SHEAR REINFORCEMENT OF GLULAM BEAMS WITH GLUED-IN STEEL RODS - A CASE STUDY

SCHUBVERSTÄRKUNG VON BRETTSCHICHTHOLZTRÄGERN MITTELS EINGEKLEBTER GEWINDESTANGEN - EIN FALLBEISPIEL

RENFORCEMENT DE CISAILLEMENT DE POUTRES EN BOIS LAMELLE-COLLE AVEC DES TIGES FILETEES - UNE ETUDE DE CAS

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SUMMARY

Besides the high assets of the building construction material wood as related to tension, compression and bending parallel to grain the natural unidirectional fiber composite shows also two pronounced weak material properties of high brittleness. These are, in the sense of classical strength properties, the tension strength normal to grain and shear strength parallel to grain. Most failures of glulam components and structures can be related to both mentioned material properties. The Timber Department of MPA University Stuttgart has been involved in many repair and retrofitting projects of glulam structures in the year 2006.

The paper reports about some aspects of a specific project where the shear reinforcement of glulam beams was performed with glued–in steel rods. By order of the building owner full scale tests and accompanying calculations were performed. One of the test configurations is discussed here altogether with results of finite element simulations. The employed shear reinforcement by glued-in steel rods proved to be a very suitable and reliable retrofitting method.

ZUSAMMENFASSUNG

Neben den hohen Vorzügen des Bauwerkstoffes Holz speziell in Verbindung mit Zug-, Druck- und Biegebeanspruchung parallel zur Faserrichtung weist der natürliche unidirektionale Faserverbundwerkstoff auch zwei ausgeprägt schwache Materialeigenschaften mit großer Sprödigkeit auf. Dies sind, im Sinne klasischer Materialfestigkeiten, die Zugfestigkeit rechtwinklig zur Faserrichtung und die Schubfestigkeit parallel zur Faserrichtung. Die meisten Schäden von Brettschichtholzbauteilen und –tragwerken können auf die beiden genannten

Materialeigenschaften zurückgeführt werden. Die Abteilung Holzbau der MPA Universität Stuttgart war im Jahr 2006 in viele Sanierungs– und Ertüchtigungsprojekte von Brettschichtholzkonstruktionen eingebunden.

Der vorliegende Aufsatz berichtet über einige Teilaspekte eines speziellen Vorhabens, bei dem die Schubverstärkung der Brettschichtholzträger mittels eingeklebter Gewindestangen durchgeführt wurde. Auf Wunsch des Bauherren wurden vollmaßstäbliche Versuche und begleitende Finite Element (FE)-Berechnungen durchgeführt. Eine spezielle Konfiguration wird in Verbindung mit der FE-Simulation besprochen. Die verwendete Schubverstärkungsmaßnahme mit eingeklebten Gewindestangen erwies sich als eine sehr geeignete und zuverlässige Ertüchtigungsmethode.

RESUME

A côté des grands avantages du bois face à la tension, compression et flexion parallèles au fil, ce matériau composite fibreux unidirectionnel naturel présente deux propriétés particulièrement défavorables, avec une fragilité prononcée. Il s'agit, en termes de résistances classiques des matériaux, de la résistance à la traction perpendiculaire au fil et de la résistance au cisaillement parallèle au fil. La majorité des défaillances d'éléments de construction et de structures en bois lamellé-collé peuvent être attribuées_à ces deux propriétés du matériau. La section Bois de la MPA de l'université de Stuttgart a participé en 2006 à de nombreux projets de rénovation et de renforcement de constructions en bois lamellé-collé.

Le présent article rend compte de certains aspects d’un projet particulier où un renforcement de cisaillement de poutres en bois lamellé-collé a été réalisé avec des tiges filetées insérées et collées. A la demande du maître d'ouvrage, des essais grandeur nature et des simulations numériques ont été réalisés. Une configuration particulière et les résultats des simulations par éléments finis sont discutés. La mesure de renforcement de cisaillement utilisée avec des tiges filetées insérées et collées s'est avérée très appropriée et fiable.

KEYWORDS: glulam, shear reinforcement, retrofitting methods, glued-in steel rods, FE analysis

1 INTRODUCTION

Besides the high assets of the building construction material wood as related to tension, compression and bending parallel to grain the natural unidirectional fiber composite shows also two pronounced weak material properties of high brittleness. These are, in the sense of classical strength properties, the tension strength normal to grain and shear strength parallel to grain. Especially loading situations where both properties contribute in interaction to the structure resistance are problematic. The problem is further aggravated as the long term behaviour is influenced considerably by small cracks induced by climate changes. Most failures of solid wood and glulam components and structures can be related to both mentioned material properties. During the year 2006, the Timber Department of MPA University Stuttgart has been involved in well over 50 investigations and expertises of large glulam constructions, partly in conjunction with actual massive damages, partly in preventive manner as a consequence of the catastrophic failure of the glued wooden roof structure of the ice rink in Bad Reichenhall at the beginning of the year 2006. Depending on the specific damage situations different upgrading and retrofitting methods were proposed, planed and surveyed during execution. In some cases building authorities or owners of the buildings asked for experimental evidence of the proposed reinforcement provisions.

In this paper a specific upgrading case is presented, where glued-in steel rods were employed as shear reinforcement of partly/fully cracked glulam beams in the areas of high shear forces. As a few beams of the construction to be upgraded had failed in shear although some cracks had been (insufficiently) repaired at an earlier stage by epoxy resin injection the owner asked for fail safe provisions in case that the predominantely performed repair by means of filling the detected cracks with epoxy resin might reveal unexpected deficiencies.

2 GLULAM SPECIMEN AND TEST SETUP

The investigated glulam specimen stemmed from the building subject to survey and retrofitting due to the fact that some beams had failed and many beams showed deep and long cracks especially in the areas of high shear close to the support locations. The crack formation in the support areas was to a very high extent a result of very unfavorable support conditions. The beams with a span of 7.55 m and cross-sectional dimensions (width b, depth h) of b/h = 18 cm/60 cm were supported, i.e. actually clamped, at both ends by four split rings with equal spacing along cross-sectional depth on both side faces between very stiff glulam twin columns. So, in contrary to the original static design, where simply supported one-span beams were assumed, actually a frame with rather stiff corners with a high degree of moment resistance had been created. All vertical and almost all horizontal displacements were restricted by the realized support conditions. Due to hindrance of the vertical displacements, crack formation due to restricted shrinkage of the beams occurred at most support areas. The very unfavorable constructive detailing was further partially associated with unsatisfactory face gluing of the glulam laminations.

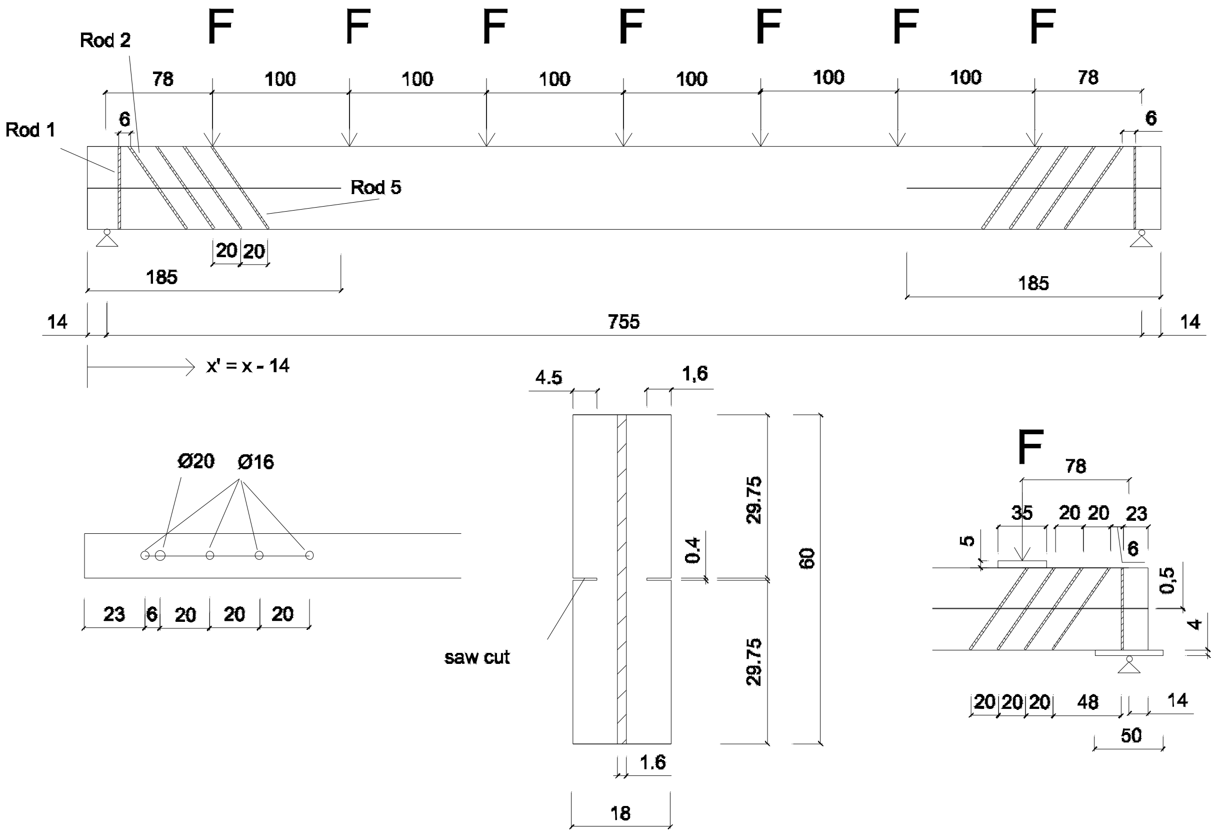


Figure 1: Specimen geometry and test set-up

Figure 1 shows the geometry and dimensions of the beam along with the test configuration. In order to simulate the situation that the repair of the cracks by means of epoxy injection was performed unsatisfactory, artificial cracks were introduced in the beam. At both ends and at both side faces 45 mm deep and 4 mm wide saw cuts were applied over a length of 1.85 m (see Fig. 1). The area of the highest shear stresses was then reinforced with glued-in threaded steel rods as shown in Fig. 1. One vertical rod with 16 mm diameter was glued-in very close to the end faces and four rods with a diameter of 16 mm and 20 mm were glued-in inclined by an angle of 55° to the beam axis (see Fig. 1). The strength class of the steel rods was 8.8 according to DIN EN ISO 898-1.

The placement of the vertical rod was chosen with respect to the described support conditions in the building which could not be changed. However, within the frame of the project it was not possible/intended to realize all on-site conditions in the accompanying experiment. The dismounted and reinforced beam was tested as a single span member with free rotating supports at the bending tension edge. Therefore, the mechanical behaviour of the rods differs partly significantly from the real building situation. Further, the arrangement of the rods was certainly not determined in the best perceivable manner for the experiment, especially when regarded detached from the real building situation.

The rods with 16 and 20 mm diameter were glued in holes with diameters d_h of 18 and 22 mm. As adhesive a special two-component epoxy resin, WEVO EP 20/VP1 with hardener B20/1, was used. The adhesive had been tested positively by MPA University Stuttgart several years ago for the purpose of gluing cracks in timber elements. In many tests and several field applications the adhesive has shown a high suitability for gluing threaded steel rods in timber, too, provided service temperatures do not exceed 50°C.

Figure 2 shows a close-up of the bending compression edge with the glued-in rods No. 1 (vertical) and No. 2 and 3 (inclined). The loading of the beam by seven equally spaced forces conformed to the loading situation in the retrofitted buildings. Figure 3 gives a view of the realized test set-up. Mid-span deflection was measured at both side faces at the bending compression edge. The test was performed at quasi force control with a constant loading rate in a not climate controlled environment with a temperature of about 20°C.

Figure 2: View of glued-in rods at the support

Figure 3: Realized test-setup

3 TEST RESULTS

Maximum load was achieved in 480 seconds. Ultimate failure occurred as a bending failure at about mid-span at the bending tension edge initiating from a knot. Figures 4a and 4b show photographs of the fractured area of the failed beam.

Within the areas of higher shear forces, between the supports to the two outermost single loads, no visibly detectable sign of cracking or other damage could be found in intensive examinations of the side and end grain faces after the test. Figure 5 shows the load (per piston) vs. mid-span deflection diagram. It can be seen that the graph is fully linear up to a small pre-peak load drop of 4% at 83% of ultimate load. The observations during the on-going test and after ultimate failure gave no indication of the location and type of the pre-peak damage. The fact that the load drop was associated to damage is proven by the load deflection curve which reveals a slightly less steep slope, i.e. reduced stiffness after the load drop.

Ultimate failure occurred at piston loads of $F_u = 41.5$ kN, giving an ultimate shear force and bending moment of

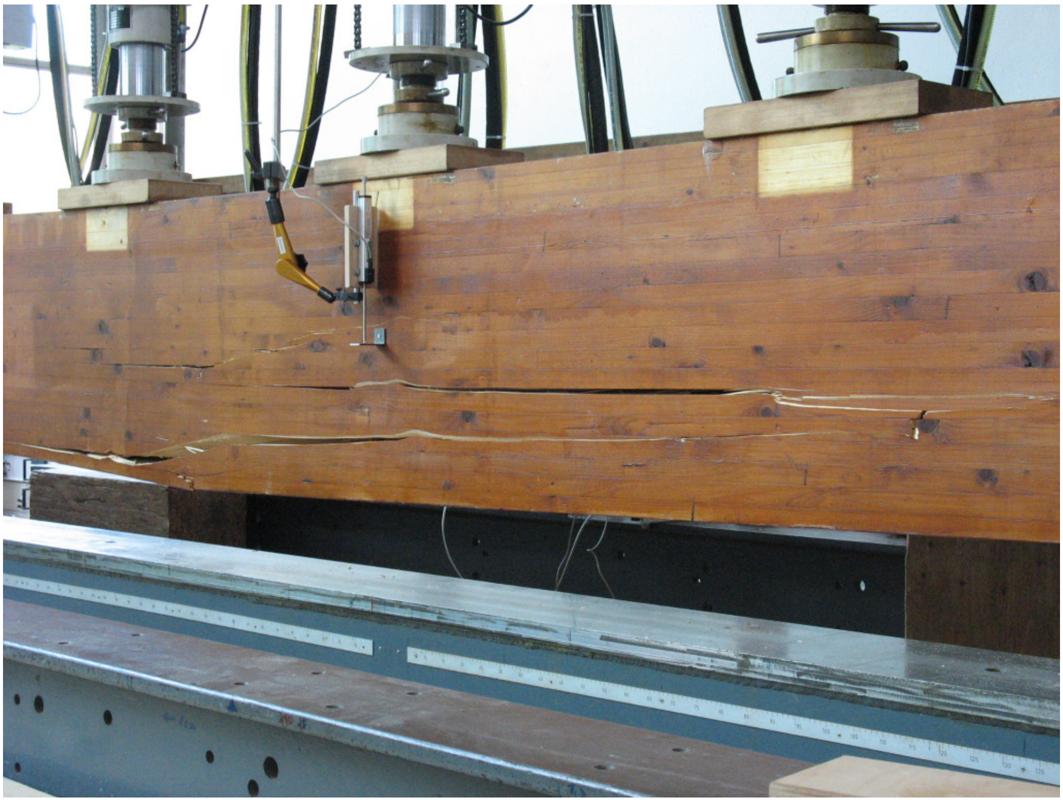
$V_u = 145.25$ kN and $M_u = 300.05$ kNm.

By assumption of the full cross-section (area $A = 1080$ cm^2, section moment $W = 10800$ m^3) the values of maximum shear stress at failure τ_u and of bending strength f_m evolve as

$\tau_u = 2.02$ N/mm^2 and $f_m = 27.8$ N/mm^2.

Employing the effective width $b_{eff} = 18$ cm – 2 x 4.5 cm = 9 cm in the support area, ultimate shear stress is

$\tau_u = 4.04$ N/mm^2.



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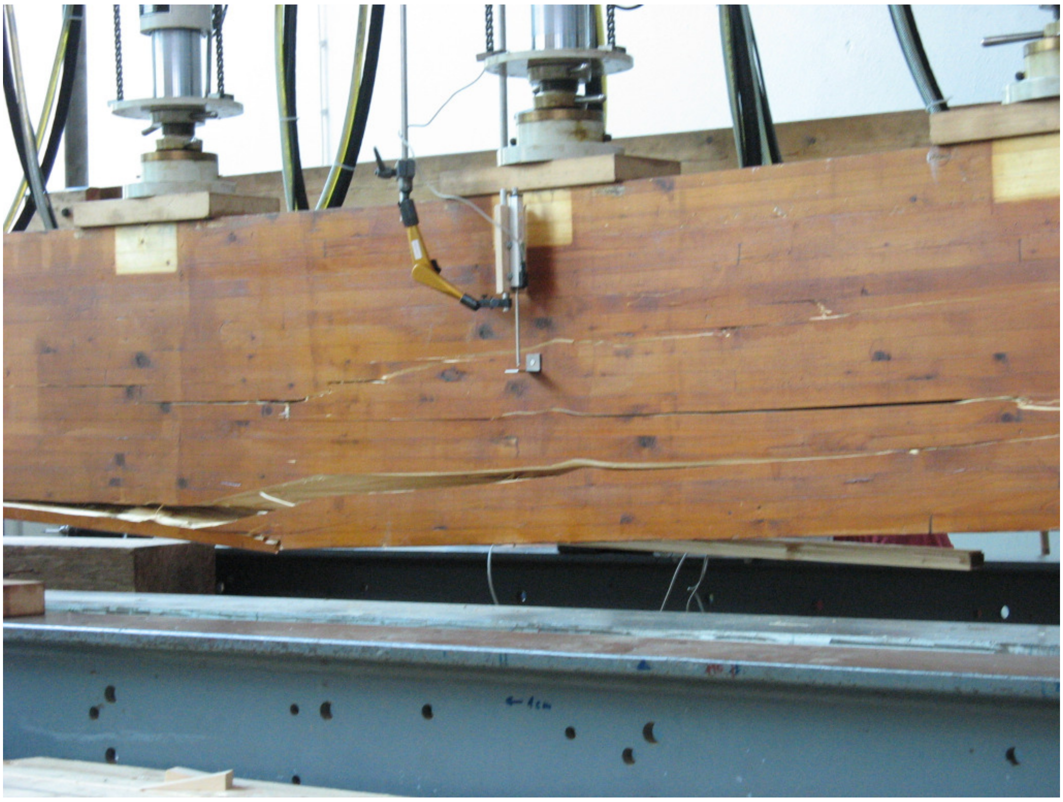

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Figure 4 a, b: Views of beam failure

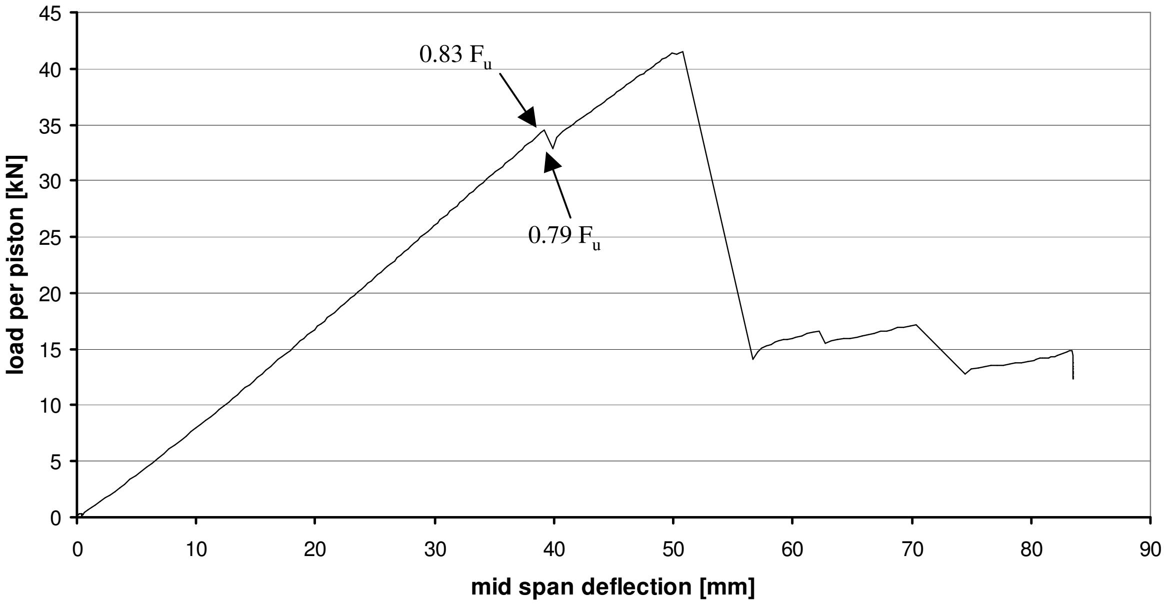


Figure 5: Load + mid span diagram

The measured apparent bending stiffness obtained from the elastic part of the load mid-span deflection diagram is (EJ)app = 4.123 x 1013 Nmm2. Disregarding any shear deformation contribution and assuming the full cross-section, a modulus of elasticity of E_m = 11650 N/mm^2 is obtained. Considering the rather small span to depth ratio of ℓ/d = 12.6 a shear contribution to the measured deflection is obvious. Assuming for the ratio of bending MOE ($E_m = E_1$) vs. shear modulus G a value of 16, as implemented for instance implicitly in the German and European timber design codes, the analytical computation without recognition of the steel reinforcement delivers an MOE of E_m = 12720 N/mm^2. On the basis of a finite element model considering geometry, sizes, MOE of the steel bars and the timber stiffness ratios $E_1/G = 16$, $E_1/E_2 = 30$ (2 = normal to the grain), the measured deflection delivers an MOE value of $E_m = E_1$= 13500 N/mm^2. It is obvious from the performed steel rod reinforcement that the ultimate shear stress values given for gross and effective cross-sectional width represent an upper limit, as a certain/considerable part of the shear force V is transferred through the steel rod action by activating some internal frame mechanism. The latter one, however, is due to strongly differing stiffnesses of timber and steel not so evident as might be supposed. A straight forward approach to confirm analytical model assumptions for the partial shear force contribution of the glued-in rods, reported in a separate paper, consists of simple linear elastic finite elements computations of the structure.

4 FINITE ELEMENT CALCULATION

In the 2D linear elastic finite element computations the glulam beam was modelled as an orthotropic material with ratios of $E_1/G = 16$ (1 = parallel to grain and beam axis), $E_1/E_2 = 30$. The Poisson ratios were taken as $\nu_{12} = \nu_{21}$ $E_2/E_1 = 0.015$ (1st Index: strain direction). In the subsequently presented results $E_1 = 11000$ N/mm^2 was used. Figure 6a shows the finite element mesh plot along with loading and boundary conditions; Fig 6b shows a close-up of the element discretization in the reinforced area.

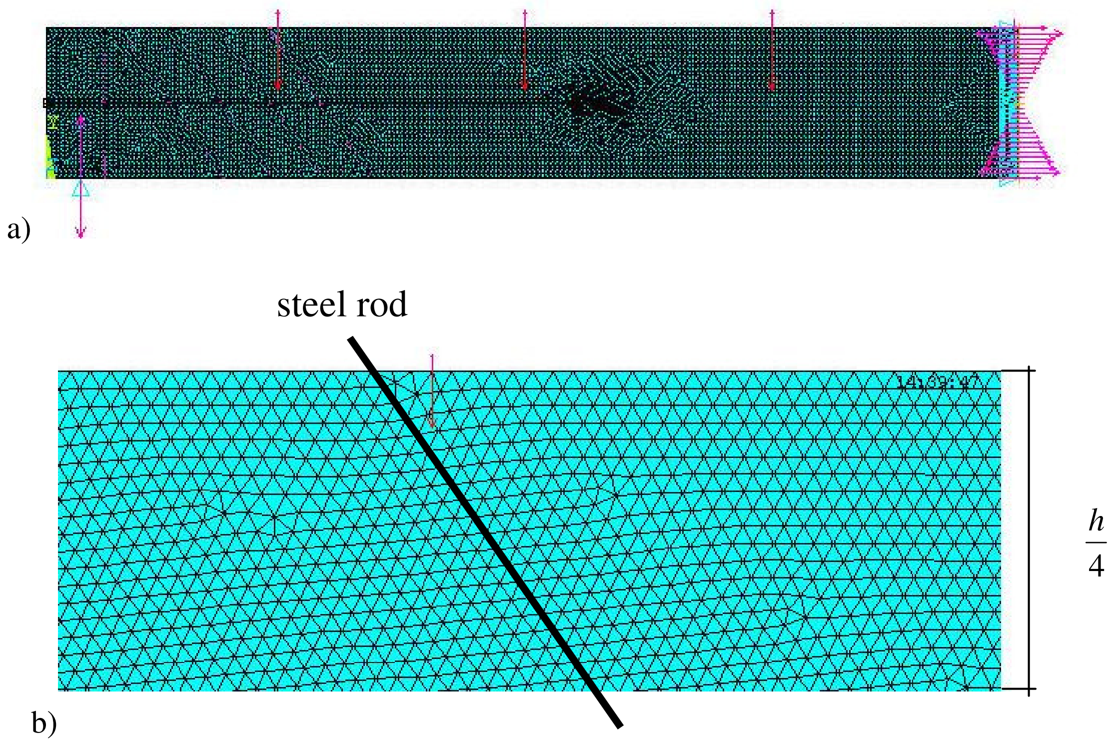


Figure 6 a,b: Finite element discretization of the investigated beam, a) total (half) of the structure with loading and boundary conditions, b) detail of reinforced area

All results for stresses and forces presented and discussed in the following refer to the experimentally obtained failure load level of the applied single forces of $F_u = 41$ kN.

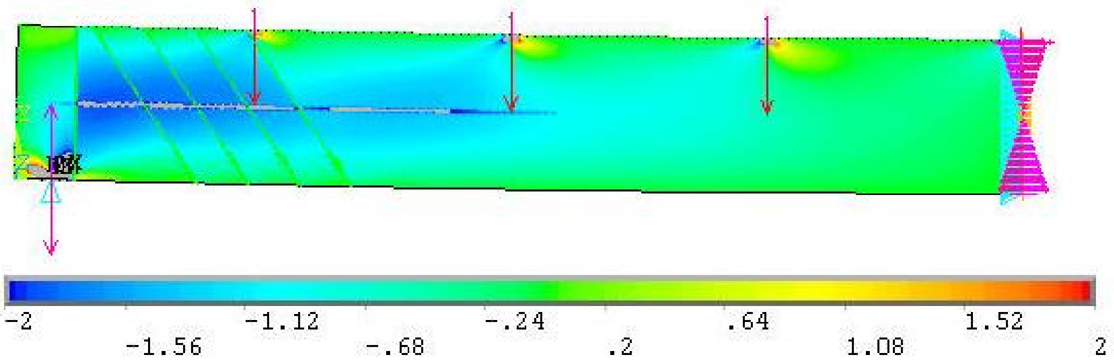


Figure 7: Contour plot of shear stresses in the timber

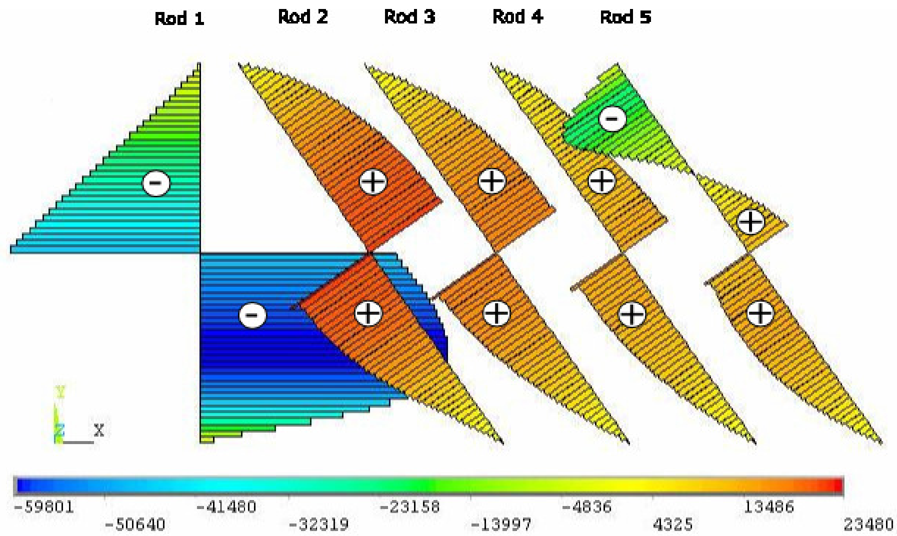


Figure 8: Axial forces in the steel rods

In a first, more qualitative overview, Figs. 7 and 8 show a contour plot of the shear stresses in the timber and the distribution of the axial forces in the steel rods, respectively. The presented graphs confirm the intuitive assessment of the stress/load distribution. Shear stresses in the wood in the highest shear stress area of the beam between support and first single load decrease as the successive arrangement of the inclined steel rods contributes to the shear force take-up. Similarly plausible are the distributions of the axial rod forces. Vertical rod Nr. 1 directly at/over the support is exclusively loaded in compression and the inclined rods No. 2, 3 and 4 transfer tension forces as a result of an internal frame action. Rod No. 5 starting at the bending compression edge almost directly at the application of an external single load is first loaded in compression and then contributes, similar as rods No. 2 to 4 by a tension force to the shear force take-up.

5 SHEAR STRESSES IN THE GLULAM

Figures 9 and 10 give the shear stresses in the timber of the glulam beam in selected sections parallel and perpendicular to beam axis within the area of highest shear forces. Figure 9 presents the distribution of shear stress τ_{xy} along beam length direction in sections y = const. = h/2, y = const. = h/2 + 15 mm and at y = const. = h/2 + 25 mm, so at mid-depth where gross cross-sectional width is reduced by 50% by the saw cuts and at two sections slightly above mid-depth in the unreduced cross-section.

It can be seen that the three curves, or their envelopes in case of mid-depth shear stress, are steadily decreasing from the maximum value close to the support to the position of rod No. 5 farest from support and already located in the beam area of second highest shear forces. Further, it can be seen that pronounced drops of the mid-depth shear stresses occur at the positions where the regarded stress path (= neutral axis) intersects with the steel rods, there acting as shear connectors and attracting the shear flow due to their high stiffness compared to wood. The difference of the shear stresses at mid-depth and at h/2 + 15 mm where width is not reduced is less pronounced ($\tau_{h/2}$ / $\tau_{h/2\,+\,15}$ = 1.5) as supposed from the analytical approximation where stresses differ by a factor of 2.

Figures 10a, b show the shear stress distribution along cross-sectional depth at five sections x = const.. The sections were chosen such that they intersect with the neutral axis at mid-distance between the steel rods No. 1 and No. 2, No. 2 and No. 3, No. 3 and No. 4 and No. 4 and No. 5, respectively. Further the section which hits rod No. 2 in the neutral axis is given. All stress distributions are quasi parabolic, as anticipated, except for the 5 mm at mid-depth where a stress peak is located in the reduced cross-section. When comparing the finite element computed maximum shear stresses at h/2 and closely thereto in the unreduced cross-section with the crude analytical approximations the following can be stated: The ratios between the maximum shear stresses obtained by the FE-computation in the reduced and full cross section vs. the crude analytical values are quantitative indicators of the effect of the retrofitting effect. The ratios vary in the reduced cross section from 2.8/4.0 = 0.7 at the support to 2.5/4.0 = 0.63 at the end of the field with the highest shear forces. In the unreduced cross-section the respective differences of the shear stresses are considerably smaller and range from 1.8/2.0 = 0.9 at the support to 1.6/2 = 0.8. The determination of the total shear force reduction in the bulk timber by integration of the timber shear stresses will be discussed in a separate paper.

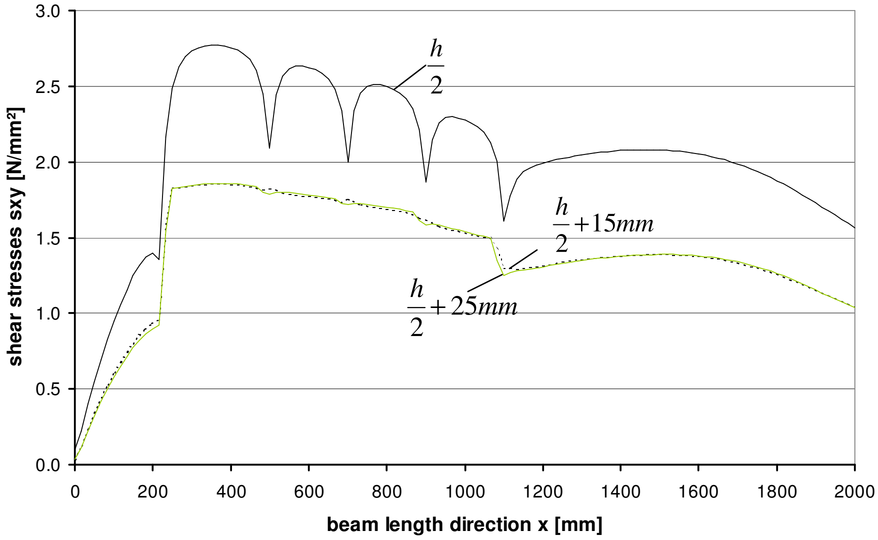
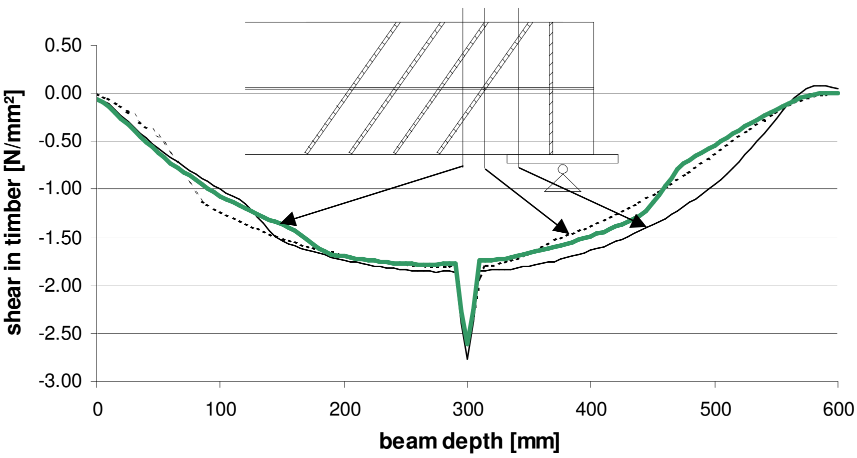


Figure 9: Shear stress distribution in the timber at sections y = const = h/2 y = const = h/2 + 15mm and y =const = h/2+25mm



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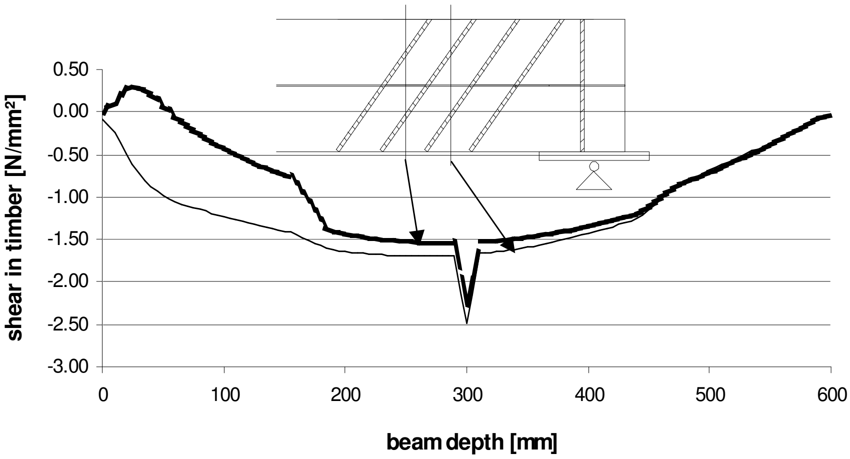
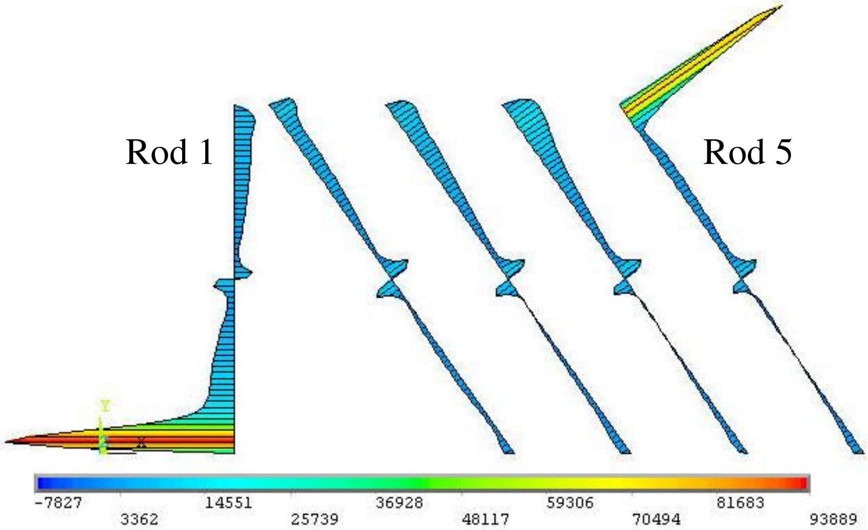


b)

Figure 10 a, b: Shear stress distributions along cross-sectional depth

6 FORCES, MOMENTS AND STRESSES IN THE STEEL RODS

In addition to the axial forces of the steel rods depicted in Fig. 8 which represent the primary load carrying contribution of the rods, Figs. 11a and b show the bending moments and the shear forces acting on the rods. As in case of the axial forces in the rods No. 1 and No. 5 also the bending moments and shear forces are highly influenced by the direct load application situation. Apart from this, considerable increases of the section forces in the section of the reduced width are evident.



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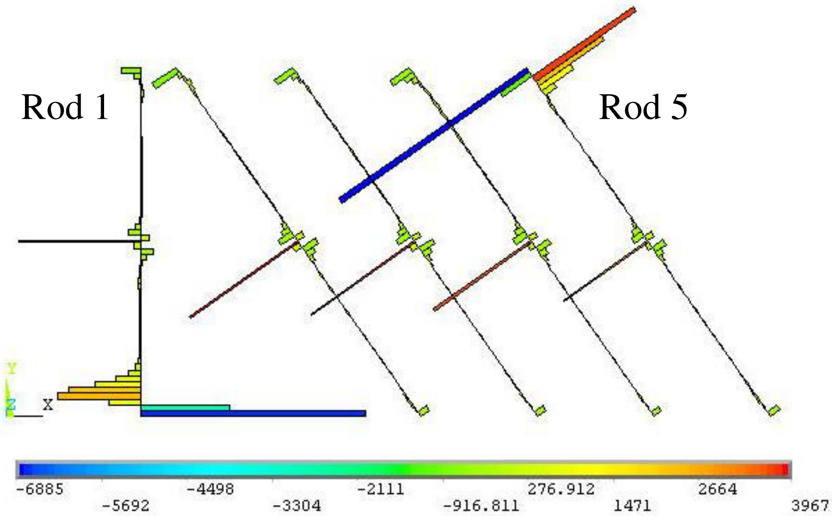


b)

Figure 11 a, b: a) bending moments, b) shear forces of the rods 1 to 5 (from left to right)

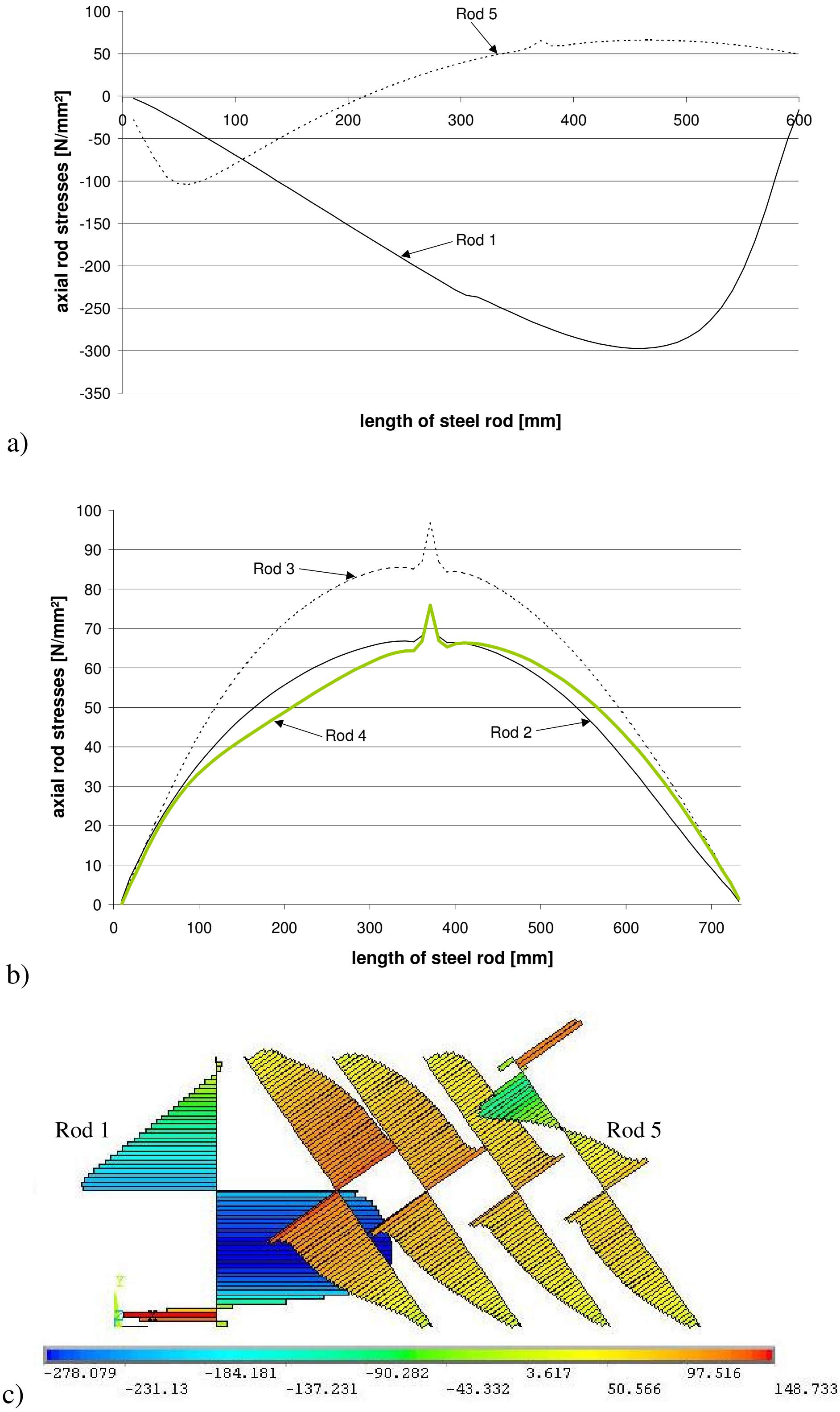


Figure 12 a,b,c: a) and b) axial stresses of the steel rods, c) bending stresses of the steel rods

Figures 12a and b give the axial stresses of the rods. Figure 12c shows the maximum bending stresses of the rods, revealing the non-neglectible influence of the bending moments.

7 SHEAR STRESSES IN THE ROD – ADHESIVE – TIMBER INTERFACE; DESIGN APPROACH

Figures 13a and b give the interface shear stresses along the rods No. 1 to 5 starting from the bending compression edge.

The course of the stresses conforms qualitatively to the anticipated shape although being more unsymmetric versus mid-depth of the beam as expected. The maximum bond line stresses τ of rods Nr. 3 and 4 are 4.5 and 5.6 N/mm^2, respectively. Following briefly the design approach is given. The lay-out and the design of the up-grading provisions was to ensure an almost fully elastic behaviour of the beams up to their respective bending capacity in the range of 24 to 30 N/mm^2. This should hold also in case of severe cracks in the support areas which might reach up to 50% of cross-sectional width. This necessitates as shown that bond line shear stresses up to about 5 N/mm^2 can be transferred. Further more, in case of an eventually fully cracked cross section the reinforcement should provide a very high shear connector effect for the resulting mechanically jointed two-part compound. According to DIN 1052:2004, the characteristic bond line capacity of a glued-in rod with an adhesive approved for the specific purpose can be assumed as

$$f_{v,k} = 5.25 - 0.005 \cdot \ell_{ad} \quad [\ N/mm^2\ ]$$

where ℓ_{ad} is the effective bond line length in mm. In the given case, with $\ell_{ad} = h / (2 \cdot \sin 55°) - 25$ mm $\cong$ 340 mm, the characteristic bond line capacity is obtained as

$$f_{v,k} = 3.6\ N/mm^2.$$

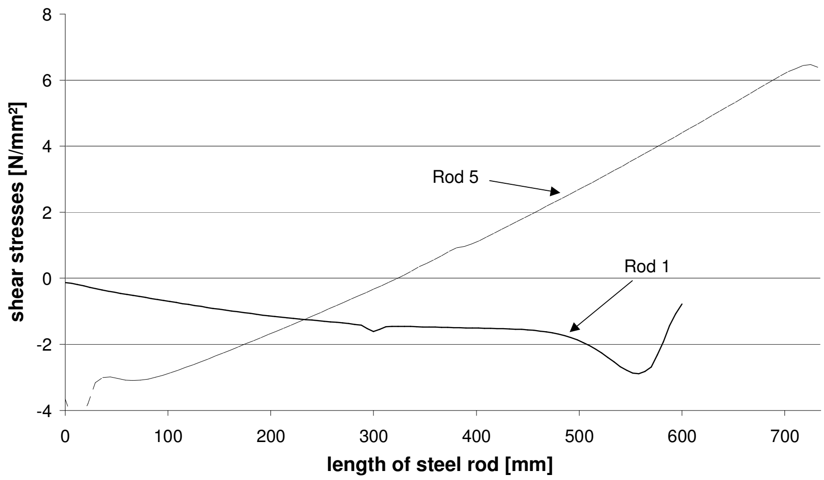
The load bearing capacity of the specific epoxy-adhesive can be defined more precisely (Aicher, 2001; 2003) as

$$f_{v,k} = 60 \cdot d_{nom}^{-0.03} \cdot \ell_{ad}^{-0.47} \quad [\ N/mm^2\ ]$$

with d_{nom} and ℓ_{ad} in mm.

The mean strength $f_{v,mean}$ is obtained from the above equation with the factor of 77 instead of 60. The given equations yield $f_{v,k} = 3.8\ N/mm^2$ and

$f_{v,mean}$ = 4.9 N/mm². So, on the mean bond line strength level deemed sufficient in the given case, the relevant bond line shear stresses can be transfered. This is in accordance with the performed beam test.



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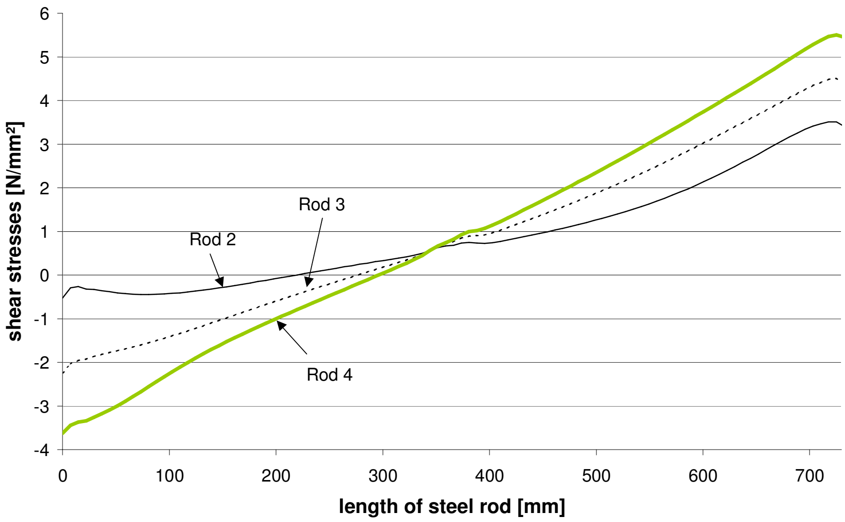

b)

Figure 13 a, b: Shear stresses in the timber on a path along the respective rods

8 CONCLUSIONS

The paper reports about some aspects of a large timber construction retrofitting project where the shear reinforcement of cracked glulam beams was performed with glued-in rods.

A test with a full-sized specimen with a span to depth ratio of 12 had been performed whereby cross-sectional width of the beam was reduced in the area of highest shear forces at both face sides with a saw cut to one half of the gross width over a length of about 2 m. The remaining net width was reinforced by four inclined threaded steel rods of 16 mm diameter bonded in the timber with a special epoxy resin.

The beam failed in bending whereby the support areas with the highest shear stresses remained fully intact. Some results of finite element computations on relevant stresses and forces in the glulam and in the steel rods are presented. The stress distributions conform qualitatively to engineering assessment. According to the calculations, the bond line stresses of the reinforcements reach the mean bond line shear capacity at about bending capacity. Furthermore, the analysis revealed that a shear failure of the structure well below the load level given by the bending capacity is unlikely. This was validated by the performed full scale test. An appropriate analytical model for the reinforcement design is being developed.

The study revealed that inclined glued-in steel rods represent a versatile and easy to apply method for shear reinforcement of glulam beams with or without cracks.

REFERENCES

DIN 1052:2004 Design of timber structures – General rules and rules for buildings

Aicher, S. 2001 Characteristic axial resistance of threaded rods glued-in spruce dependent on adhesive type – a complementary database for the GIROD project. Research Report, Otto-Graf-Institute, University Stuttgart

Aicher, S. 2003 Structural adhesive joints including glued-in bolts. In: Timber Engineering, Eds. S. Thelanderson, H. J. Larsen, John Wiley & Sons