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CONNECTIONS WITH GLUED-IN RODS IN TRUSSES MADE OF BEECH-LVL

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ABSTRACT: The suitability of axially loaded glued-in rods to connect diagonals with chords in trusses made of Beech-LVL is studied experimentally. In a first step the strength and stiffness properties and the necessary structural boundary conditions (minimum spacing and distances, anchoring length) are determined and compared with the state of the art. To verify the parameters as well as the load-bearing behaviour, results of full-scale tests with trusses (span 10 m, static height 1.25 m) with different node configurations are presented.

KEYWORDS: LVL, beech, glued-in rods, block shear, trusses

1 INTRODUCTION

Trusses are an efficient and architecturally favoured structural form to realize long-span roof structures. Due to the predominant normal forces in truss members, the use of materials with high tensile and compressive strengths favourably enables long-span and slender trusses. With a characteristic tensile strength up to 70 N/mm^2, Beech-LVL offers an interesting alternative to glulam made of softwood.

A common option to realize truss joints are steel-to-timber joints with laterally loaded dowels. The necessary holes for fasteners and slots for steel plates lead to significantly reduced net cross-sections. Particularly in components under tensile load, this reduction must be taken into account. However, using suitable fasteners the attenuation may be reduced to a minimum.

The objective of a research project at Karlsruhe Institute of Technology was to identify connections able to transmit high forces and simultaneously providing high net cross-sections. Experimental studies on glued-in rods bonded in glulam made of spruce already showed that the tensile capacity of the cross-section could be achieved (see Figure 1).

Therefore the focus is placed on the study of the load-carrying capacity of glued-in rods in Beech-LVL. This paper presents experimental results of axially loaded glued-in rods arranged parallel or under an angle to the grain. On this basis, experiments on full-scale trusses with joints comprising glued-in rods were carried out to verify the load-carrying behaviour of the connections.

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Figure 1: Glued-in rods bonded parallel to the grain in glulam made of spruce

2 GLUED-IN RODS PARALLEL TO THE GRAIN

2.1 GENERAL

Connections with axially loaded glued-in rods in timber structures in Germany are designed according the national annex of Eurocode 5 [1]. To ensure a ductile behaviour of the connection the tensile strength of the rods F_{tens} should be governing. For this purpose, the failure of the bond line has to be excluded:

$$F_{tens} \leq \pi \cdot d \cdot L_{ad} \cdot f_{k1} \quad (1)$$

As a result, an uneven load transfer in connections with a group of glued-in rods may be compensated for by plastic strain in the steel.

There are already numerous publications on the general load-bearing behaviour of glued-in rods (e.g. [2] gives a comprehensive view), so this will not be discussed in more detail.

The characteristic value of the bond line strength $f_{k1,k}$ is given in [1], or in the technical approvals of the adhesives. However, the use for this connection system was until recently limited on softwood. First investigations of Enders-Comberg [3] showed that a viable connection is possible. Based on this study, 125 further specimens were tested in the laboratory of Karlsruhe Institute of Technology to determine the bond line strength and the constructive boundary conditions. The complete investigation in this section is summarized in [4].

2.2 EXPERIMENTAL PROGRAMME

The experimental setup to determine the bond line strength of glued-in rods in LVL made of beech is shown in Figure 2.

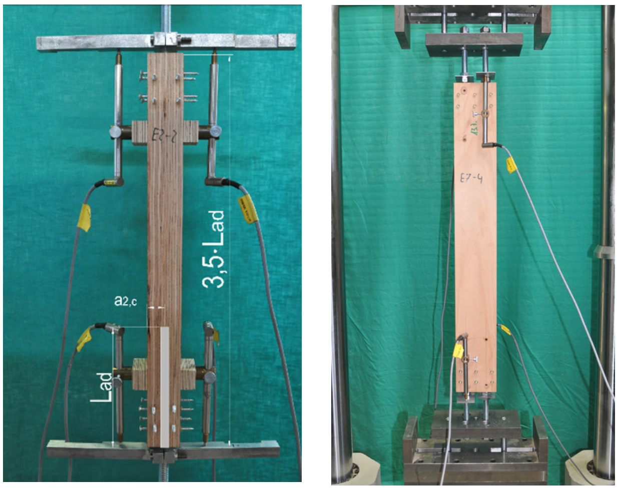

Figure 2: *Test setup for the determination of the pull-out strength for connections with one and groups of glued-in rods*

The rods (strength class 10.9 and 8.8) were bonded with the two component adhesive WEVO special resin EP 32 S with hardener B 22 TS in specimens made of Beech-LVL and Beech plywood according to [5] and [6]. The rod diameter d (12 mm, 16 mm, 20 mm), the anchoring length L_{ad} (8·d, 10·d, 12·d, 15·d) , the edge distance a_{2c} (1.5·d, 1.75·d) and the number of fasteners n_{rod} ($n_{rod,max} = 4$) were varied. All specimens were reinforced perpendicular to the grain with fully threaded screws to avoid premature splitting failure close to the end-grain (see Figure 3). One test series was carried out without reinforcement to assess the effect.

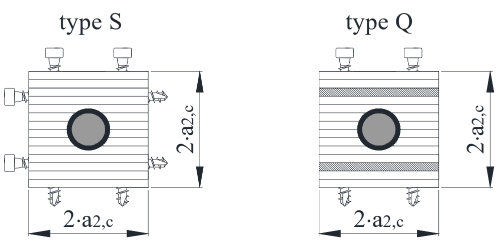

Figure 3: *Reinforcements to avoid premature splitting failure*

2.3 RESULTS

2.3.1 Connections with one rod

Most specimens failed by shear failure of the bond line or of the surrounding cross-section. In one of the test series the tensile strength of the rod was reached. In this case, the rod d = 16 mm with strength class 8.8 was glued in with an anchoring length of 15·d = 240 mm. As expected, the unreinforced specimens failed due to end-grain splitting. The observed failure modes are shown in Figure 4.

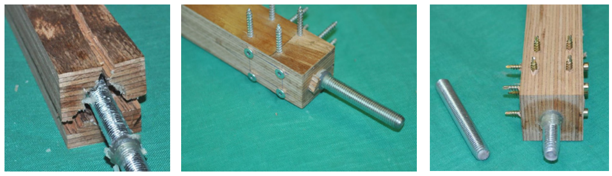

Figure 4: *Observed failure modes: splitting (left), shear failure along the rod (middle), steel failure (right)*

A decrease of the bond line strength is not detectable within the range of the investigated anchoring lengths or for the different edge distances. Differences of the load carrying capacity between Beech-LVL and Beech plywood are also not detectable. Only the rod diameter seems to have a distinct influence. The experiments show that the bond line strength decreases with an increasing rod diameter. The resulting bond line strength f_{k1} is given in Figure 5.

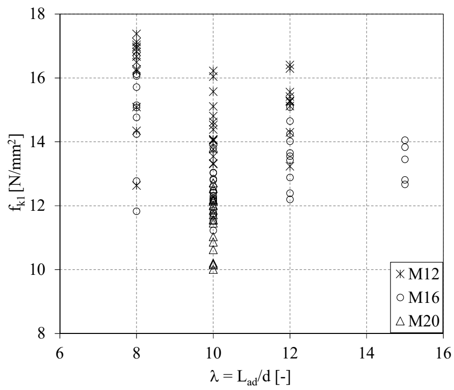

Figure 5: *Determined bond line strength f_{k1}*

Based on these test results the characteristic bond line strength $f_{k1,k}$ was calculated according to DIN EN 14358 [7]. The results are given in Table 1. Furthermore, the mean value $f_{k1,mean}$ and the coefficient of variation COV is indicated.

Table 1: *Resulting bond line strength*

diameter	$f_{k1,mean}$ [N/mm²]	COV [%]	$f_{k1,k}$ [N/mm²]
M12	15.2	8.67	12.8
M16	13.3	10.4	11.0
M20	11.5	8.62	9.74

Compared to the bond line strength according to [1] (4 N/mm²), the characteristic strength is up to 220 %

higher. This allows significantly lower anchoring lengths to ensure the ductile failure of the rods. Even the use of high-strength steel rods (8.8 and higher) is possible with small drilling depths.

Especially in view of the small necessary edge distance, a reliable and efficient connection with glued-in rods in Beech-LVL seems to be possible. Here, an edge distance of 1.75·d is favoured to allow the arrangement of the reinforcements.

2.3.2 Sustainability of groups

To study a possible group-tear out 15 specimens were tested. Similarly to the previous tests, the specimens failed by achieving the shear strength of the bond line of at least one rod.

Compared with the maximum loads of a connection with a single rod, the load carrying capacity increases proportionally with the number of rods (see Figure 6).

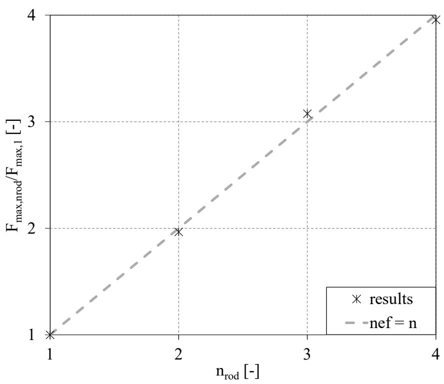

Figure 6: *Load capacity with increasing number of fasteners*

No group effect was observed based on these experimental results. It should be noted, however, that during the experiments a uniform loading of all rods in the group was provided for. For the design, an uneven load transfer should always be considered. Therefore, as already mentioned, a ductile failure of the steel rod should be ensured for robust connections. In design, the possibility of a block-shear failure should be considered as well.

3 FULL-SCALE TESTS

3.1 EXPERIMENTAL PROGRAMME

To verify the determined parameters, tests on four full-scale trusses were carried out. The span of the tested trusses was 10 m with a static depth of 1.25 m (L/H = 8). The angle between the chords and the diagonals was 45°. Beech-LVL type S was used for all truss components. The experimental setup of the four-point bending tests and the shape of the test trusses are shown in Figure 7 and Figure 8.

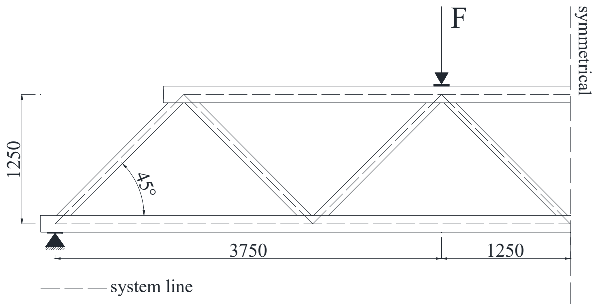

Figure 7: *Experimental setup, dimensions in mm*

Figure 8: *Shape of the test-trusses*

To measure the deflection of the trusses and the displacement between the diagonals and the chords displacement transducers were arranged in the middle of the trusses and in each node.

The tension diagonals were joined by means of four rods M12 8.8 according to [5]. The minimum distances from section 2 were used and resulted in a height h = 90 mm and width b = 120 mm of the tension diagonals. The anchoring length L_{ad} of 180 mm was dimensioned in such a way that a ductile failure of the joints should be ensured. To avoid a premature failure of diagonals, reinforcements with fully threaded screws were arranged. The compression diagonals, on the other hand, were connected by means of contact joints. In this case, the multiple step joint was used. An example of a node with glued-in rods and a multiple step joint is shown in Figure 9.

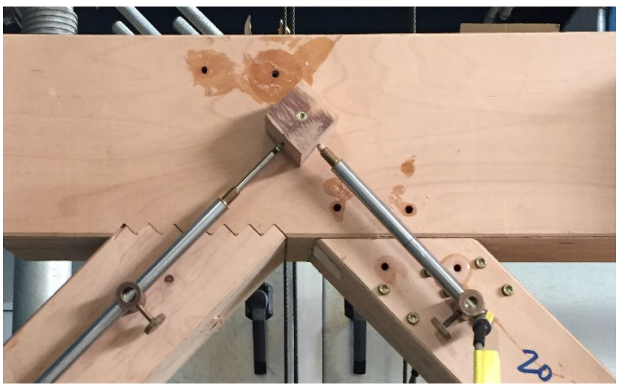

Figure 9: *Node with glued-in rods and multiple step joint*

To compare the load carrying capacity of glued-in rods with a common steel-to-timber shear connection, two reference trusses with identical cross-sections were tested ($b \times h = 160 \times 100\ mm^2$ of the tension diagonals). The shear connection in specimen Std_ref consisted of two slotted-in steel plates and dowels (S355) with a diameter of 8 mm, see Figure 10. These were arranged with minimum distances according to EC5 [8]. Reinforcements were not provided.

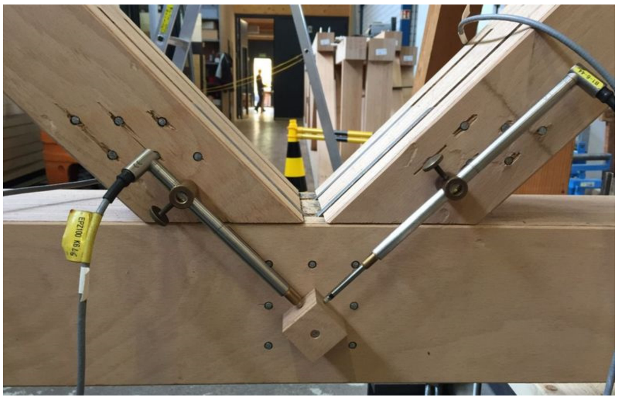

Figure 10: *Node with steel-to-timber connection of specimen Std_ref*

The nodes of the other reference specimen Gewi_ref were constructed in the same way as the other trusses with glued-in rods. The complete test program is shown in Table 2.

Table 2: *Test programme and anchoring length of the fasteners*

	Std_ref	Gewi_ref	Gewi_opt
fastener	dowels	rods	rods
diameter	8 mm	12 mm	12 mm
strength class	S355	8.8	8.8
t / L_{ad} [mm]	35 - 70 - 35	180	180
$n_{fastener}$	6	4	4
$n_{trusses}$	1	1	2

3.2 RESULTS

Due to the selected anchoring length, the tensile capacities of the steel rods in one of the two tension diagonals were reached in all trusses. The failure is shown in Figure 11.

Figure 11: *Failure of a tension diagonal with glued-in rods*

Figure 12 shows the observed failure of the reference specimen Std_ref. After reaching the maximum load the tension diagonal split in the connection area. This sudden failure occurred after reaching the yield moment of the dowels. The occurred failure mode of the dowels is also given in Figure 12.

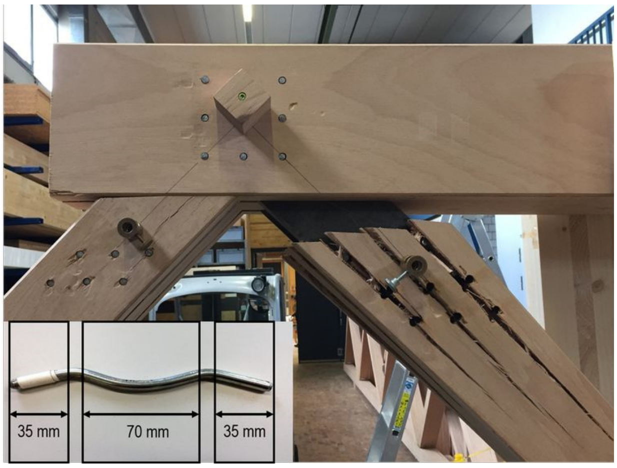

Figure 12: *Failure of reference specimen Std_ref*

In spite of the sudden failure, a more ductile behaviour of the specimen Std_ref could be observed (see load-deflection diagram in Figure 13). However, the higher stiffness of the axially loaded glued-in rods results in a lower deflection in the centre of the truss Gewi_ref.

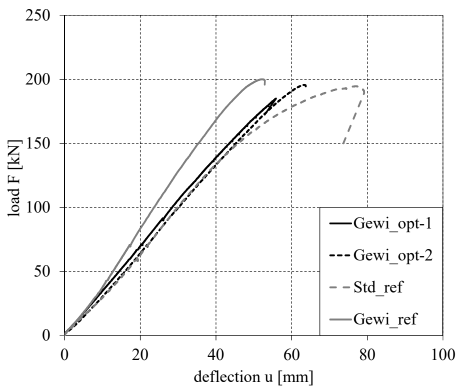

Figure 13: *Load-deflection behaviour of the reference truss specimens*

Figure 13 also shows the load-deflection diagrams of the series Gewi_opt. Due to the smaller cross-sections of the diagonals and the chords of this series, a lager deflection occurred compared to Gewi_ref. Furthermore, compared with the truss Std_ref the bending stiffness could be increased.

Table 3 shows a summary of the achieved maximum loads F_{max} and maximum deflection u (F_{max}). In addition, the estimated loads F_{est} are given. Those are based on the results of the connection tests in section 2. Furthermore, the steel properties of the fasteners were determined and used for the calculation of F_{est}.

Table 3: Experimental results of the full-scale tests

specimen	F_{max} [kN]	$u(F_{max})$ [mm]	F_{est} [kN]	F_{max}/F_{est} [-]
Std_ref	195	77.2	158	1,23
Gewi_ref	200	52.1	190	1,05
Gewi_opr-1	185	55.9	190	0,97
Gewi_opt-2	196	63.4	190	1,04

In comparison to the estimated loads F_{est}, the maximum loads F_{max} confirm the load-bearing capacity of glued-in rods. Unexpectedly the specimen Std_ref was able to reach an identical maximum load as the other tested trusses. The 23 % higher load-bearing capacity can be attributed to load-increasing friction effects between the steel plates and the wood (rope effect).

4 GLUED-IN RODS UNDER AN ANGLE TO THE GRAIN

4.1 EXPERIMENTAL PROGRAMME

At the time of the full-scale tests, the load carrying capacity of glued-in rods in Beech LVL under an angle to the grain was still unknown. To determine the influence of the angle between force and the grain direction (α) on the bond line strength, 45 specimens were tested to failure.

For increasing angles higher bond line strengths are to be expected. Therefore, rods of strength class 10.9 were chosen to ensure a shear failure of the bond line. Those were glued in according to [5] with a length of 10·d in the specimens made of Beech-LVL type S. The whole test programme is given in Table 4.

The following parameters of the connection were varied:

- Rod diameter d
- Number of fastener n_{rod} arranged parallel to the grain with a distance $a_1 = 3.5 \cdot d$
- Angle between force and grain α
- Veneer orientation (rods in LVL's wide or edge face)

Table 4: Test programme

series	d [mm]	n_{rod} [-]	α [°]	orientation [-]	n [-]
E90.1	12	1	90	wide face	5
E90.2	12	1	90	edge face	5
E90.3	12	2	90	wide face	5
E90.4	12	2	90	edge face	5
E90.5	16	1	90	edge face	5
E90.6	16	2	90	edge face	5
E45.1	12	1	45	wide face	5
E45.2	12	2	45	wide face	5

The test setup for a connection with one and two fasteners is shown in Figure 14.

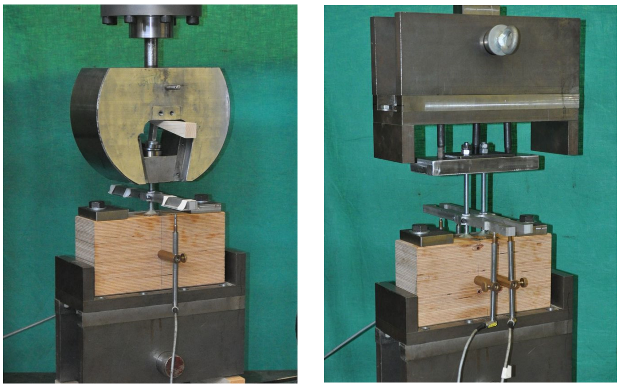

Figure 14: *Test setup to determine the influence of the angle between the force and the grain*

4.2 RESULTS

4.2.1 Connections with one rod

The shear failure of the bond line was only observed in the test series with rod diameter of 16 mm. In all other specimens the tensile strength of the steel rod was achieved. This failure mode was independent of the angle between load and grain as well as of the veneer orientation. Figure 15, for instance, shows the two observed failure modes for specimens with an angle between force and grain of 90°.

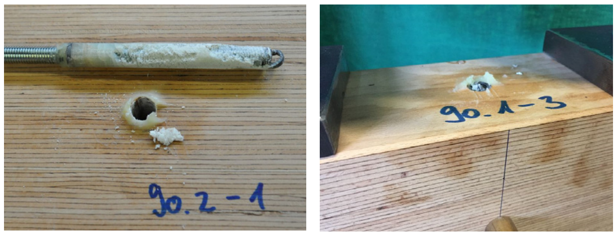

Figure 15: *Observed failure modes: pull-out (right), steel failure (left)*

The resulting bond line strengths are significantly higher than those determined in section 2.3.1. An influence of the veneer orientation was not observed. Based on the test results, the use of bond line strengths for rods glued-in parallel to the grain would be conservative for the design of the bond line.

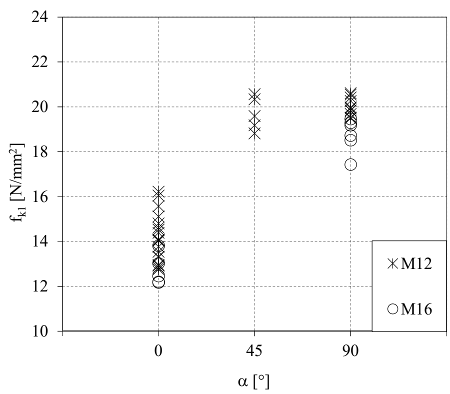

Figure 16: *Bond line strength f_{k1} for increasing angle α between force and grain direction*

It should be noted that the bond line strength values in Figure 16 for angles exceeding 0° do not represent the real bond line strength. Due to the steel failure of the rods higher strengths are to be expected in most cases. Table 5 summarizes the test results in more detail.

***Table 5:** Mean value of the maximum loads and bond line strength*

series	$F_{max,mean,1}$ [kN]	$f_{k1,mean}$ [N/mm²]	COV [%]	failure [-]
E90.1	90.9	20.1	2.39	rod
E90.2	90.2	19.9	1.68	rod
E90.5	141	18.8	3.97	bond line
E45.1	89.1	19.7	3.81	rod

4.2.2 Groups of glued-in rods

In contrast to the results of the full-scale test and the investigation in section 2.3.2, the expected load for groups of glued-in rods could not be achieved. When the maximum load was reached, the two rods were pulled-out together with the LVL block between them. This failure mode was observed for all specimens and is referred to as block shear.

The failure mode of the specimens with an angle to the grain of 90° is characterised by rolling shear failure in the shear planes between the rods (see Figure 17 *left*). However, the specimens with an angle of 45° failed due to combined longitudinal and rolling shear stresses. Furthermore, only a part of the block was pulled-out (see Figure 17 *right*).

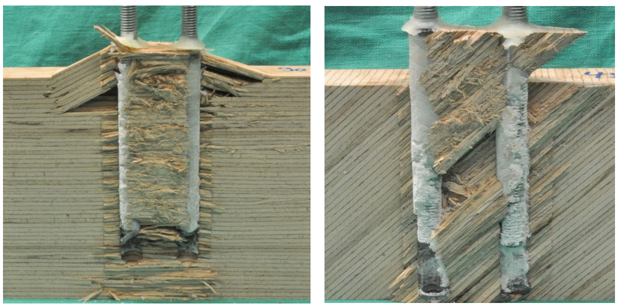

***Figure 17:** Shear failure in the shear planes between the rods*

The test results are given in Table 6. The given bond line strength results from the assumption of evenly loaded rods.

***Table 6:** Mean value of maximum loads and bond line strength of the test series*

series	$F_{max,mean,2}$ [kN]	$f_{k1,mean}$ [N/mm²]	COV [%]
E90.3	161	17.8	3.47
E90.4	153	16.9	6.15
E90.6	212	14.0	5.63
E45.2	172	19.0	3.64

Figure 18 illustrates that the load capacity per rod decreases with increasing number of rods. The ratios $F_{max,2}$ / $F_{max,1}$ are all below the grey dashed line. This line describes the ideal behaviour for $n = n_{ef}$.

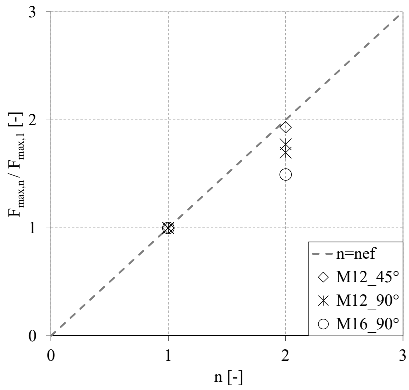

***Figure 18:** Behaviour of the load capacity with increasing number of fasteners*

One explanation for the more favorable steel failure in the full-scale tests is the larger anchoring length of the glued-in rods. Because of the larger shear planes, the resistance of the block correspondingly increases. Furthermore, the compression diagonal has a favorable effect. The additional pressure in the cross-section surrounding the anchoring length increases the shear strength. To take a closer look at the experimental results and to verify the load carrying capacity, a first calculation model is discussed below.

4.3 DISCUSSION

All tested two-rod connections with rods at 90° to the grain showed a block-shear failure in the area between the rods. The resulting block shows a depth of the respective drill hole diameter d_{dh}, a width of the distance a_1 between the rods and a height of the anchoring length L_{ad} (see Figure 17).

In this step, this failure mode is assumed to be a simple interaction of the activated fracture planes and the corresponding strengths. The calculated loads are compared with the mean values of the test results (see Table 7). For simplicity, only the test results with an angle to the grain of 90° are considered and the tensile strength perpendicular to the grain is neglected. Taking into account the two rolling shear planes and two times half the bond line of each rod results in:

$$F_{bs} = 2 \cdot (f_{v,r} \cdot L_{ad} \cdot a_1 + 0,5 \cdot f_{k1} \cdot L_{ad} \cdot d \cdot \pi) \qquad (2)$$

***Table 7:** Calculated load capacity*

series	$F_{max,mean,2}$ [kN]	$f_{v,r,mean}$ [N/mm²]	$f_{k1,mean}$ [N/mm²]	F_{bs} [kN]	Δ [%]
E90.3	161	5.54	20.1	147	8.69
E90.4	153	4.00	19.9	131	14.3
E90.6	212	4.00	18.8	210	0.94

The rolling shear strengths $f_{v,r}$ of Beech-LVL were determined in a further test programme on the basis of 21 specimens (see [9]). In the experimental programme, the veneer orientation was considered. The resulting mean values of the different veneer orientations are given in

Table 7. The bond line strength f_{k1} is assumed according to Table 5.

One reason for the deviations of up to 14.3 % in Table 7 can be associated with the chosen experimental set-up. Due to the support areas close to the connection, the resulting compressive stresses prevent rolling shear failure on the outer side of the rods. Therefore, the experimental setup prevents the activation of rolling shear areas before and after the connection. Otherwise, shear failure could also occur outside. In this case lower load carrying capacities would be conceivable.

A first study with a Finite Element model supports this assumption. Test series E90.4 was simulated. The geometry of the connection was chosen to represent the experiments. Deviating from this, the length of the specimens and thus the distance of the support from the connection area was significantly increased. In this way, an undisturbed connection was modelled. The simulation is based on an orthotropic linear-elastic material model. Because of the linear-elastic behaviour up to the maximum load of the specimens, this simplification sufficiently represents the stress distribution (see Figure 19).

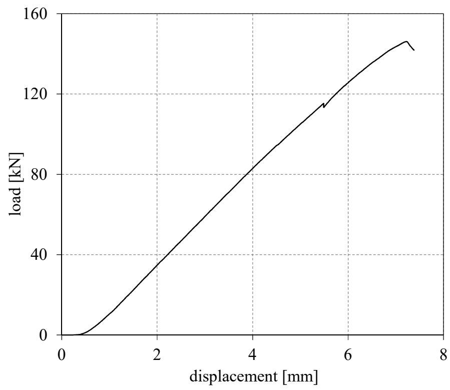

Figure 19: *Load-deflection behaviour of specimen E90.4-1*

The geometry and the evaluated stress distribution paths are shown in Figure 20. These are defined along the maximum stresses. The resulting stress distribution of the rolling shear stress τ_r at maximum load $F_{max,mean}$ = 153 kN is shown in Figure 21.

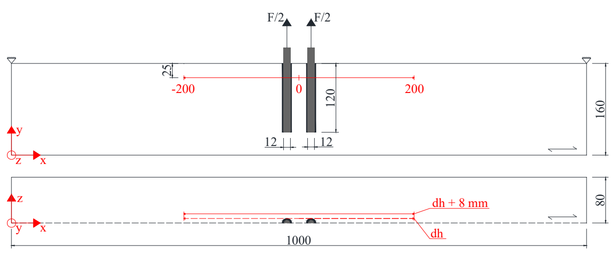

Figure 20: *Modified test series E90.4, dimensions in mm*

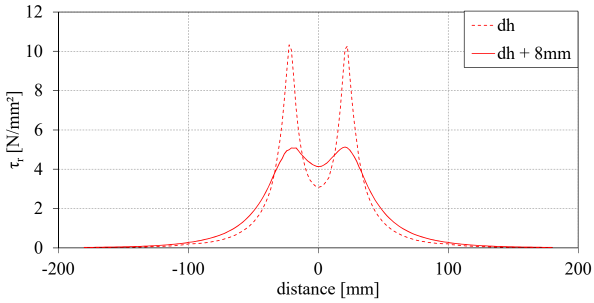

Figure 21: *Stress distribution of the rolling shear τ_R*

The diagram shows that the stresses decrease up to a distance of 200 mm from the centre of the connection. By applying the mean fracture load, the maximum rolling shear stresses exceed 4 N/mm^2. This corresponds to the mean rolling shear strength of Beech-LVL (see Table 6). Hence the observed pulling out of the block in the middle of the connection is plausible.

Apart from the area between the rods, a load transfer by rolling shear stresses takes place up to a distance of 100 mm from the centre of the connection. However, due to the rapid decline of the stresses the failure mode according to Figure 17 cannot be excluded. In order to support a general design model further investigations are necessary.

5 CONCLUSIONS

Experiments with glued-in rods show the potential to introduce large forces with high remaining cross-sections in timber structures made of Beech-LVL. The potential is significantly underestimated by the current design values. In order to enable material-appropriate and economic designs of structures, it is necessary to adapt the design parameters, especially the bond line strength f_{k1}.

Using glued-in rods to join the tension diagonals in the test trusses, the truss deflection could be significantly decreased compared to conventional steel-to-wood connection. The smaller edge and fastener distances allowed smaller cross-sections and higher slenderness with consistent load-carrying capacity.

Nevertheless, further studies need to be carried out to study the load-bearing behaviour of rods oriented under an angle to the grain. First test results show that the bond line strength increases with an increasing angle between force and grain direction. However, the expected load carrying capacity of groups could not be achieved in this case. All specimens failed due to block shear whereby the rods with the LVL block in between were pulled out. For verification of the illustrated equation further investigations are necessary.

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