{ef} = \min(12d, b)$ sets a limit to the load-carrying capacity in the approaches by the approaches by DIN 1052 and Gehri (1988).

4.3.2 Relative connection height α

The impact of the relative connection height α is shown in Fig. 10. For $\alpha > 0.7$ the differences between the approaches increase considerably. Splitting of the timber becomes less relevant in this region and failure of the fasteners may be the relevant failure mechanism. Also for $\alpha < 0.2$ large differences exists between the approaches. The low load-carrying capacity together with the brittle failure mechanism make it difficult to determine the exact structural behaviour of the connection perpendicular to the grain.

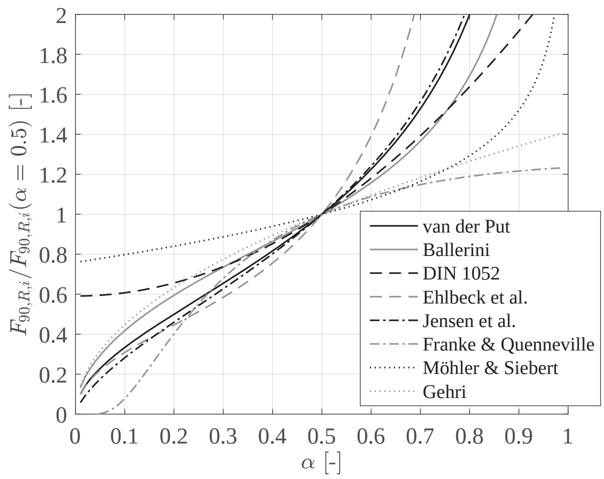


Fig. 10: Impact of the relative connection height α on the relative load-carrying capacity.

4.3.3 Connection geometry m, n, a_r and h_m

The impact of the geometry of the connections is accounted for in the approaches based on strength theory (Möhler and Siebert 1980, Ehlbeck et al. 1989, Gehri 1988, DIN 2008) and the semi-empirical approaches by Ballerini (2004) and Franke and Quenneville (2011). Factors are included in these approaches accounting for the increase in strength with increasing connection height and connection width as summarized in Tab. 2. The comparison of the impact of connection geometry is shown in Fig. 11.

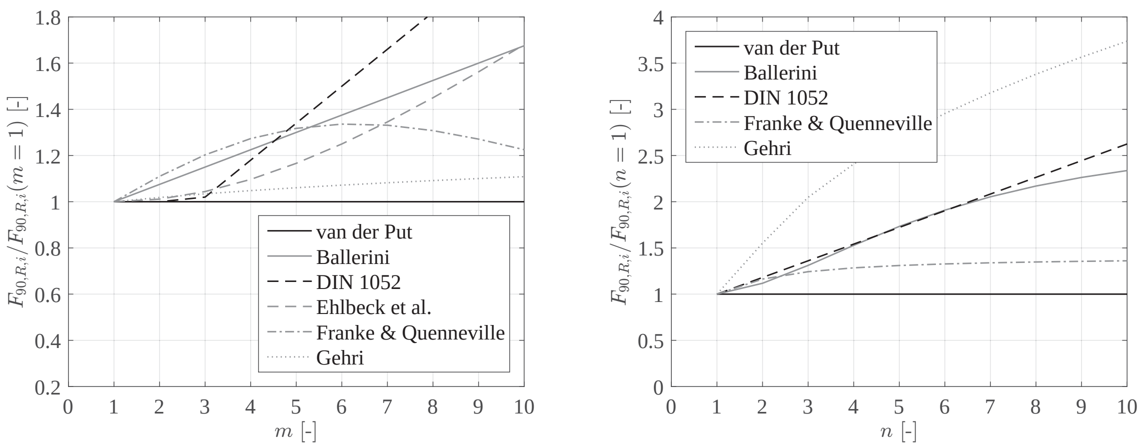


Fig. 11: Impact of the width of the connection $a_r = 5d\,(m-1)$ with the number of fastener columns m (left) and the height of the connection $h_m = 3d\,(n-1)$ with the number of fastener rows n (right) on the relative load-carrying capacities $F_{90,R,i}/F_{90,R,i}\,(ref)$.

5 Benchmarking of design approaches by experiments

5.1 Experimental data reported in literature

A summary of tests on connections loaded perpendicular to the grain reported in literature is given in Tab. 3.

Tab. 3: Tests on connections loaded perpendicular to the grain from literature

Literatur	No. of tests	Species	Fastener type
Glulam			
Möhler and Siebert (1981)	28	Softwood	Nails, dowels, connectors
Ehlbeck and Görlacher (1983)	57	Softwood	Nails
Ballerini (1999)	49	Norway Spruce	Dowels
Ballerini and Giovanella (2003)	72	Norway Spruce	Dowels
Reske (1999)	138	Spruce/Pine, GL 20f-E	Bolts
Kasim (2002)	90	Spruce/Pine & Douglas Fir	Bolts
Habkirk (2006)	50	Spruce/Pine & Douglas Fir	Bolts
Jensen and Quenneville (2011)	18	Douglas Fir, GL8 + GL15	Dowels
Schoenmakers (2010)	69	Spruce	Nails, dowels
Jockwer et al. (2015)	20	Spruce, GL24h	Dowels
Solid Timber			
Möhler and Lautenschläger (1978)	44	Spruce	Nails, dowels
Schoenmakers (2010)	261	Spruce	Nails, dowels

A summary of the dimensions of the beams and the geometry of the connections tested is shown in Fig. 12 and 13. It can be seen, that a large number of tests has been carried out on specimens with $h \leq 300$ mm and $b \leq 100$ mm. The tests cover a good range of relative connection height α with the majority of the tests between $0.2 \leq \alpha \leq 0.7$. A large portion of the tests had less than 5 fasteners ($m \cdot n \leq 4$).

In many of the tests the diameter of the fastener was relatively large compared to the width of the beams. This very stiff configuration with thick fasteners allows for an uniform loading over the entire beam width. However, the load-carrying capacity of the fasteners was often much higher compared to the load observed at brittle failure of the timber. Due to economic reasons it should be aimed at achieving an equal load-carrying capacity of the fasteners and the timber in practice .

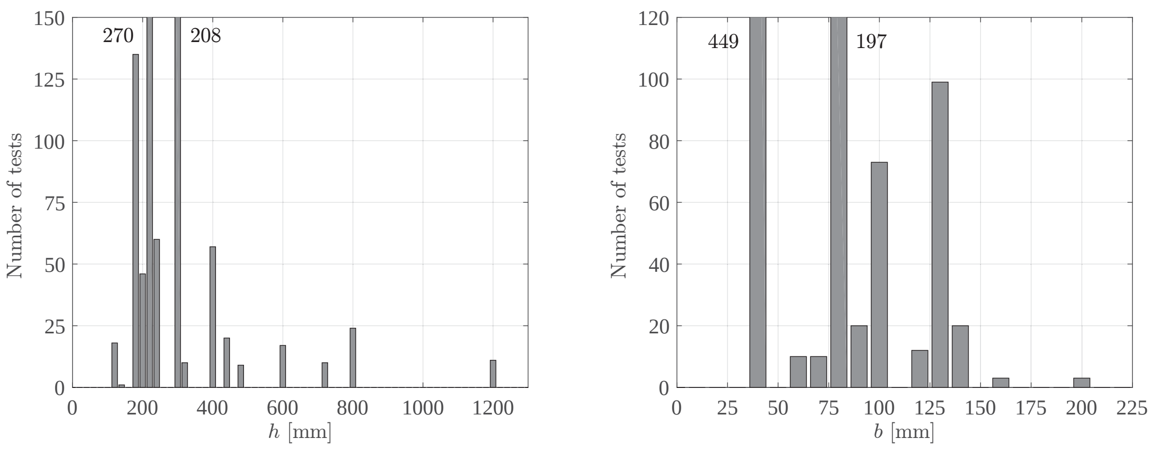


Fig. 12: Histograms of beam parameters height h and width b as studied in the publications listed in Tab. 3.

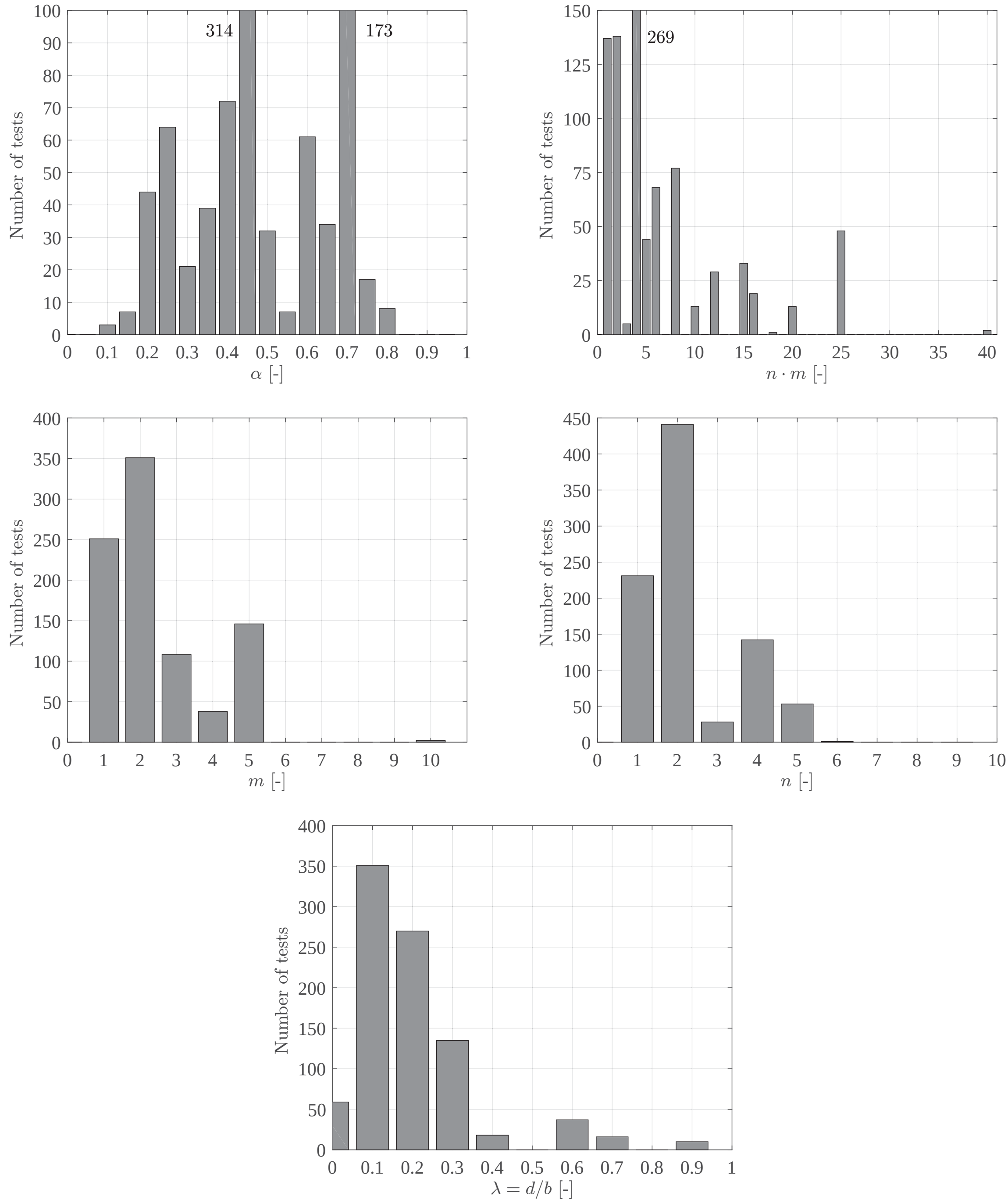


Fig. 13: Histograms of beam and connection parameters relative connection height α, number of fasteners $m \cdot n$, number of fastener columns m, number of fastener rows n and slenderness of the fasteners $\lambda = d/b$ as studied in the publications listed in Tab. 3.

For the evaluation of the impact of geometrical parameters on the load-carrying capacity of connections loaded perpendicular to the grain, those test series are selected from the large number of individual tests available, in which the respective parameter is varied with two or more values and the rest of the geometrical parameters is kept constant. This constraint reduces the number of relevant test series considerably, since in most of the test series two or more parameters were varied at a time.

5.2 Influence of geometric parameters on the load-carrying capacity

5.2.1 Selected design approaches

The influence of geometric parameters on the load-carrying capacity of connections loaded perpendicular to the grain is evaluated in the following sections. For this evaluation the load-carrying capacity determined in the tests is normalized with regard to a reference configuration. The influence of the individual geometric parameters on the load-carrying capacity depends on the type of design approach used for the normalization of the test results. Several design approaches were selected for the more detailed evaluation. The selection is based on the following criteria:

- already existing implementation in design codes,
- consideration of a wide range of geometrical parameters,
- ease of use,
- availability of relevant material parameters.

Based on these criteria the design approach in EC5, the approach by Ballerini and the approach given in DIN 1052 were selected for the further evaluation.

5.2.2 Impact of beam height h

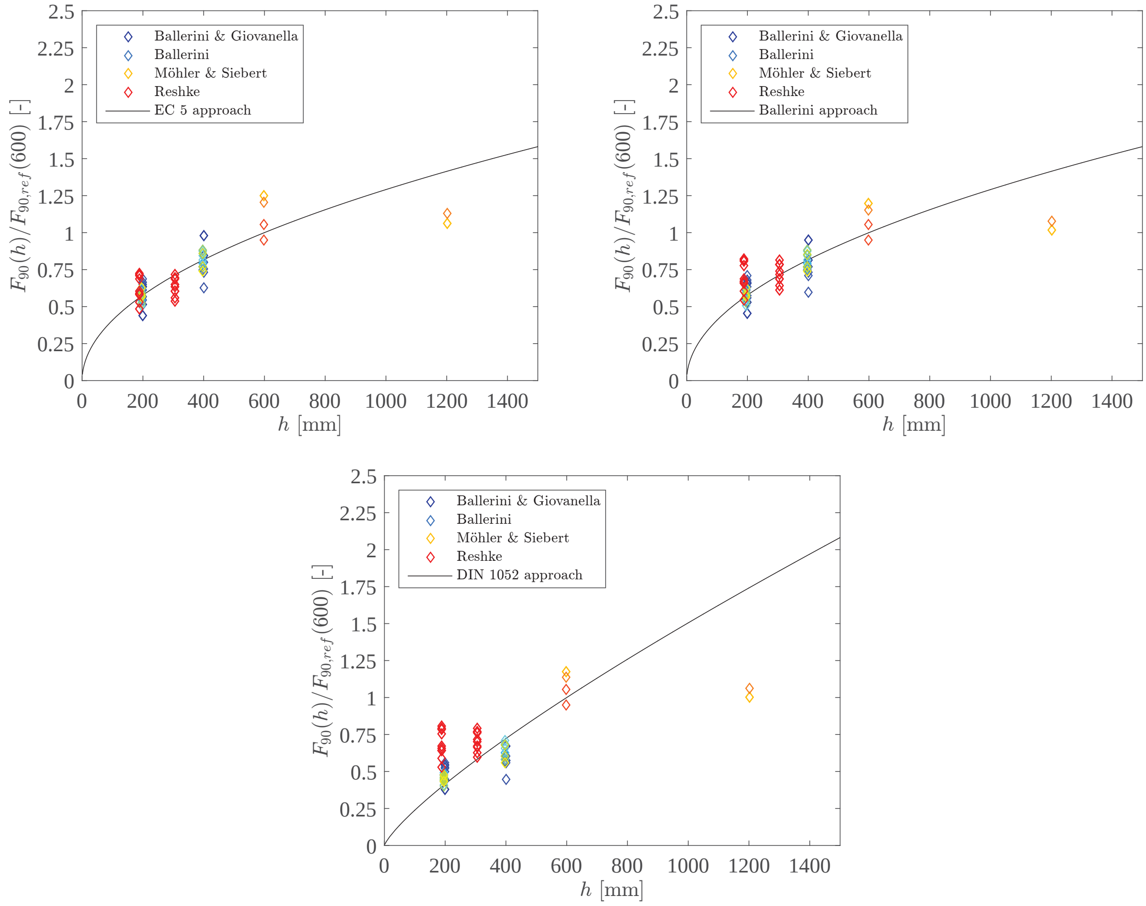


Fig. 14: Impact of beam height h on the relative load-carrying capacity with a reference to $h = 600$ mm.

The relative load-carrying capacity of connections loaded perpendicular to the grain increases with increases beam height h as shown in Fig. 14. The increase, however, is not linearly as according to strength theory but follows more or less a square-root shape as according to fracture mechanics theory. The volume based size effect included in the design approach in DIN 1052 overestimates the impact of beam height on the load-carrying capacity.

The test series with $h = 1200$ mm by Möhler and Siebert (1980) shows a comparatively low load-carrying capacity. This is not considered adequately by any of the approaches. However, too few test results are available for $h \geq 600$ mm to make clear statements on the validity of design approaches for $h \geq 600$ mm.

5.2.3 Impact of beam width b

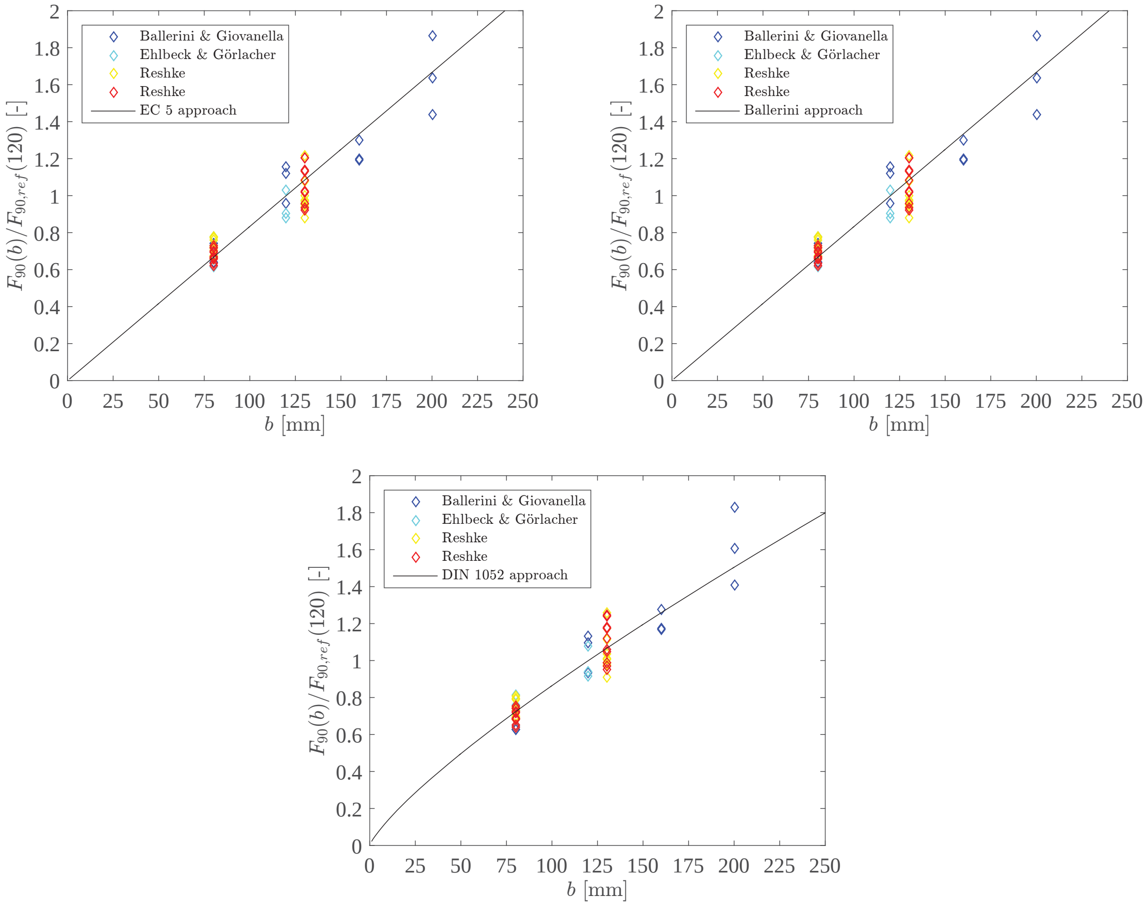


Fig. 15: Impact of beam width b on the relative load-carrying capacity with a reference to $b = 120$ mm.

The width b of the beam has a linear impact on the load-carrying capacity according to fracture mechanics based design approaches. This linear impact fits well the results of the test series shown in Fig. 15. The volume based size effect with $b^{0.8}$ accounted for by the design approach in DIN (2008) fits the results of the test series in a similar good way.

The test series available for studying the impact of beam with b are limited, especially for beam widths $b \geq 150$ mm. A more detailed experimental evaluation of the impact of beam width, especially on partial splitting due to Mode 1, Mode 2 or Mode 3 failure according to Johansen theory (European yield model, EYM) would be desirable. Partial splitting of the beam is already accounted for in the approach by Zarnani and Quenneville (2013). It can be expected, that crack initiation and partial splitting reduces the impact of beam width on the load-carrying capacity as accounted for in the design approaches given in DIN (2008) and by Gehri (1988) especially for large beam width.

5.2.4 Impact of relative connection height α

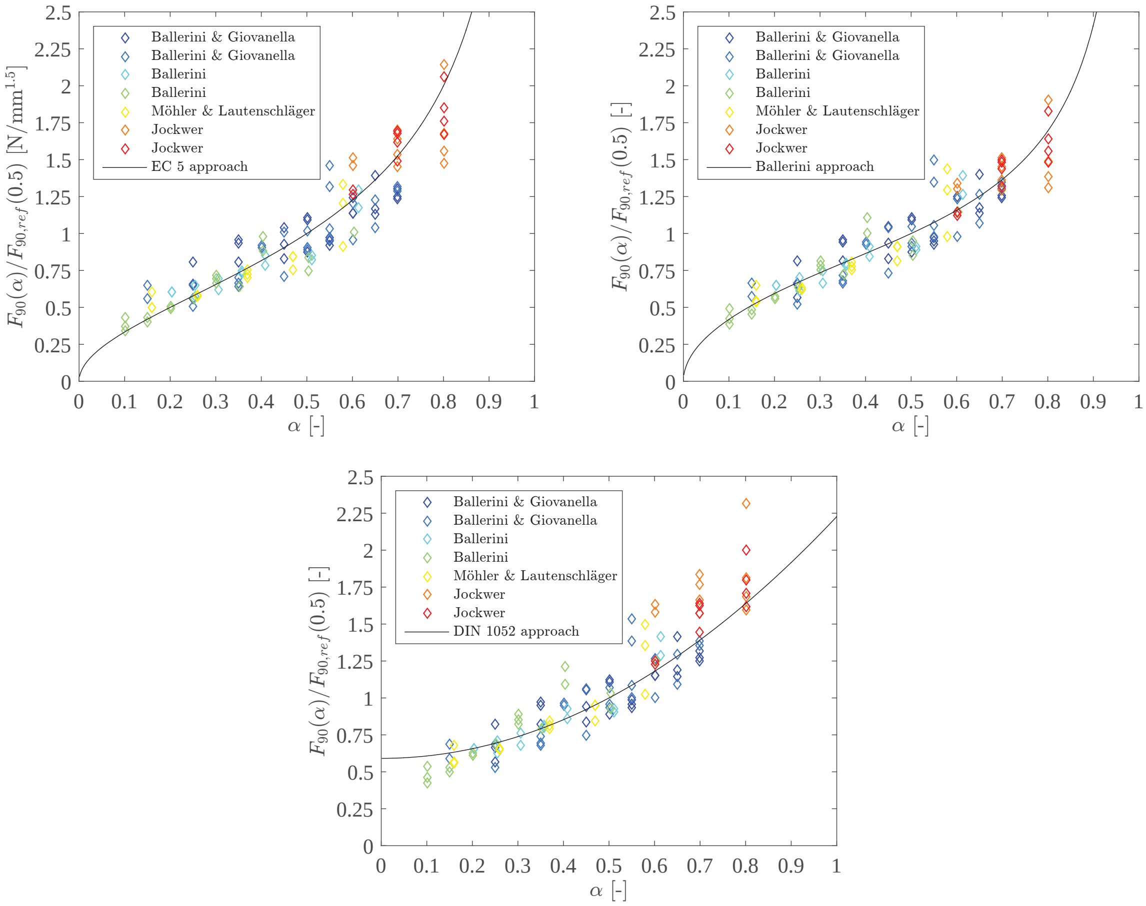


Fig. 16: Impact of relative connection height α on the relative load-carrying capacity with a reference to $\alpha = 0.5$.

The relative connection height α shows a non-linear impact on the relative load-carrying capacity as shown in Fig. 16. The best fit is achieved by the approach by Ballerini (2004). The scatter of the test results around the estimated load-carrying capacity (solid black lines in Fig. 16) can be described by an factor ϵ with a mean value $\mu_\epsilon = 1$. For the approach by Ballerini (2004) this error term exhibits a coefficient of variation $CoV_\epsilon = 12\%$. The approach given in DIN 1052 with $CoV_\epsilon = 14.9\%$ and in EC5 by van der Put (1990) with $CoV_\epsilon = 15.4\%$ have an inferior fit with the test results. These approaches predict a reasonable load-carrying capacity also for small relative connection heights with $\alpha \approx 0$. The approach given in DIN 1052 predicts a non-zero load-carrying capacity for $\alpha = 0$. Therefore it is limited to relative connections height $\alpha \geq 0.2$ for loads longer than short load duration. The disadvantage of this inaccuracy and additional requirement from a pedagogical point of view can be discussed.

For the opposite case of large relative connection heights the design approach given in DIN 1052 predicts a finite load-carrying capacity. The fracture mechanics design approaches predict infinite resistance against splitting for $\alpha = 1$. For these large relative connection heights failure of the fasteners or (shear or bending) failure in the timber cross-section can be expected. The limitation of relevancy of the design approaches in DIN 1052 and EC5 for $\alpha < 0.7$ seems reasonable. An adaptation of the relevant range of the design approaches for splitting of connection perpendicular to the grain can be matter of discussion.

5.2.5 Impact of connection width a_r and m

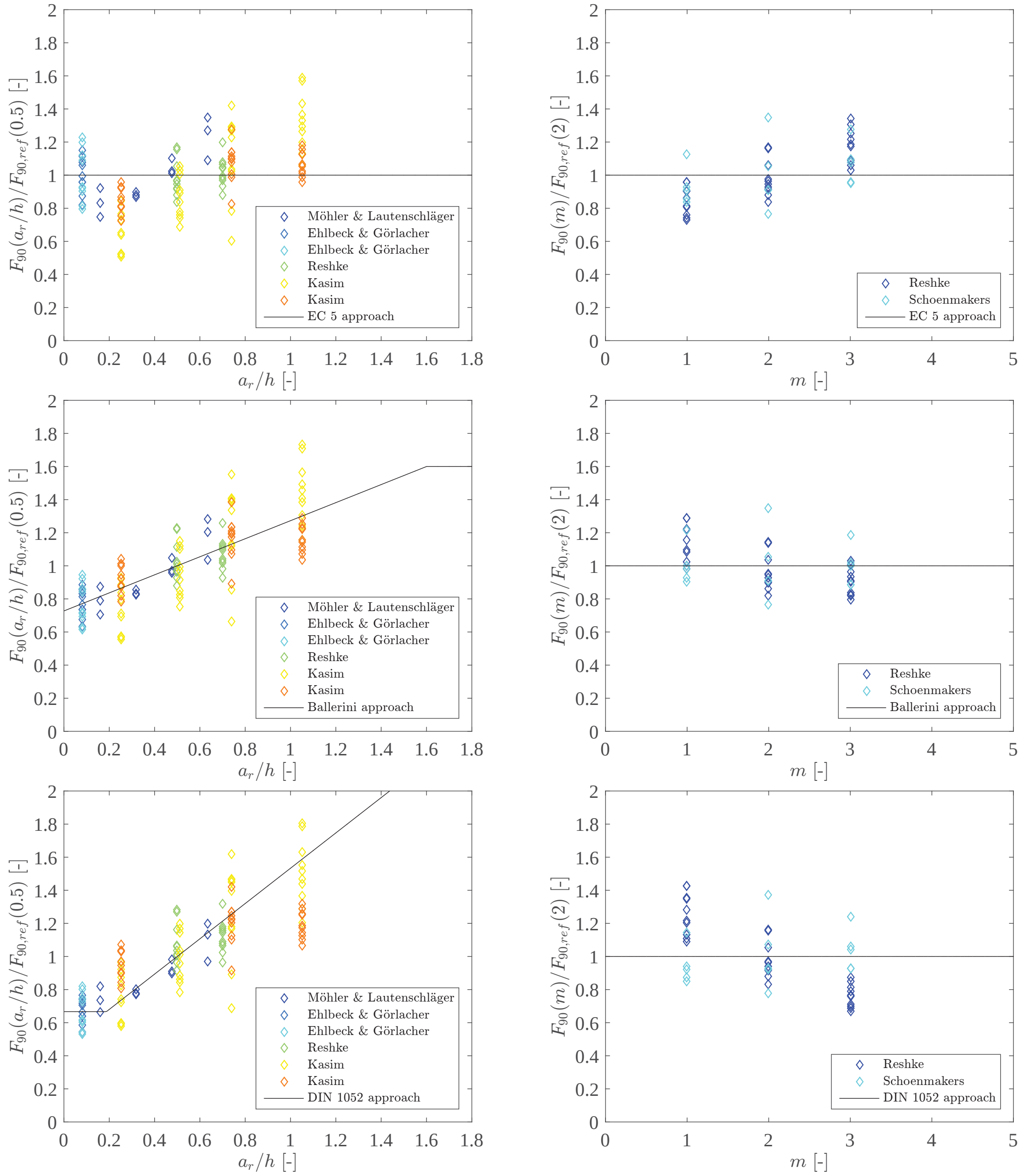


Fig. 17: Impact of connection width a_r/h and number of fastener columns m on the relative load-carrying capacity with a reference to $a_r/h = 0.5$ and $m = 2$.

The relative load-carrying capacity increases with increasing width of the connection as shown in Fig. 17. The approach in EC5 does not account for the effect of geometry of the connection. However, the inaccuracy of the approach for small ratios a_r/h is small. The approach by Ballerini limits the impact of connection width to a maximum factor of 1.6. Connections with a distance $a_r/h > 0.5$ shall be considered as two separate connections according to DIN 1052. This specification is necessary since no upper limitation is given for the factor accounting for the connection width in the approach according to DIN 1052 for large values of a_r/h. The impact of the number of columns of fasteners is not accounted for by any of the approaches studied in this chapter. However, in the test series shown in Fig. 17 the connection width a_r/h was increased with increasing number of columns of fasteners m. This allows to answer the question of the impact of connection width only with limited precision.

5.2.6 Impact of connection height h_m and n

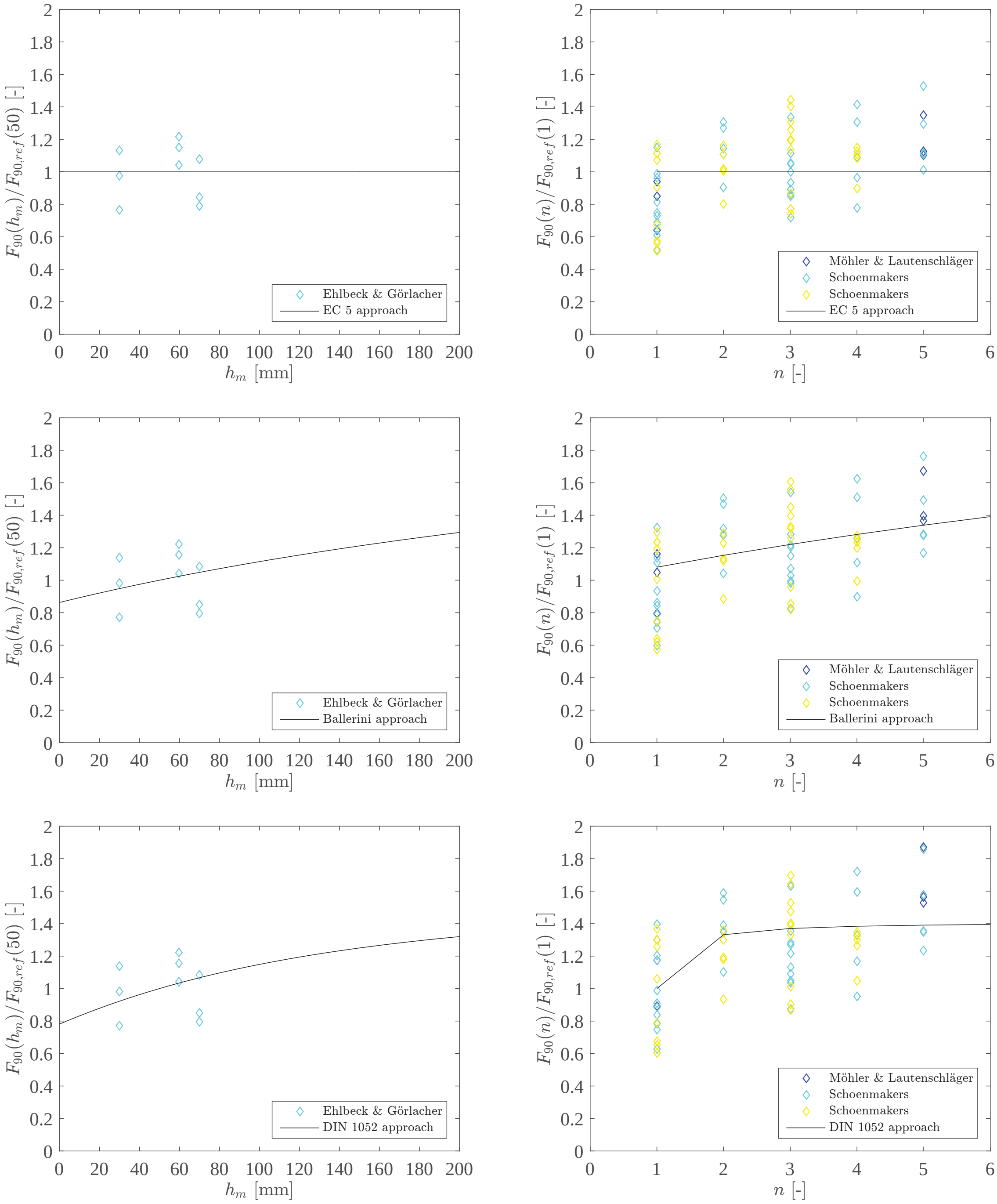


Fig. 18: Impact of connection height h_m and number of fastener rows n on the relative load-carrying capacity with a reference to $h_m = 50$ mm and $n = 1$.

Only one test series exists that allows studying the impact of variable connection height h_m with all other parameters kept constant. From these tests only a limited impact of the connection height h_m on the load-carrying capacity is observed as shown Fig. 18. However, a clear trend of increasing relative load-carrying capacity with increasing number of fastener rows can be observed. The approach given in DIN 1052 shows a fast convergence of the increase of relative load-carrying capacity with increasing number of rows of fasteners.

5.2.7 Impact of position of the connection

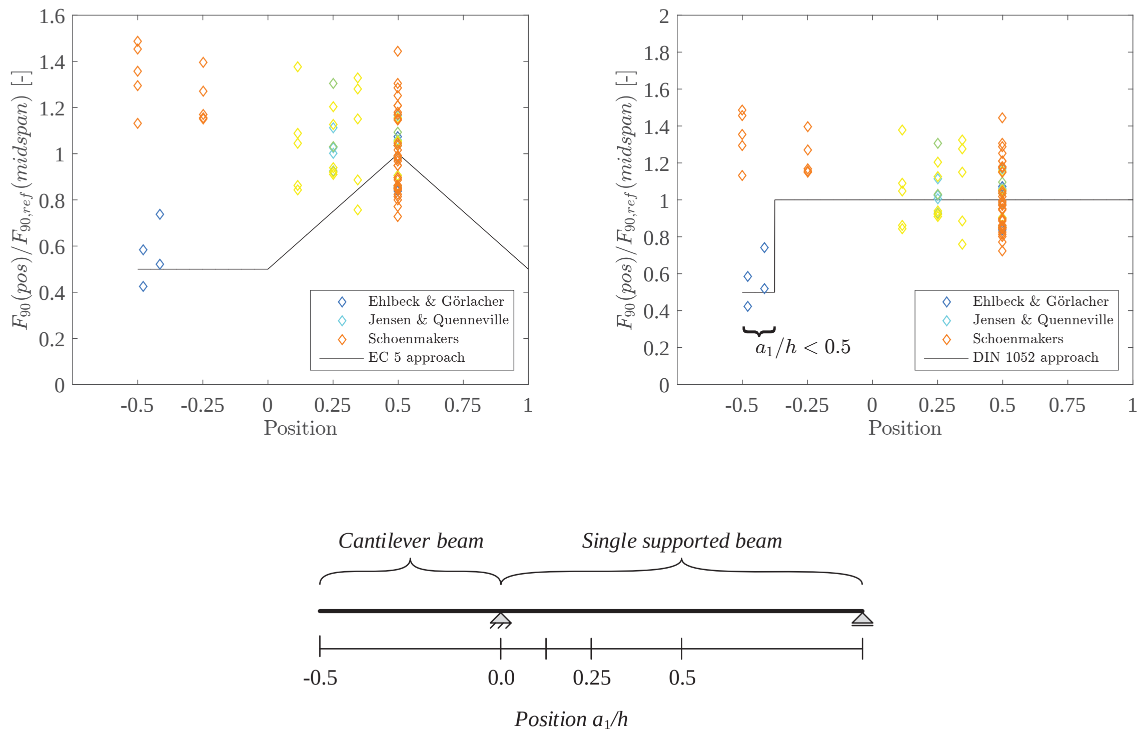


Fig. 19: Impact of the position a_1/h of a connection on the relative load-carrying capacity with a reference to $a_1/h = 0.5$ for single supported beams.

The relative load-carrying capacity of connections loaded perpendicular to the grain at different positions along the beam span of single supported beams and cantilever beams is shown in Fig. 19. The approach by Ballerini is not included in this comparison, since no specification regarding the impact of the position is given for this approach. The load carrying capacity of the EC5 approach is based on the maximum value of shear force on the relevant side of the connection. This leads to a decrease of up to 50% for connections located at positions outside of midspan of single supported beams. The approach given in DIN 1052 assumes a constant load-carrying capacity of the connection independent of it's position along the beam axis. Only for connections located at a distance to the end of a cantilever beam of $a_1/h < 0.5$ the load-carrying capacity is reduced by 50%.

In Fig. 19 only for the test series by Ehlbeck and Görlacher (1983) a reduced load-carrying capacity at the end of a cantilever beam can be observed. The other test series show no impact of the position of the connection along the beam span on the load-carrying capacity.

5.2.8 Impact of the distance between connections

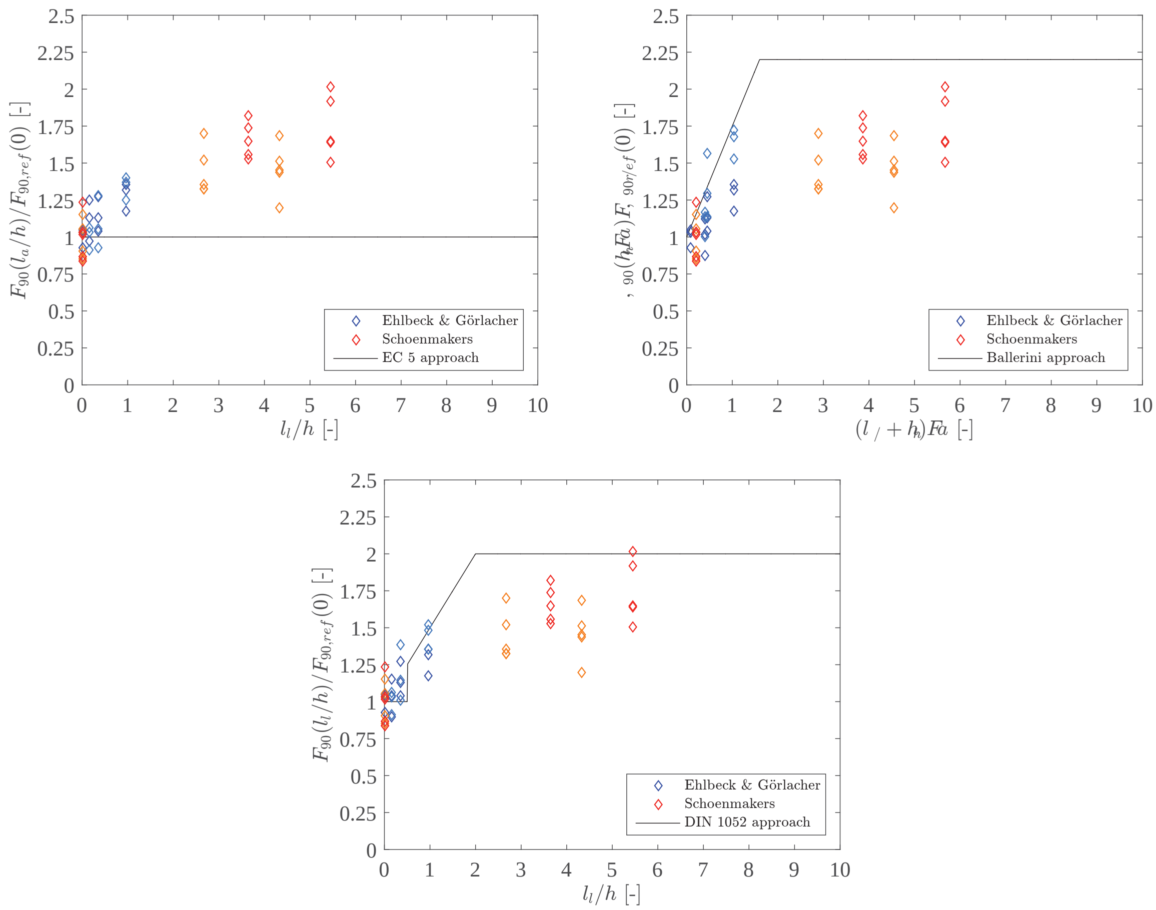


Fig. 20: Impact of the distance between connections l_l on the relative load-carrying capacity.

The total load-carrying capacity of two connections next to each other increases with increasing distance l_l/h as shown in Fig. 20. The approach given in EC5 does not account for this effect and predicts the same total load-carrying capacity for one or multiple connections. The approach by Ballerini allows an increase of the total load-carrying capacity for two connections with a distance $(a_r + l_l)/h$ up to a factor of 2.2 compared to the single connection. The approach given in DIN 1052 allows to design two connections with a distance $l_l > 2h$ as individual connections. Though a remarkable increase of the total load-carrying capacity for two connections can be seen in the tests, the total load-carrying capacity is lower than 2 times the load-carrying capacity of a single connection. This issue should be matter of discussion, especially when accounting for the impact of the total volume loaded in tension perpendicular to the grain in the beam.

6 Evaluation of selected design approaches

A more detailed evaluation with regard to the capability to predict the load-carrying capacity of the whole sample of individual test results available was made for the design approach in EC5, the approach by Ballerini and the approach given in DIN 1052. For this evaluation the material strength parameters $C_{1,i}$ and $f_{t,90,DIN}$ were calculated from the test results according to Eqs. 41 - 43:

$$C_{1,EC5} = \frac{\max\left\{F_{v,90,i}\right\}}{b\sqrt{\frac{\alpha h}{(1-\alpha)}}} \tag{41}$$

$$C_{1,Ballerini} = \frac{F_{90}}{2bf_w f_r\sqrt{\frac{\alpha h}{(1-\alpha^3)}}} \tag{42}$$

$$f_{t,90,DIN} = \frac{F_{90}}{k_s k_r\left(6.5+18\alpha^2\right)\left(b_{ef}h\right)^{0.8}} \tag{43}$$

The evaluation of these material strength parameters gives information about the capability of the design approaches to account for the specific parameter studied. In the ideal case the evaluation of test results with regard to the material strength parameters would yield a constant value. In reality this will not be the case due to a natural variability of the material wood and an inaccuracy of the design approaches. It should be aimed at a model that minimizes the latter reason for deviation of the constant material strength parameters.

6.1 Beam dimensions

The impact of the beam height and the beam width on the material strength parameters is shown Fig. 21. The approach given in EC5 underestimates the load-carrying capacity of beams with large height, which can be seen from the increase of $C_{1,EC5}$ with increasing beam height. The linear and quadratic regression according to the EC5 approach and the approach by Ballerini are almost equal. This shows that no specific value of beam height has an exceptional impact on the material strength parameters. The approach given in DIN 1052 slightly overestimates the load carrying capacity of beams of small height. However, the linear regression shows only a small decreasing trend with increasing beam height.

The beam width is well accounted for by all three approaches. The linear regression functions have almost no slope. The test series by Ballerini with a beam width $b = 40$ mm yields to comparably low material strength parameters according to the EC5 approach. This series corresponds to tests on connections with a single fastener. Since the EC5 approach does not account for the number of fasteners or the geometry of the connection, this series reflects the lower bound of the material strength parameters. The quadratic regression function for the material strength parameters according to EC5 shows a strong curvature which is influenced by this test series with low relative load-carrying capacity.

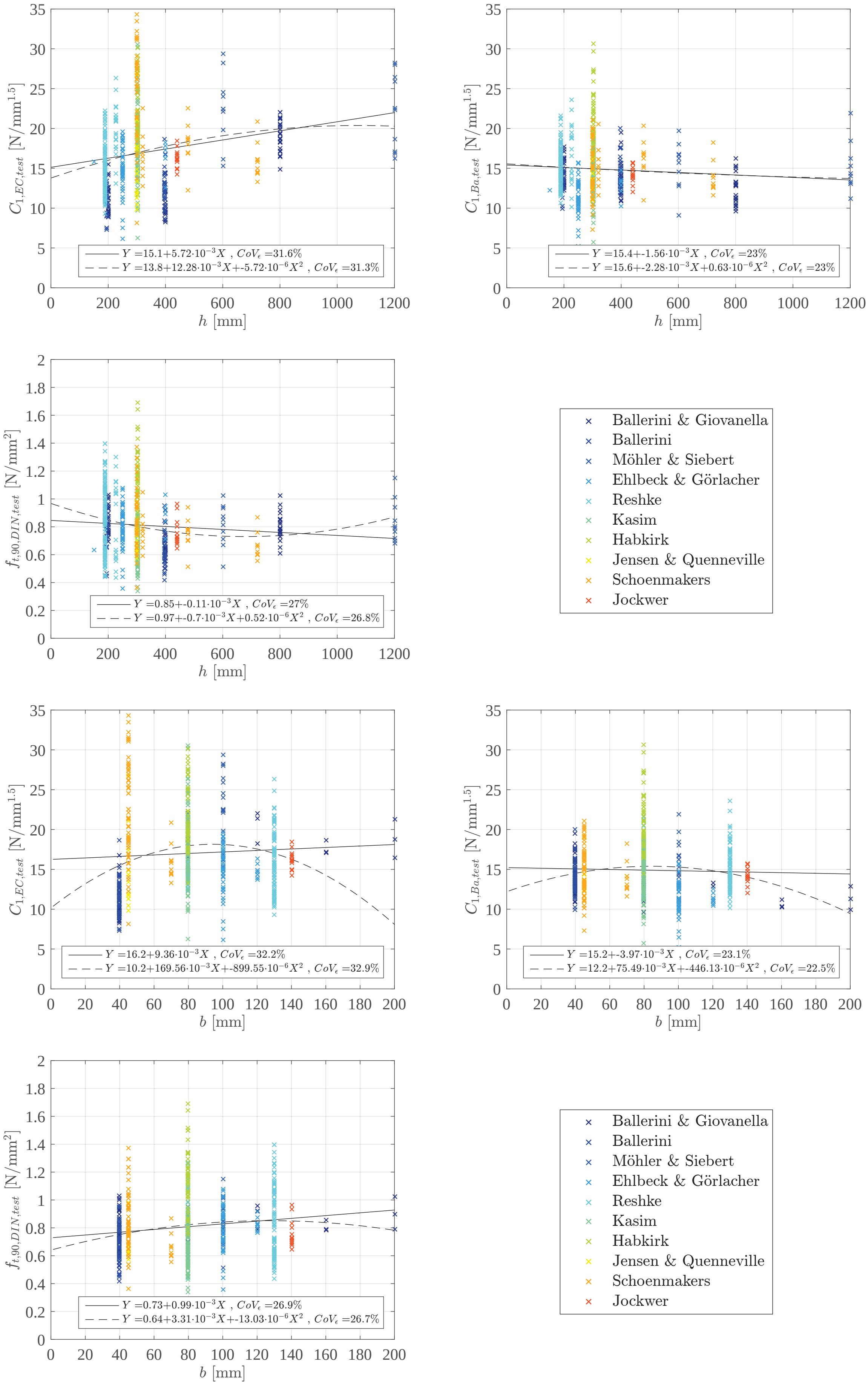


Fig. 21: Evaluation of the material strength parameters according to test results with regard to the beam height h and the beam width b.

6.2 Relative connection height

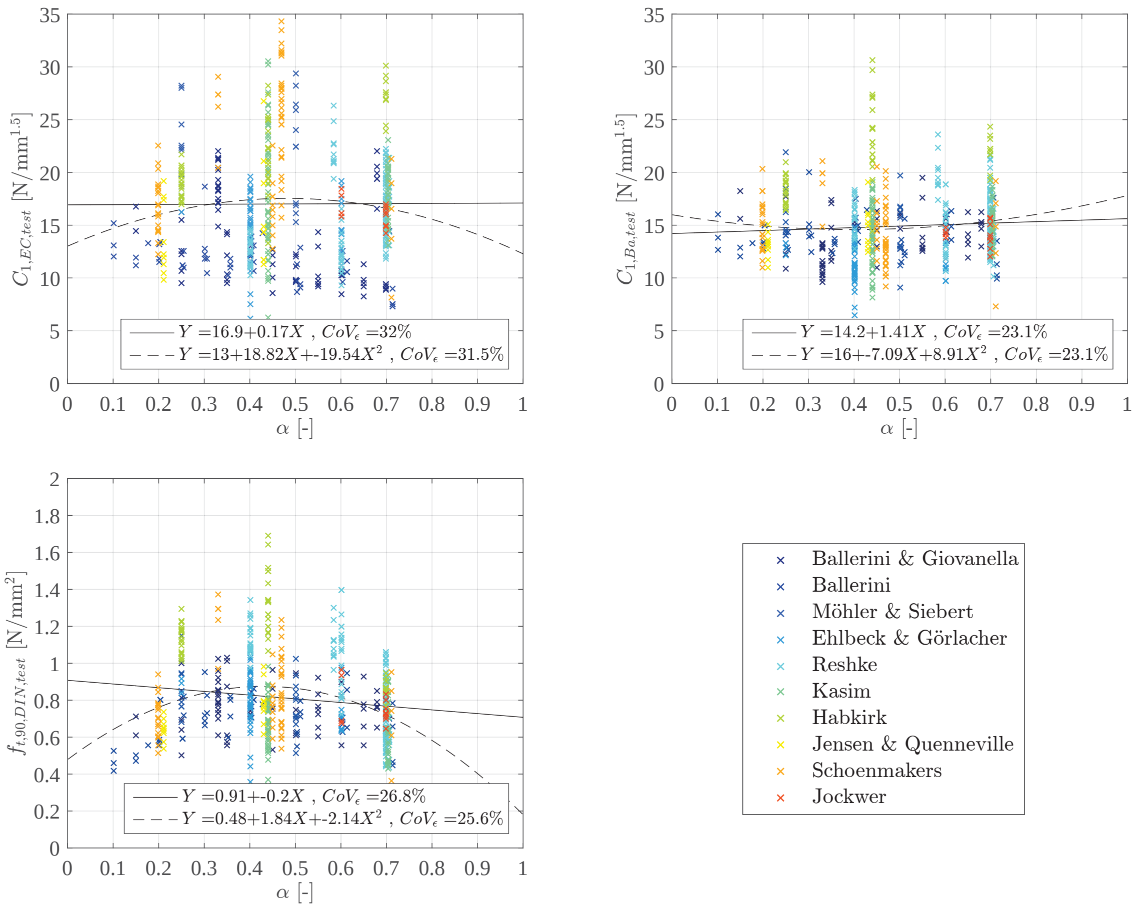


Fig. 22: Evaluation of the material strength parameters according to test results with regard to the relative connection height α.

Both the design approaches given in EC5 and by Ballerini yield to a material strength parameter with an almost horizontal regression with regard to the impact of relative connection height as shown in Fig. 22. The variation of the material strength parameter according to Ballerini is considerably small which reflects the higher precision of this approach. The series of relatively low material strength parameters according to the EC5 approach calculated from the test series by Ballerini corresponds to tests on connections with a single fastener that reflects the lower bound of the material strength parameters for EC5. The approach given in DIN 1052 slightly overestimates the strength of connections with small α.

6.3 Geometry of the connection

The geometry of the connection is accounted for only in the approach by Ballerini and in the approach given in DIN 1052. Hence, the material strength parameter according to these approaches show a much lower dependency on the connection width a_r/h and number of fastener columns m as shown in Fig. 23 and on the connection height h_m/h and number of fastener rows n in Fig. 24.

The quadratic regression function for the material property value according to the approach given in EC5 shows a strong increase for small connection width up to $a_r/h \approx 0.5-0.6$. For connections exceeding this width there is only a small increase in load-carrying capacity to be expected. The material property value according to the design approach given in DIN 1052 shows an opposite behaviour: the quadratic regression function decreases considerably for connection width $a_r/h > 0.5$ and the design approach overestimate the load-carrying capacity for very wide connection. This behaviour is due to the constantly increasing parameter k_s for larger connection width a_r as already shown in Fig. 11 and 17. This shows that the limitation of the approach given in DIN 1052 to connection width $a_r < 0.5h$ is clearly necessary. The material property values according to the design approach by Ballerini are very constant with regard to the impact of connection width a_r/h and show a comparably low variation. The parameter f_w considering the impact of the width of the connection can be considered as sufficiently accurate.

The approach by Ballerini and the approach given in DIN 1052 show a slight underestimation of the load-carrying capacity of connections with only a single column of fastener ($m = 1$), which delivers the lower bound of the material property values for the design approach given in EC5.

The connection height h_m/h shows no linear impact on the material strength parameters according to the design approaches by Ballerini and the approach given in DIN 1052. The material strength parameter according to the approach given in EC5 shows a strong dependency on the connection height h_m/h. However, it can be seen from the quadratic regression function that the maximum load-carrying capacity is achieved in the range between $0.1 \leq h_m/h \leq 0.3$. The approach by Ballerini shows a slight overestimation of the load-carrying capacity for more than 3 rows rows of fastener ($n = 3$).

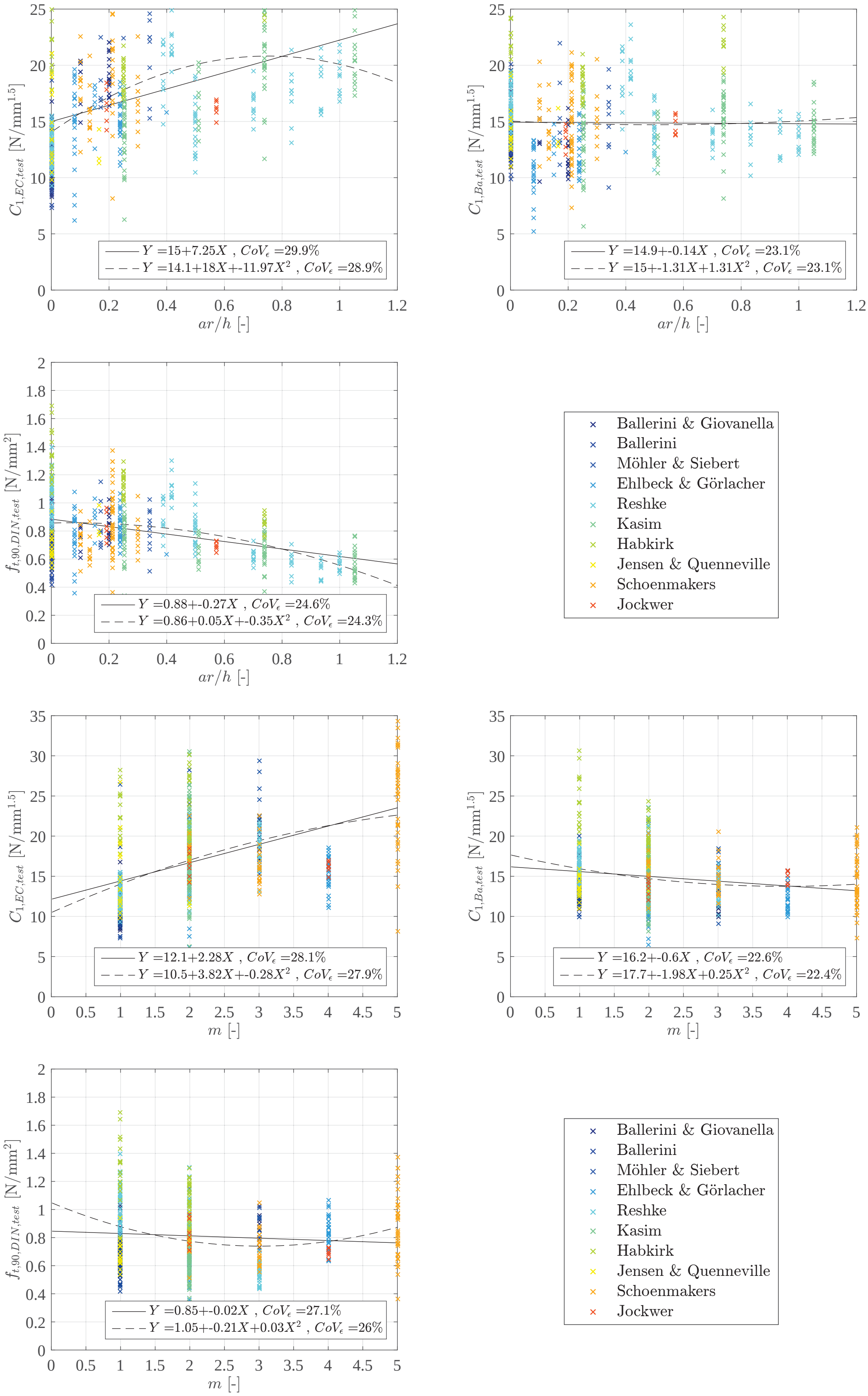


Fig. 23: Evaluation of the material strength parameters according to test results with regard to the relative connection width a_r/h and the number of fastener columns m.

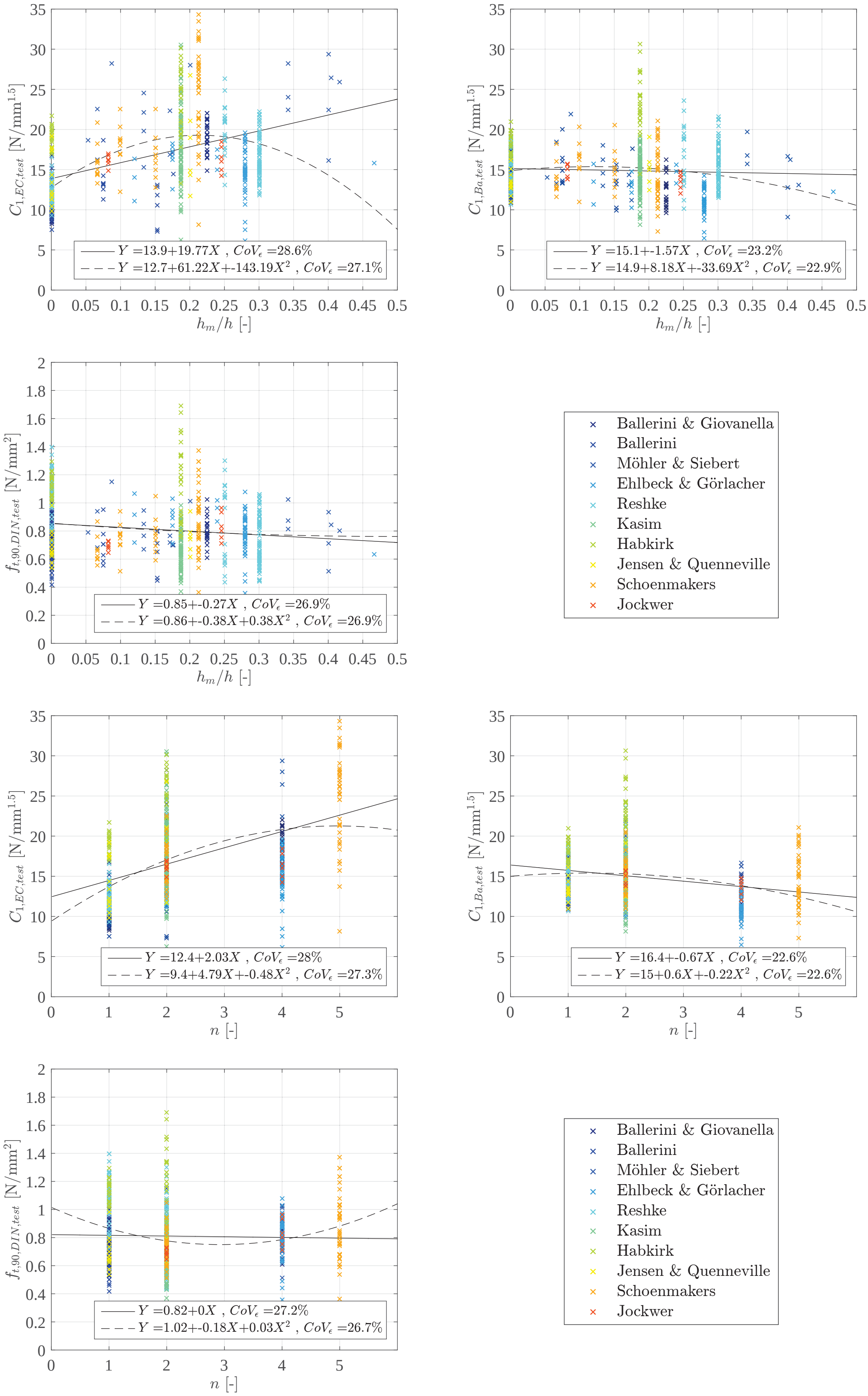


Fig. 24: Evaluation of the material strength parameters according to test results with regard to the connection height h_m/h and the number of fastener rows n.

6.4 Study on the variability of the design approaches when accounting for the geometry of the connection

In order to quantify the adequacy of the design approaches for the prediction of load-carrying capacity of connections loaded perpendicular to the grain the distribution characteristics of the material strength parameters $C_{1,EC}$, $C_{1,Ba}$ and $f_{t,90,DIN}$ were determined. In addition the potential for optimization of the design approach was identified by combining the basic design approaches with the parameters accounting for the geometry of the connection. The basic design approaches are dependent only on the dimensions of the beam (b, h) and the relative connection height α as given in Eq. 44 - 46:

$$C_{1,EC5,basic} = \frac{\max\left\{F_{v,90,i}\right\}}{b\sqrt{\frac{\alpha h}{(1-\alpha)}}} \tag{44}$$

$$C_{1,Ballerini,basic} = \frac{F_{90}}{2b\sqrt{\frac{\alpha h}{(1-\alpha^3)}}} \tag{45}$$

$$f_{t,90,DIN,basic} = \frac{F_{90}}{(6.5 + 18\alpha^2)\left(b_{ef}h\right)^{0.8}} \tag{46}$$

The parameters for accounting for the geometry of the connection from the approach by Ballerini (f_w and f_r) and the approach given in DIN 1052 (k_s and k_r) can be specified as follows:

$$f_w\left(a_r, l_l, h\right) = min\begin{cases} 1 + 0.75\left(\frac{a_r + l_l}{h}\right) \\ 2.2 \end{cases} \tag{47}$$

$$f_r\left(n, h_m\right) = 1 + 1.75\frac{\frac{nh_m}{1000}}{1 + \frac{nh_m}{1000}} \tag{48}$$

$$k_s = max\begin{cases} 1 \\ 0.7 + \frac{1.6a_r}{h} \end{cases} \tag{49}$$

$$k_r = \frac{n}{\sum_{i=1}^{n}\left(\frac{h_1}{h_i}\right)^2} \tag{50}$$

The basic design approaches and the parameters were combined as follows:

Combination:								
1	2	3	4	5	6	7	8	9
Fracture mechanics based approaches:								
$C_{1,basic}$	$\frac{C_{1,basic}}{f_w}$	$\frac{C_{1,basic}}{f_r}$	$\frac{C_{1,basic}}{f_w f_r}$	$\frac{C_{1,basic}}{k_s}$	$\frac{C_{1,basic}}{k_r}$	$\frac{C_{1,basic}}{k_s k_r}$	$\frac{C_{1,basic}}{f_w k_r}$	$\frac{C_{1,basic}}{f_r k_s}$
Approach based on strength criterion:								
$f_{t,90,basic}$	$\frac{f_{t,90,basic}}{f_w}$	$\frac{f_{t,90,basic}}{f_r}$	$\frac{f_{t,90,basic}}{f_w f_r}$	$\frac{f_{t,90,basic}}{k_s}$	$\frac{f_{t,90,basic}}{k_r}$	$\frac{f_{t,90,basic}}{k_s k_r}$	$\frac{f_{t,90,basic}}{f_w k_r}$	$\frac{f_{t,90,basic}}{f_r k_s}$

The results of these combination as mean values with the corresponding coefficient of variation are given in Tab. 4. The best combination for each approach is highlighted with light green colour. The second best fit is highlighted in light grey colour as an alternative. It can be seen that the fracture mechanics based approaches show best results in combination with the parameters by Ballerini f_w and f_r (Eqs. 47 and 48).

The design approach based on a strength criterion given in DIN 1052 achieves the best results together together with its parameters k_s and k_r (Eqs. 49 and 50) for glulam and as a combination of parameters f_r and k_s for solid timber. However, the corresponding coefficient of variation is still slightly higher compared to the one of the best fit of the approach by Ballerini.

Fig. 25 shows the cumulative distribution of the original material strength parameter according to Eqs. 41 - 43 and the parameters of the combination with the best or second best fit. From this can be seen that there is a large potential for improvement for the design approach given in EC5 by combination with the parameters developed by Ballerini. The approaches by Ballerini and the approach given in DIN 1052 allow only for minor improvement compared to the original approaches.

Tab. 4: Mean values of material parameter and coefficient of variation when considering different parameters for width and height of the connection.

Combination		1	2	3	4	5	6	7	8	9
Parameters		-	$\frac{1}{f_w}$	$\frac{1}{f_r}$	$\frac{1}{f_w f_r}$	$\frac{1}{k_s}$	$\frac{1}{k_r}$	$\frac{1}{k_s k_r}$	$\frac{1}{f_w k_r}$	$\frac{1}{f_r k_s}$
Glulam										
Eurocode 5:										
$C_{1,mean}$	$[N/mm^{1.5}]$	17.0	13.8	14.2	11.6	14.2	13.3	11.3	11.0	11.8
CoV	[%]	0.32	0.28	0.28	0.26	0.34	0.29	0.38	0.31	0.30
Ballerini:										
$C_{1,mean}$	$[N/mm^{1.5}]$	22.0	17.8	18.3	14.9	18.1	17.0	14.3	14.0	15.1
CoV	[%]	0.33	0.25	0.29	0.23	0.30	0.26	0.30	0.25	0.25
DIN 1052:										
$f_{t,90,mean}$	$[N/mm^{1.5}]$	1.15	1.01	0.96	0.86	1.07	0.91	0.85	0.81	0.89
CoV	[%]	0.36	0.34	0.34	0.36	0.32	0.29	0.26	0.31	0.29
Solid timber										
Eurocode 5:										
$C_{1,mean}$	$[N/mm^{1.5}]$	16.8	14.0	14.6	12.1	15.1	13.3	11.8	11.0	13.1
CoV	[%]	0.30	0.26	0.31	0.26	0.31	0.34	0.32	0.29	0.30
Ballerini:										
$C_{1,mean}$	$[N/mm^{1.5}]$	21.7	18.1	18.8	15.6	19.5	17.0	15.2	14.1	16.8
CoV	[%]	0.29	0.26	0.29	0.24	0.30	0.30	0.29	0.25	0.28
DIN 1052:										
$f_{t,90,mean}$	$[N/mm^{1.5}]$	1.24	1.05	1.07	0.91	1.12	0.97	0.87	0.82	0.96
CoV	[%]	0.27	0.29	0.27	0.27	0.28	0.29	0.27	0.29	0.26

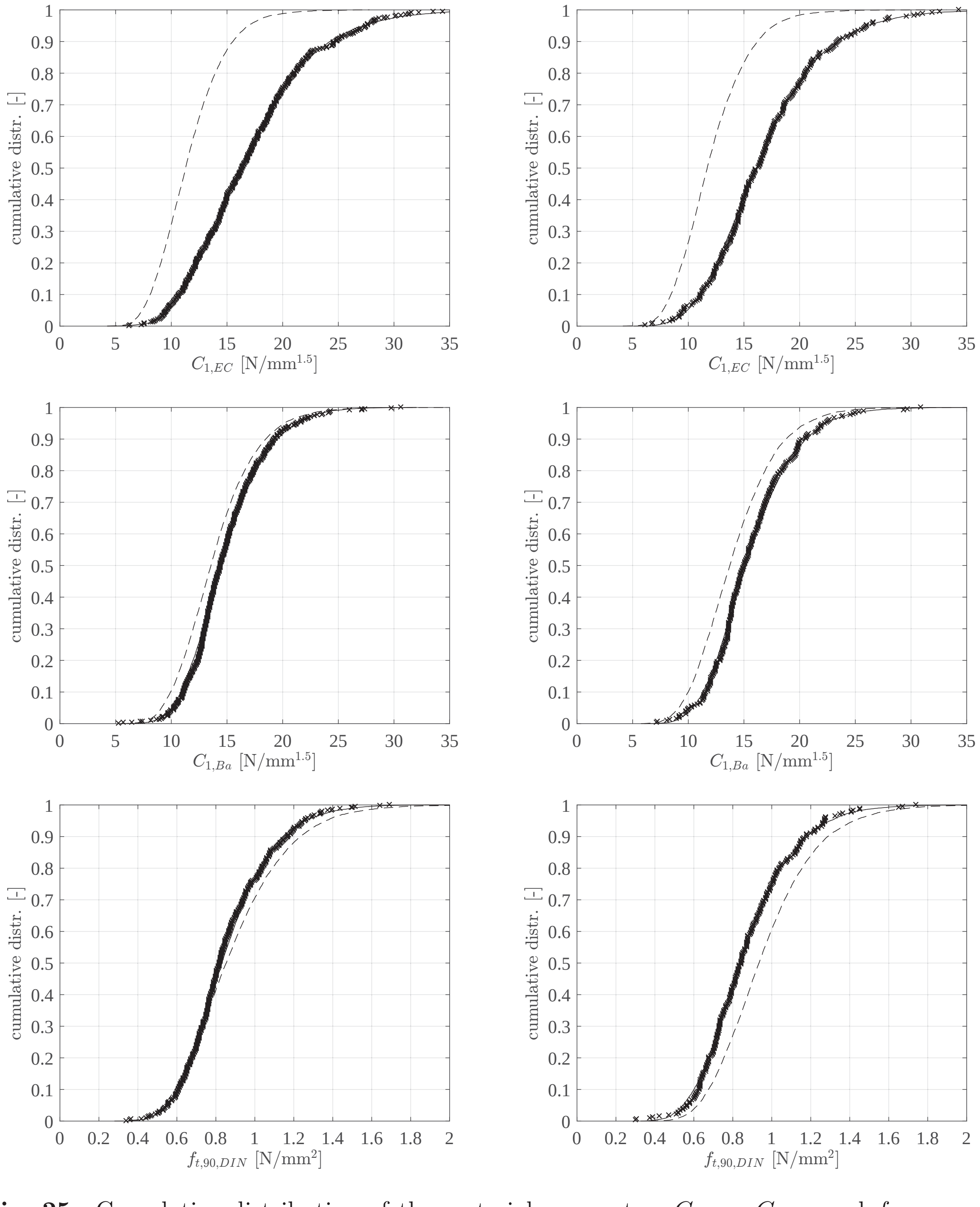


Fig. 25: Cumulative distribution of the material parameters $C_{1,EC}$, $C_{1,Ba}$ and $f_{t,90,DIN}$ and lognormal distribution function for the best or second best fit for glulam (left) and solid timber (right).

7 Conclusions

7.1 Summary

From the studies performed during the STSM at TU Munich, the following conclusions with regard to the performance of the design approaches can be summarized:

- The most relevant geometrical parameters with regard to the load-carrying capacity of connections loaded perpendicular to the grain are height h and width b of the beam, relative connection height α, connection width a_r and connection height h_m. In case of multiple connections the distance l_l between these has an important influence as well. The position of the connection along the beam span or at the end of a cantilever beam are of minor relevance with regard to the load-carrying capacity.
- The various design approaches from literature can be separated into approaches based on strength criteria (like e.g. the approach given in DIN 1052) and into approaches based on fracture mechanics theory (like e.g. the approach given in EC5 or the one by Ballerini). The basis of the approach can be of relevance when discussing the implementation into standards with specific material property values.
- None of the design approaches compared was found to be universally valid. The various design approaches account for different factors and require different additional limitations. A general good fit was found by the design approaches given in DIN 1052 and by the design approach by Ballerini. The approach given in EC5 can be considerably improved by including the parameters accounting for the geometry of the connection proposed by Ballerini.
- Reinforcement is an easy and efficient measure to restore the load-carrying capacity of beams with connections loaded perpendicular to the grain as discussed e.g. in Schoenmakers (2010), Jockwer et al. (2015). When implementing a design approach for unreinforced connections loaded perpendicular to the grain into a design code, the benefit of reinforcement of such connections should be pointed out and design equations for the design of the reinforcement should be given.

7.2 Outlook and need for further research

The following need for further research can be identified:

- Most studies have been made on bolts, dowels, nails or shear connectors. Also other types of fasteners should be studied more in detail.
- The impact of two or more adjacent connections should be studied more in detail, like e.g. screws or glued-in rods. The potential for an increase of load-carrying capacity is high for this situation, however, the risk of failure is impacted by e.g. size or volume effects for multiple connections close to each other.
- The beam width is accounted for by most design approaches in a linear way. Other approaches use a reduced, effective width of the beam in dependency of the type and slenderness of the fasteners used. The impact of the slenderness of the fastener and risk of partial splitting during initiation of failure should be studied more in detail.
- Duration of load effects and effects from moisture variations are constant matter of concern for situations with tension perpendicular to the grain.

7.3 Dissemination of the results

It is intended to use the results of this STSM and this report for a revision and an improvement of the design equations and recommendations given in EN 1995-1-1 for the design of connections loaded at an angle to the grain.

References

Aicher S., Dill-Langer G. (1997): Dol effect in tension perpendicular to grain of glulam depending on service classes and volume. In: Proc. of the CIB-W18 Meeting 30, Vancouver, Canada, Paper No. CIB-W18/30-9-1

Aicher S., Boström L., Gierl M., Kretschmann D., Valentin G. (1997): Determination of fracture energy of wood in mode II - RILEM TC 133 Report. Tech. rep., SP Swedish National Testing and Research Institute, Borås, Sweden

Aicher S., Dill-Langer G., Ranta-Maunus A. (1998): Duration of load effect in tension perpendicular to the grain of glulam in different climates. European Journal of Wood and Wood Products 56(5):295–305

Aicher S., Dill-Langer G., Klöck W. (2002): Evaluation of different size effect models for tension perpendicular to grain strength of glulam. In: Proc. of the CIB-W18 Meeting 35, Kyoto, Japan, Paper No. CIB-W18/35-6-1

Ballerini M. (1999): A new set of experimental tests on beams loaded perpendicular-to-grain by dowel-type joints. In: Proc. of the CIB-W18 Meeting 32, Graz, Austria, Paper No. CIB-W18/32-7-2

Ballerini M. (2004): A new prediction formula for the splitting strength of beams loaded by dowel-type connections. In: Proc. of the CIB-W18 Meeting 37, Edingburgh, Scotland, Paper No. CIB-W18/37-7-5

Ballerini M., Giovanella A. (2003): Beams transversally loaded by dowel-type joints: influence on splitting strength of beam thickness and dowel size. In: Proc. of the CIB-W18 Meeting 36, Colorado, USA, Paper No. CIB-W18/36-7-7

Barrett J.D., Foschi R.O., Fox S.P. (1975): Perpendicular-to-grain strength of douglas-fir. Canadian Journal of Civil Engineering 2(1):50–57

CEN (2004): 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, Bruxelles, Belgium

CEN (2009): EN 338: Structural timber - Strength classes. European Committee for Standardization CEN, Bruxelles, Belgium

CEN (2010): EN 408: Timber structures - Structural timber and glued laminated timber - Determination of some physical and mechanical properties. European Committee for Standardization CEN, Bruxelles, Belgium

CEN (2013): EN 14080: Timber structures - Glued laminated timber and glued solid timber - Requirement. European Committee for Standardization CEN, Bruxelles, Belgium

DIN (2008): DIN 1052: Entwurf, Berechnung und Bemessung von Holzbauwerken - Allgemeine Bemessungsregeln und Bemessungsregeln für den Hochbau. DIN 1052 (2008-12), DIN Deutsche Institut für Normung e.V., Berlin, Germany

Ehlbeck J., Görlacher R. (1983): Tragverhalten von Queranschlüssen mittels Stahlformteilen, insbesondere Balkenschuhen, im Holzbau. Tech. rep., Versuchsanstalt für Stahl, Holz und Steine, Abteilung Ingenieurholzbau, Universität Fridericiana Karlsruhe, Germany

Ehlbeck J., Görlacher R. (1985): Zum Querzugnachweis bei Anschlüssen mittels Stahlblechformteilen. Bauen mit Holz 87

Ehlbeck J., Görlacher R. (1991): Empfehlung zum einheitlichen genaueren querzugnachweis für anschlüsse mit mechanischen verbindungsmitteln. Bauen mit Holz 93:825–828

Ehlbeck J., Görlacher R., Werner H. (1989): Determination of perpendicular to grain stresses in joints with dowel-type fasteners - a draft proposal for design rules. In: Proc. of the CIB-W18 Meeting 22, Berlin, Germany, Paper No. CIB-W18/22-7-2

Franke B., Quenneville P. (2011): Design approach for the splitting failure of dowel-type connections loaded perpendicular to grain. In: Proc. of the CIB-W18 Meeting 44, Vancouver, Canada, Paper No. CIB-W18/44-7-5

Gehri E. (1988): Tragmodell für Queranschlüsse. private notes

Habkirk R. (2006): Bolted wood connections loaded perpendicular-to-grain - Effect of wood species. Master Thesis, Department of Civil Engineering, Royal Military College of Canada, Kingston, Ontario, Canada

Jensen J.L. (2003): Splitting strength of beams loaded by connections. In: Proc. of the CIB-W18 Meeting 36, Colorado, USA, Paper No. CIB-W18/36-7-8

Jensen J.L. (2005): Quasi-non-linear fracture mechanics analysis of splitting failure in simply supported beams loaded perpendicular to grain by dowel joints. Journal of Wood Science 51(6):577–582

Jensen J.L., Quenneville P. (2011): Splitting of beams loaded perpendicular to grain by connections–some issues with EC 5. In: Proc. of the CIB-W18 Meeting 44, Alghero, Italy, Paper No. CIB-W18/44 Note

Jensen J.L., Gustafsson P.J., Larsen H.J. (2003): A tension fracture model for joints with rods or dowels loaded perpendicular to grain. In: Proc. of the CIB-W18 Meeting 36, Colorado, USA, Paper No. CIB-W18/36-7-9

Jensen J.L., Quenneville P., Girhammar U.A., Källsner B. (2012): Beams loaded perpendicular to grain by connections–combined effect of edge and end distance. In: Proc. of the CIB-W18 Meeting 45, Växjö, Sweden, Paper No. CIB-W18/45-7-2

Jockwer R., Frangi A., , Steiger R. (2015): Evaluation of the reliability of design approaches for connections perpendicular to the grain. In: Proc. of the 2nd Meeting of INTER, Šibenik, Croatia, Paper No. INTER/48-7-4

Kasim M.H. (2002): Bolted timber connections loaded perpendicular-to-grain - Effect of row spacing on resistance. Master Thesis, Department of Civil Engineering, Royal Military College of Canada, Kingston, Ontario, Canada

Larsen H., Gustafsson P. (2001): Dowel joints loaded perpendicular to grain. In: Proc. of the CIB-W18 Meeting 34, Venice, Italy, Paper No. CIB-W18/34-7-4

Larsen H., Gustafsson P.J. (1990): The fracture energy of wood in tension perpendicular to the grain. In: Proc. of the CIB-W18 Meeting 23, Lisbon, Portugal, Paper No. CIB-W18/23-19-2

Lehoux M.C.G., Quenneville J.H.P. (2004): Bolted wood connections loaded perpendicular-to-grain, a proposed design approach. In: Proc. of the CIB-W18 Meeting 37, Edinburgh, Scottland, Paper No. CIB-W18/37-7-4

Leijten A.J.M., Jorissen A.J.M. (2001): Splitting strength of beams loaded by connections perpendicular to grain, model validation. In: Proc. of the CIB-W18 Meeting 34, Venice, Italy, Paper No. CIB-W18/34-7-1

Lignum (1990): Holzbau-Tabellen HBT 2. Lignum, Zurich, Switzerland

Mistler H.L. (1998): Design of glulam beams according to EC 5 with regard to perpendicular-to-grain tensile strength - A comparison with research results. Holz als Roh - und Werkstoff 56(1):51–60

Möhler K., Lautenschläger R. (1978): Großflächige Queranschlüsse bei Brettschichtholz. Forschungsbericht des Lehrstuhls für Ingenieurholzbau und Baukonstruktion, Universität Karlsruhe, Karlsruhe, Germany

Möhler K., Siebert W. (1980): Ausbildung von Queranschlüssen bei angehängten Lasten an Brettschichtträgern oder Vollholzbalken. Lehrstuhl für Ingenieurholzbau und Baukonstruktionen, Universität Karlsruhe, Karlsruhe, Germany

Möhler K., Siebert W. (1981): Queranschlüsse bei Brettschichtträgern oder Vollholzbalken. Bauen mit Holz 83:84–89

Möhler K., Siebert W. (1983): Erhöhung der Querzugfestigkeit in gefährdeten Bereichen, Vol. T1194. Fraunhofer IRB Verlag

van der Put T.A.C.M. (1990): Tension perpendicular to the grain at notches and joints. In: Proc. of the CIB-W18 Meeting 23, Lisbon, Portugal, Paper No. CIB-W18/23-10-1

van der Put T.A.C.M., Leijten A.J.M. (2000): Evaluation of perpendicular to grain failure of beams caused by concentrated loads of joints. In: Proc. of the CIB-W18 Meeting 33, Delft, The Netherlands, Paper No. CIB-W18/33-7-7

Quenneville J., Mohammad M. (2001): A proposed canadian design approach for bolted connections loaded perpendicular-to-grain. In: Proceedings of the International RILEM symposium on joints in timber structures, pp 61–70

Reske R. (1999): Bolted timber connections loaded perpendicularto grain: influence of joint configuration parameters on strength. Master Thesis, Department of Civil Engineering, Royal Military College of Canada, Kingston, Ontario

Schoenmakers J.C.M. (2010): Fracture and failure mechanisms in timber loaded perpendicular to the grain by mechanical connections. PhD thesis, Technische Universiteit Eindhoven

Thiede C. (2014): Vergleich vorhandener Bemessungsansätze für Queranschlüsse im Holzbau. Master Thesis, Lehrstuhl für Holzbau und Baukonstruktion, Technische Universität München, Germany

Zarnani P., Quenneville P. (2013): Wood splitting capacity in timber connections loaded transversely: Riveted joint strength for full and partial width failure modes. In: Proc. of the CIB-W18 Meeting 46, Vancouver, Kanada, Paper No. CIB-W18/46-7-5