SELF-TAPPING SCREWS AS REINFORCEMENT PERPENDICULAR TO THE GRAIN IN TIMBER CONNECTIONS

Hans Joachim Blaß[1)] and Martin Schmid[1)]
[1)]Lehrstuhl für Ingenieurholzbau und Baukonstruktionen,
University of Karlsruhe (TH), Germany

Abstract

The use of self-tapping screws with continuous threads as a reinforcement to avoid splitting of members in connections with dowel-type fasteners was studied in two projects carried out at the Universities of Nottingham and Karlsruhe. The reinforced specimens showed a very ductile failure mode whereas the non-reinforced specimens failed in a brittle way. Although the reinforced specimens also showed small cracks in the wood directly under the dowels, a complete splitting of the whole specimen, as regularly observed in the tests with the non-reinforced connections, did not occur. A model for calculating the load in the reinforcing screws was developed and its use is shown in some examples.

1. Introduction

The design of joints with dowel-type fasteners in a number of codes is based on Johansen's theory [1] later extended by Meyer [2]. If more than one fastener is used in a connection, the load-carrying capacity per fastener is lower than predicted by the Johansen theory. This decrease in load-carrying capacity compared with a single fastener connection is mainly caused by non uniform load distribution between the fasteners at the time of failure [4].

Splitting of timber members often takes place at loads lower than predicted by Johansen's equations. This is more severe if the number of fasteners increases and the ratio of penetration depth to fastener diameter decreases. Hence, Johansen's equations are less conservative in multiple fastener connections and if stout fasteners are used.

Fax: +49 721 698116
E-mail:Martin.Schmid@bau-verm.uni-karlsruhe.de

Based on Jorissen's research [4] the effective number of fasteners for two or more fasteners in line is calculated according to the draft German timber design code E DIN 1052:

$$n_{ef} = \min\begin{cases} n \\ n^{0,9} \cdot \sqrt[4]{\dfrac{a_1}{10 \cdot d}} \end{cases} \qquad (1)$$

Jorissen's original equation reads:

$$n_{ef} = 0,37 \cdot n^{0,9} \cdot \left(\frac{a_1}{d}\right)^{0,30} \cdot (\lambda)^{0,20} \qquad (2)$$

$$\text{with } \lambda = \min\begin{cases} t_m/d & t_m \text{ thickness of middle member} \\ 2t_s/d & t_s \text{ thickness of side members} \end{cases}$$

where a_1 is the dowel spacing parallel to the grain. As an example, four dowels in line with $a_1 = 7$ d lead to $n_{ef} = 3,2$ according to equation (1). As mentioned above this reduction is caused by non-uniform load distribution between the single fasteners if failure occurs at low displacements e.g. due to timber splitting. The advantage of reinforcing the timber in the connection area therefore is to avoid splitting and consequently to minimise or avoid the group effect.

Known methods to prevent splitting of timber members are reinforcements of the connection area with glued-on wood-based panels, pressed-on punched metal plates or glass fibre reinforcements. The new approach presented here is to use self-tapping screws as internal reinforcement similar to reinforcement bars in concrete. Compared to the reinforcement methods mentioned before self-tapping screws are easier to apply and aesthetically pleasing as they are practically invisible.

Figure 1 shows the screws used in the studies in Karlsruhe and Nottingham, respectively. The upper screw with the dimensions l x d = 182 mm x 7,5 mm and a continuous thread was in Karlsruhe, the lower with l x d = 75 mm x 4,8 mm and a threaded length of 50 mm in Nottingham. Here, d is the outer diameter of the threaded part.

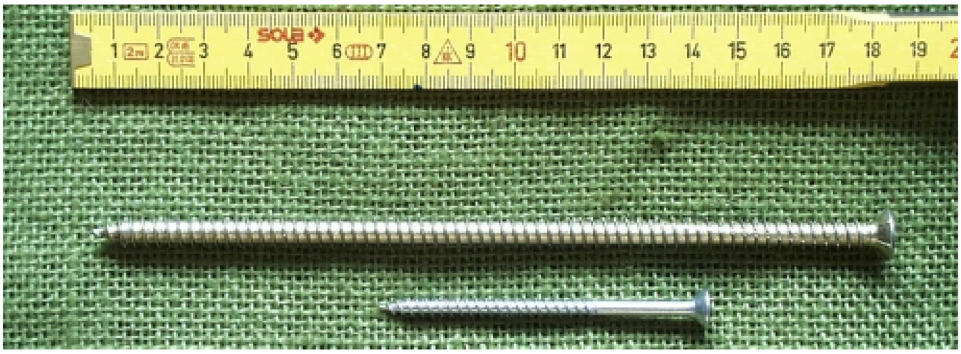


Figure 1: Screws used for reinforcement

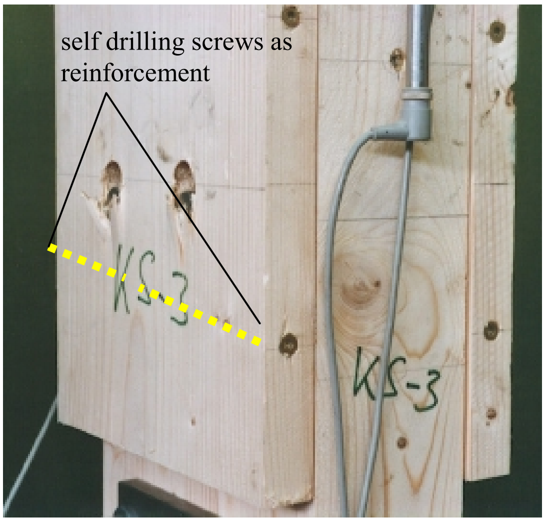
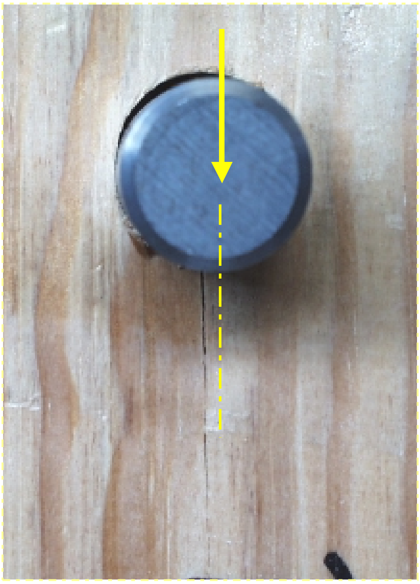


Figure 2 shows a test specimen with reinforcement by self-tapping screws, arranged perpendicular to both, the grain and the dowel's axis.

Figure 2: Detail of reinforced test specimen

2. Mechanical Model for reinforced and non-reinforced joints



Before the splitting of a timber member as a sudden, unstable crack growth in the joint area a crack starting from the contact area between the dowel and the hole surface is observed in most cases, see figure 3.

After the crack initiation the problem of crack growth in a continuum is simplified using a beam model where the stiffness perpendicular to the grain is taken into account by an elastic foundation approach, see figure 5. This simplification was first used by Jorissen [3]. While Jorissen assumed two parallel cracks per dowel, here one central crack is considered based on the observed behaviour of the wood in the vicinity of the dowel.

Figure 3: Crack at a loaded dowel

The following parameters are needed for the beam model:

Modulus of elasticity parallel to grain: E
Second moment of area: $I = t_s \cdot h^3 / 12$ (3)
Foundation modulus: $K = E_{90} \cdot t_s / (0{,}5 \cdot h)$ (4)
For the beam parts on elastic foundation, $x_0 < x < x_1$ and $x_2 < x < x_3$,
the parameter L is used with: $L^4 = 4 \cdot E \cdot I / K$ (5)

The deformation of the beam on elastic foundation is according to Szabó [5]:

$$w(x) = e^{-x/L} \cdot (C_1 \cdot \cos(x/L) + C_2 \cdot \sin(x/L)) + e^{x/L} \cdot (C_3 \cdot \cos(x/L) + C_4 \cdot \sin(x/L)) \quad (6)$$

For the elastic founded parts and the non founded beam parts, $x_1 < x < 0$ and $0 < x < x_2$, the differential equations read:

$$\varphi = \frac{d^1 w(x)}{dx} \quad (7)$$

$$\frac{M(x)}{EI} = \frac{d^2 w(x)}{dx^2} \quad (8)$$

$$V(x) = \frac{dM(x)}{dx} = \frac{d^3 w(x)}{dx^3} EI \quad (9)$$

In the equations (6) to (9) the orientation of co-ordinates and loads are as shown in figures 4 and 5.

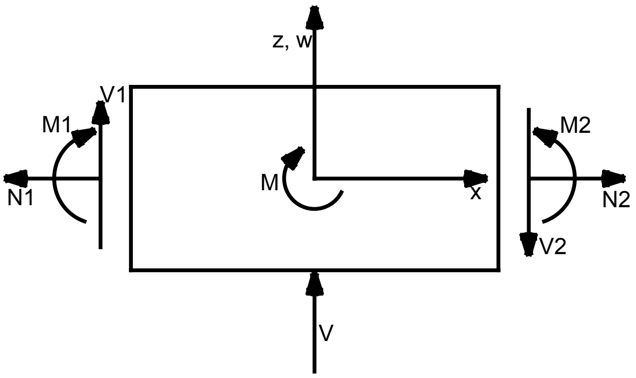


Figure 4: Co-ordinates, loads and reaction forces

The reinforcement is taken into account by modelling a pinned support at the position x_1. For the non reinforced system the shear force $V_1(x_1)$ of the beam on elastic foundation to the left of x_1 and for the non supported beam $V_{2a}(x_1)$ to the right of x_1 have to be equal:

$$V1(x1) = V2a(x1) \quad (10)$$

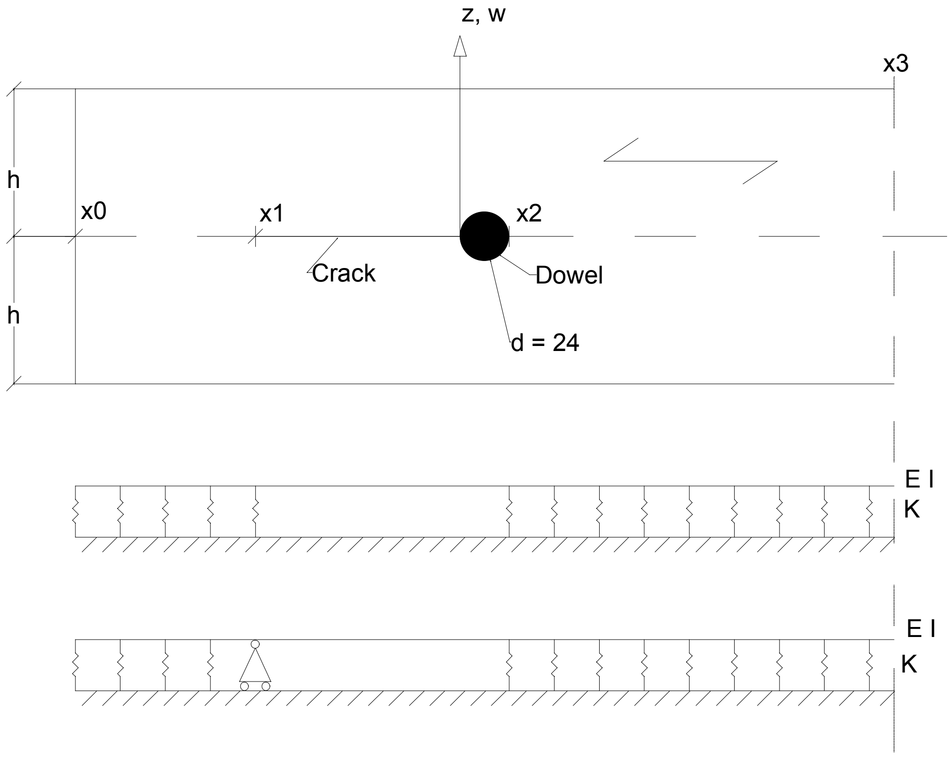


Figure 5: Mechanical model for the non-reinforced and the reinforced joint

For the reinforced specimen the deformations at x_1 are zero:

$$w_1(x_1) = w_{2a}(x_1) = 0 \tag{11}$$

The tensile force F_Z of the reinforcement is calculated as

$$F_z = V_1(x_1) - V_{2a}(x_1) \tag{12}$$

As the continuity and boundary conditions of the different beam parts lead to a complex system of equations the algebraic software mathematica 4 of Wolfram Research is used advantageously.

Figure 6 shows the deformations for a reinforced (fat line) and a non reinforced connection (thin line). For this calculation the specimen geometry of the tests performed in Karlsruhe and the following parameters were used:

$E = 12000$ N/mm², $E_{90} = 300$ N/mm², $t_s = 30$ mm, $h = 86$ mm,
$x_0 = -168$ mm, $x_1 = -72$ mm, $x_2 = 24$, $x_3 = 336$ mm.

The loads were calculated as:

$M = N / 2 \cdot h / 2$ (13)

$V = N / 10$ according to Jorissen [4] (14)

with N = 18750 N as load per dowel per shear plane.

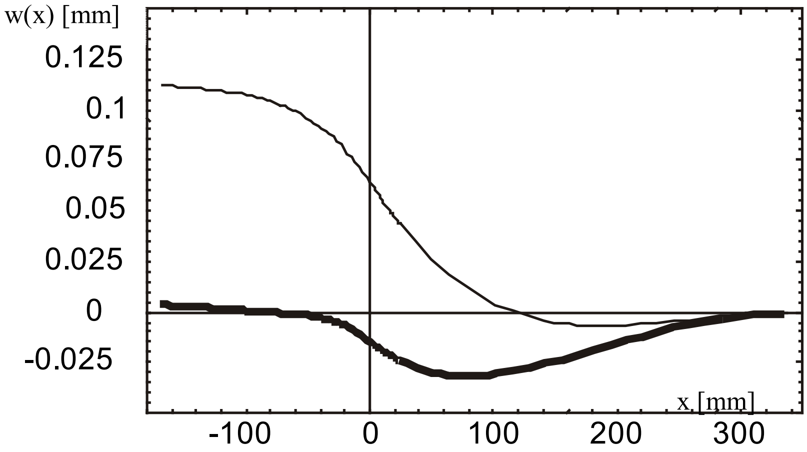


Figure 6: Deformation figure of reinforced (fat line) and non reinforced (thin line) beam model

The model can be extended to a multiple fastener connection by adding further beam segments and reinforcing screws.

Figure 7 shows the tensile force F_z of the screw versus x_1, the position of the reinforcement. The force V caused by the wedge-like action of the embedded dowel mainly determines F_z. Jorissen [4] and Werner [6] show different equations for calculating the load V perpendicular to the grain. V = N / 5 is the highest value given by these two authors.

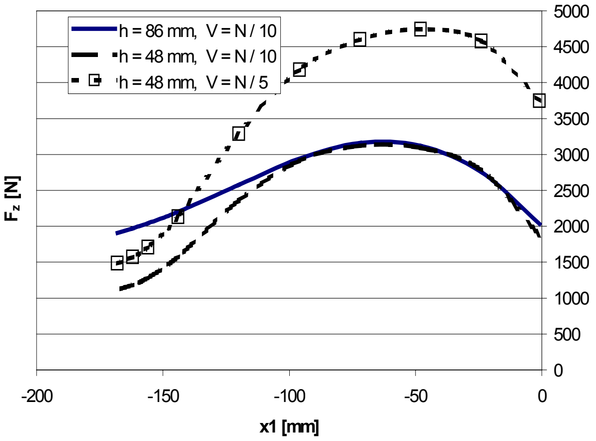


Figure 7: Tension force in the screw

3. Test results

The tests presented here are part of two student's theses at the University of Karlsruhe [7] and at the University of Nottingham [8]. In Karlsruhe, three tests each were carried out with reinforced and non reinforced specimens. The side members were made of spruce boards, the middle members of glued laminated timber. Figure 8 shows the test set-up. The dowels were made of high-grade steel with a tensile strength of $f_{u,k}$ = 1000 N/mm² in order to prevent yielding of the dowels.

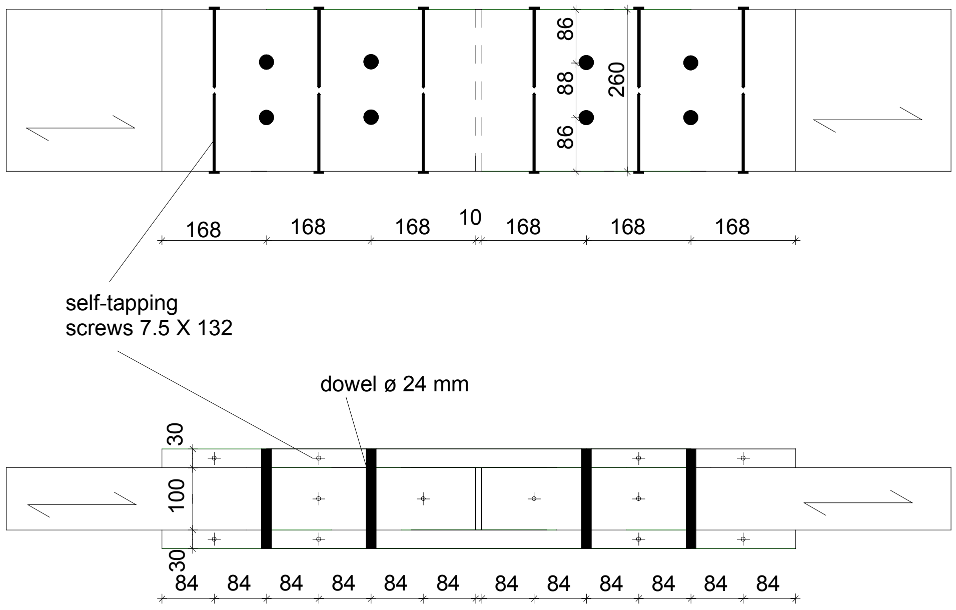


Figure 8: Test set-up used in Karlsruhe

Table 1 presents the average densities of the side members, the calculated embedding strengths, the estimated and the observed maximum load F_{max}. As no yielding of the dowels took place the estimated maximum load is calculated as:

$$F_{est} = 4 \cdot 2 \cdot f_h \cdot t_s \cdot d \qquad (15)$$

where t_s is the side member thickness,

$$f_h = 0{,}082 \cdot (1 - 0{,}01 \cdot d) \cdot \rho \qquad (16)$$

and ρ is the mean density of the side members.

Equation (2) yields n_{ef} = 1,49 and equation (1) n_{ef} = 1,71.

Table 1: Test results of Karlsruhe

Test	ρ [kg/m³]	f_h [N/mm²]	F_{est} [kN]	F_{max} [kN]	$n = 2 \cdot F_{max} / F_{est}$
Non reinforced S1	384	23,9	138	116	1,68
Non reinforced S2	421	26,3	152	114	1,51
Non reinforced S3	373	23,3	134	109	1,63
Reinforced KS1	398	24,8	143	131	1,83
Reinforced KS2	398	24,8	143	134	1,87
Reinforced KS3	412	25,7	148	121	1,64

Since the test specimens contained two connections each, the weaker of the two joints determines the maximum load.

Figure 9 shows the large deformation of a reinforced specimen. The non reinforced specimens on the other hand split at very low deformations as shown in figure 10.

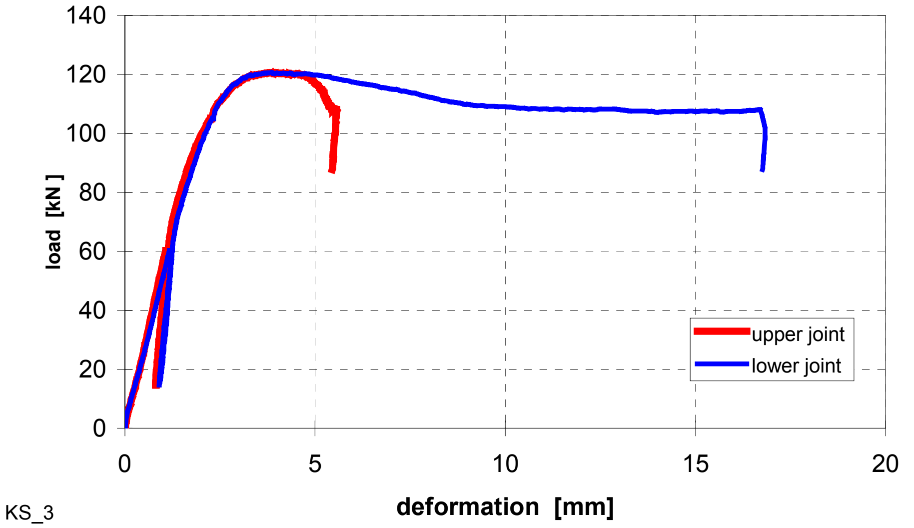


Figure 9: Load-deformation diagram of reinforced test specimen KS3

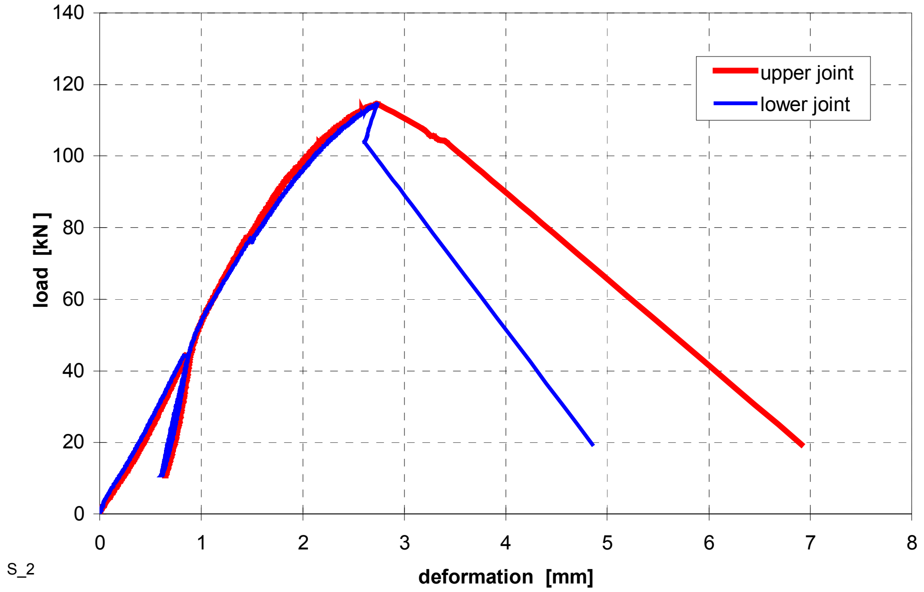


Figure 10: Load-deformation diagram of non reinforced test specimen S2

The softwood species used in Nottingham was pine. Table 2 shows the results, F_{est} was again calculated using equation (15).

Table 2: Test results of Nottingham

Timber	n	ρ_{mean} kg[m³]	F_{mean} [kN]	F_{min} [kN]	F_{max} [kN]	F_{est} [kN]	$n = 2 \cdot F_{max} / F_{est}$
unsorted soft-wood							
non reinforced	5	556	151	133	164	175	1,73
Reinforced	11	519	151	135	173	163	1,85
Swedish 5th							
non reinforced	6	517	147	143	155	163	1,80
Reinforced	13	528	157	146	165	166	1,89

The non reinforced specimen showed rather high deformations before failure compared to the tests performed in Karlsruhe. This may be due to the use of pine which tends less to splitting compared with spruce [9]. The ultimate load does not differ significantly between the reinforced and the non reinforced specimens tested in Nottingham. Some of the reinforced specimens even showed cracks up to the member ends.

The reason for this failure of the reinforcement are the screws used in Nottingham which were not threaded over the whole length of the shaft. For these screws, the head pull-through resistance determines the axial capacity which is significantly lower than the withdrawal resistance of threaded screws.

4. Conclusions

The model presented in chapter two allows to predict the tensile force in the reinforcing screw. In the tests carried out in Karlsruhe screws with continuous threads were used which had a withdrawal capacity several times larger than the calculated load. Consequently, the cracks stopped at the position of the reinforcements. The average value of the maximum load in the reinforced specimens was only about 10 % higher than in the non reinforced specimens.

Further research is necessary with more than two dowels in a row to clarify the effect of reinforcing screws on n_{ef} and of reduced dowel end distances and spacing.

5. References

1. Johansen, K.W, ‘Theory of timber connections’, in ‘International Association of Bridge and Structural Engineering’, Publication No. **9** (Bern, 1949), 249-262.
2. Meyer, A., ‘Die Tragfähigkeit von Nagelverbindungen bei statischer Belastung‘, Holz als Roh- und Werkstoff **15**, (1957), 96-109.
3. Jorrisen, A.J.M., ‘Multiple fastener timber connections with dowel type fasteners; Theoretical models’, TU Delft, Faculteit der Civiele Techniek (Delft, 1997).
4. Jorrisen, A.J.M., ’Double Shear Timber Connections with Dowel Type Fasteners’, (Delft University Press, Delft, 1998).
5. Szabó, I., ‘Einführung in die technische Mechanik‘, 8. Auflage (Springer-Verlag, Berlin, 1984).
6. Werner, H., ‘Tragfähigkeit von Holz-Verbindungen mit stiftförmigen Verbindungsmitteln unter Berücksichtigung streuender Einflussgrößen‘, in ‘4. Folge – Heft 28, Berichte der Versuchsanstalt für Stahl, Holz und Steine der Universität Fridericiana in Karlsruhe‘ (1993).
7. König, A., ‘Spaltbewehrung bei Stabdübelverbindungen‘, Vertieferarbeit an der Versuchsanstalt für Stahl, Holz und Steine, Abteilung Ingenieurholzbau, der Universität Karlsruhe (TH) (2000), not published.
8. Kreutel, F., ‘Split-reinforcement of dowel-type fastener connections’, thesis of the School of Civil Engineering at the University of Nottingham (2000), not published.
9. Schmid, M., ‘Risswiderstandskraft von im Bauwesen verwendeten Nadelhölzern‘, in ‘GESA-Symposium 2001, Sicherheit und Zuverlässigkeit durch experimentelle Struktur- und Beanspruchungsanalyse‘, VDI-Bericht 1599, (VDI Verlag Düsseldorf, 2001).

Fax: +49 721 698116
E-mail:Martin.Schmid@bau-verm.uni-karlsruhe.de