HYBRID TIMBER-STEEL-CONCRETE GIRDERS – A SUSTAINABLE ALTERNATIVE FOR HEAVY DUTY APPLICATIONS

HOLZ-STAHL-BETON-HYBRIDTRÄGER – EINE NACHHALTIGE ALTERNATIVE FÜR SCHWERLAST-ANWENDUNGEN

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SUMMARY

In an ongoing applied federally supported research project, a composite beam with a high proportion of wood for heavy-duty applications is being developed and investigated experimentally and computationally. The composite cross-section consists of a special combination of laminated veneer lumber and glulam layers in the cross-section web combined with a steel profile in the bending tension zone and a steel reinforced polymer concrete layer at the bending compression edge. The connection between the wood web and the steel and concrete layers is realised with inclined threaded bolts glued into the wood. First results of a bending test with a full-scale prototype and principles of a three-dimensional finite element simulation of the hybrid beam are presented.

ZUSAMMENFASSUNG

In einem laufenden ZIM-Forschungsprojekt wird ein Verbundträger mit hohem Holzanteil für Schwerlastanwendungen entwickelt und experimentell wie rechnerisch untersucht. Der Verbundquerschnitt besteht aus einer speziellen Kombination aus Furnierschichtholz und Brettschichtholzlagen im Querschnittskern sowie einem Stahlprofil im Biegezugbereich und einer stahlbewehrten Polymerbetonschicht am Biegedruckrand. Die Verbindung zwischen dem Holzkern und der Stahl- bzw. Betonschicht erfolgt mit geneigt angeordneten, in das Holz eingeklebten Gewindekopfbolzen. Es werden erste Ergebnisse einer Biegeprüfung an einem vollmaßstäblichen Prototyp und Grundlagen einer dreidimensionalen Finite-Element-Simulation des Hybridträgers vorgestellt.

1. INTRODUCTION

Modern wood-based materials such as glued-laminated timber (GLT), cross-laminated timber (CLT) and laminated veneer lumber (LVL) are increasingly used in new areas of application in civil engineering constructions, such as wide-span hall structures or multi-storey buildings, even beyond the limits of sky scrapers (see e.g. [1]). Due to the low density of the renewable raw material, pure wood constructions are especially advantageous for structures predominantly loaded by the self- weight of the structural elements. For heavy-duty applications – such as e.g. traffic structures – timber structures are still not very widespread due to the need for considerably larger cross-sections compared to other building materials such as steel and concrete. Although investors of modern infrastructure projects such as monorail systems have to pay increasingly attention to sustainability aspects, and in particular to the CO_2 balance of the building materials used, it has not been possible to date to realise e. g. the elevated guideway girders for monorail systems based on wood materials. Due to the limited mechanical performance of the natural material wood the restricted component heights, dictated e.g. by the necessary clearance gauge in the urban space, could not be achieved even by using modern wood materials.

In an on-going "ZIM" research project the MPA University of Stuttgart develops and investigates a wood based heavy duty beam build-up in collaboration with a GLT manufacturer and a steel construction company. The goal of the study is an optimal combination of different materials with targeted exploitation of the respective advantages. Thus, steel is used in the tensile and polymer concrete in the compression zone of the beam. The web of the composite beam – mainly loaded by shear stresses – consists of a tailored combination of solid wood glulam and laminated veneer lumber with cross-layers.

Some aspects of the basic idea have been investigated before (see e.g. [2]). However, these earlier studies did not deliver satisfactory results especially with respect to a sufficient stiff and strong connection between the timber web and the concrete and steel layers and with respect to a satisfactory shear capacity of the wooden web. Besides some novel details for the connections and the structure of the wood-based web the here reported study differs from earlier research attempts by a more consistently followed aim of both high load and stiffness capacity without major restriction with respect to an esthetical "wooden experience". However, the goal should on the other hand still be achieved with the highest possible proportion of wood within the hybrid beam lay-up.

2. CROSS-SECTIONAL BUILD-UP

The build-up of the hybrid heavy duty beam element is sketched in Fig. 1 which shows the realised cross-sectional lay-up and the principle arrangement of the fasteners of a prototype beam.

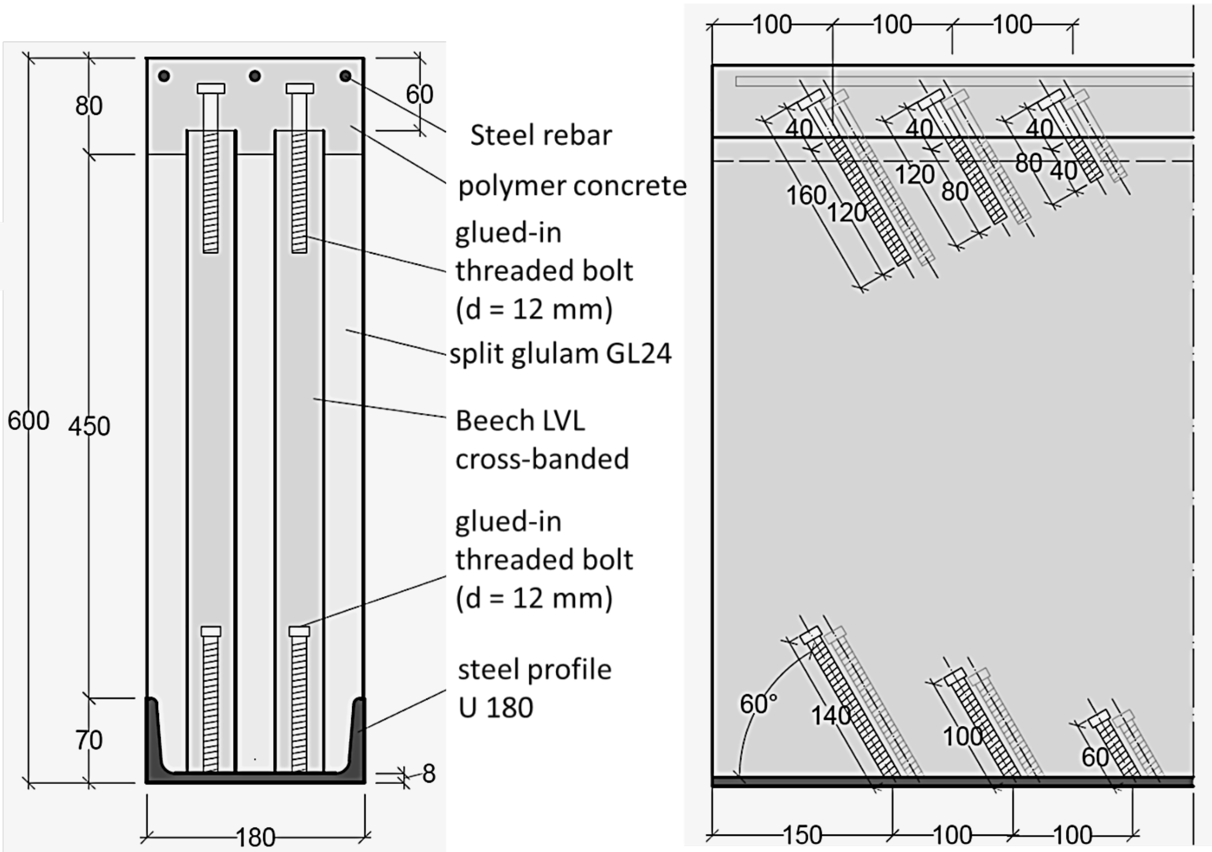


Fig. 1: Lay-up of the hybrid beam

2.1 Wooden web

One of the most innovative details is the composite web consisting of several layers of two different wood-based materials: on the one hand layers of split glued-laminated timber made of solid softwood and on the other hand LVL plates with cross-layers made of hardwood veneers. The softwood glulam slices and the cross-banded LVL plates are bonded together alternatingly.

The sketched layered build-up of the wooden web aims at partly competing goals to be achieved:

- *The shear strength should be significantly higher compared to GLT*: therefore, LVL with cross-layers has been chosen, whereby the cross-layers act as shear strengthening.

- *The web should exhibit a rather high bending capacity:* LVL with cross-layers exhibits lower bending properties than unidirectional LVL. Therefore, high strength LVL made of beech wood has been chosen resulting in a bending strength larger than solid softwood glulam despite of the cross-layers.

- *The proportion of beech LVL should be limited, as the material is quite costly.* This is one of the reasons, why not pure beech LVL, but a compound with considerably cheaper softwood GLT, has been chosen.

- *The beam should be applicable at (sheltered) outdoor conditions:* This is the second reason for the compound lay-up: The outer layers made of solid wood glulam should act as a climate protection for the quite moisture sensitive beech material.

The realised build-up as sketched in Fig. 1 is not yet completely optimised with respect to width ratios, because some boundary conditions – i.e. maximal total width due to limited testing equipment and fixed thickness of available beech LVL plates – had to be considered. However, it has prospect to meet all the above defined targets.

2.2 Bending compression edge: Steel bar reinforced polymer concrete layer

The bending compression edge of the beam is made by a special high strength reinforced polymer epoxy concrete (PRC) with very advantageous shrinkage properties. Two ribbed steel bars with a diameter of 10 mm were inserted as structural reinforcement.

The load transfer between the PRC layer and the wood-based web is realised by two rows of threaded head bolts glued into the beech LVL plates by means of an apt structural epoxy adhesive. The bolts were arranged in an angle of 60 ° to the main fiber direction of the LVL revealing an improved transfer of the shear loads. The bolt inclination is symmetric to beam mid-length. The bolt heads increase the compound effect when bonded into the concrete layer. In order to avoid a continuous shear plane in the wooden web screws with three different lengths, arranged alternatingly along beam length, have been chosen (see Fig. 1).

2.3 Bending tensile edge: Steel profile

The bending tensile edge of the cross-section is strengthened by a U-type steel profile. The connection to the beech LVL plates is performed by two rows of threaded bolts welded to the steel profile and glued to the LVL. The bolts are inclined with the same angle to main fiber direction as in the case of the compression zone. The bolts – with three different lengths to avoid a continuous shear plane – are glued into the LVL by means of the same epoxy adhesive as in case of the compression zone.

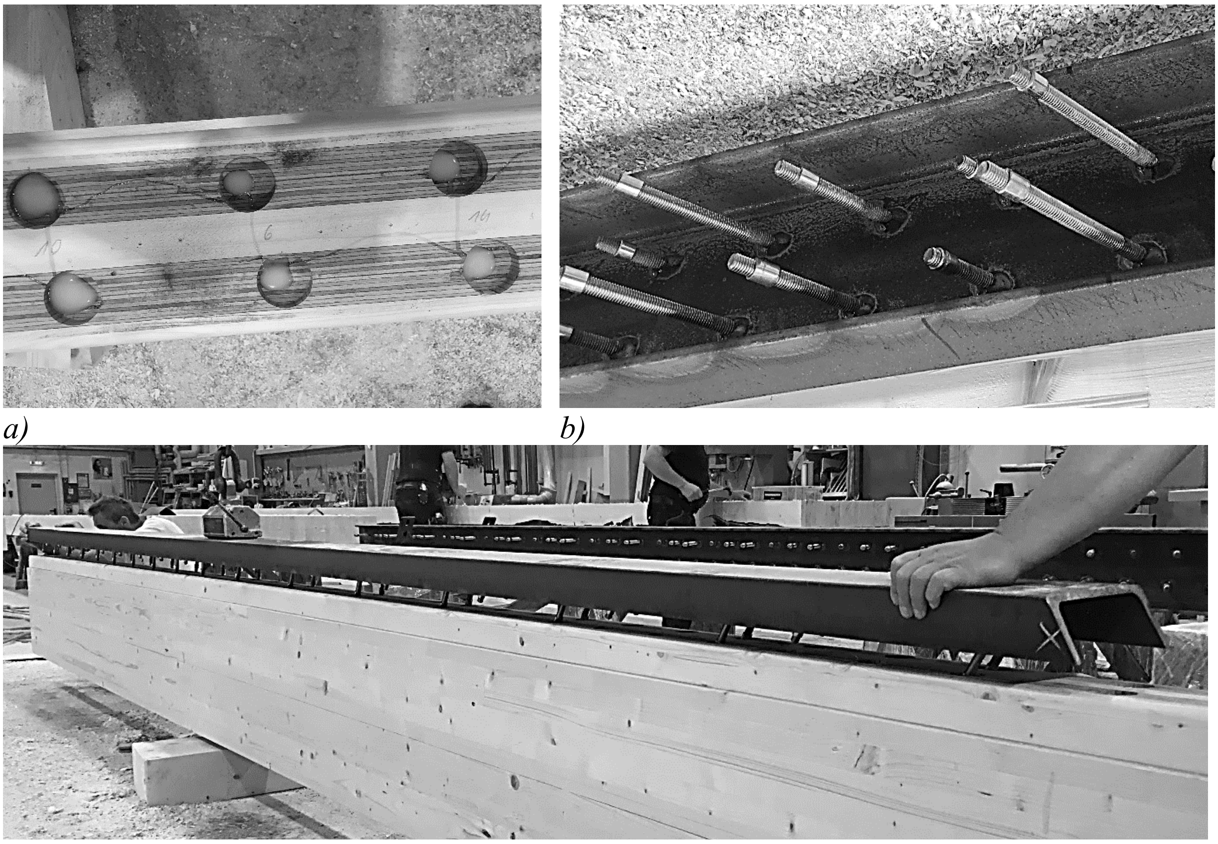
3. PRODUCTION OF PROTOTYPE

In order to check the feasibility of the designed cross-sectional lay-up a prototype has been produced by the glulam company STRAB, Hermsdorf, in collaboration with steel construction company Sachse, Hermsdorf. The production process can be subdivided in five steps i) to v) sketched below.

i) In a first step a standard glulam beam of strength class GL24h according to European harmonised standard EN 14080 [3] made of finger-jointed spruce laminations was produced. The dimensions were: height h_{web} = 520 mm, b_{glulam} = 105 mm and length L = 12000 mm. The beam was then vertically split / resawn with respect to width direction into three slices, each with a width b_{slice} = 33 mm. The GLT-slices were then bonded in alternating order with two LVL plates, whereby the outer layers were formed by the spruce solid glulam slices (see Fig. 1). The cross-banded LVL plates with a thickness of 40 mm consisted of beech veneers with cross-layers (build-up III – IIIIII – III) and conformed to the harmonised standard EN 14374 [4] for LVL. The bonding was performed by means of a phenolic resorcinol adhesive system apt for block-gluing and certified for the species spruce and beech. The fiber direction of the solid wood slices aligned with the main direction of the LVL veneers. The cross-layers of the LVL were oriented perpendicular to the beam axis and hence parallel to cross-sectional depth direction.

ii) In a second step the holes for the bolts were drilled into the narrow faces of the two LVL web-plates at an angle of 60 °(see Fig. 2a). The distance between the holes in length direction varied according to the expected shear force profile starting from 100 mm (next to the end grain faces) to 150 mm near mid-length. Two rows of bolts had edge distances of 53 mm in width direction and a spacing of 73 mm in between. In total 204 holes were drilled into both narrow faces, i.e. at the bending compression and tensile edges of the beam. The hole diameter was chosen 1mm larger than the diameter of the heads of the bolts.

iii) In step three at the bending tensile edge of the beam two U-profiles – each with the half beam length – were equipped with the inclined threaded bolts by welding (see Fig. 2b). The U profile consisted of two halves in order to enable the mating of the U-profiles with the timber web, as the angle of the inclined bolts / holes changes at mid-length. The bolts were arranged in such manner to exactly fit into the drilled holes in the lower narrow edges of the LVL plates. The holes were filled with epoxy resin and the U-profiles with the welded-on bolts were pressed into the timber, resulting in a glued connection of all bolts (see Fig. 2c).



a) *b)* *c)*

Fig. 2: Fotos of the production steps i)-iii) related to the bending tensile edge
a) holes drilled into the beech LVL plates and filled with expoxy adhesive for the threaded bolts
b) upper right: threaded bolts welded to the inner face of the steel U-profile
c) lower: connection of the steel profile (with welded-on bolts) to the wooden web

iv) In a second last step the bending compression edge was composed. Firstly, the holes at the compression edge were filled with expoxy resin and the threaded bolts were glued into the holes with protruding bolt heads of 40 mm length. A formwork was then attached to the wide faces of the wooden web at the compression edge and the threaded rebars were positioned. Finally, the formwork was filled with the polymer concrete, hereby bonding the screw headed bolts into the PRC flange. Fig. 3a shows the last production step of the compression edge, i.e. the filling with polymer concrete, and Fig. 3b gives a view of the realised cross-sectional lay-up.

v) In a final step the two parts of the U-profile were connected at mid-length by a welded splice plate.

a) *b)*

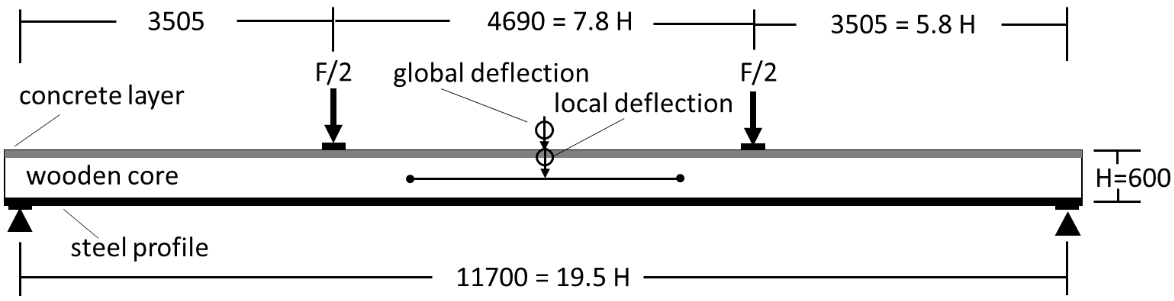
Fig. 3: Fotos of bending compression edge production and of the completed cross-section
a) bending compression edge being formed by pouring polymer concrete into the formwork: also visible are the glued-in headed bolts and the steel rebars
b) view of the complete hybrid cross-section

The production of the prototype beam proved the principal feasibility of the layup concept. However, it showed also the quite high effort with many handcraft steps. It turned out that especially the inclined orientation of the bolts in conjunction with the welding of the bolts to the U-profile caused most of the effort which could be minimised by an automated robotic welding process. It is further planned for a second phase of the research project to study alternative lay-ups with either not-inclined bolts and / or screw connection instead of welding between the steel profile and the bolts.

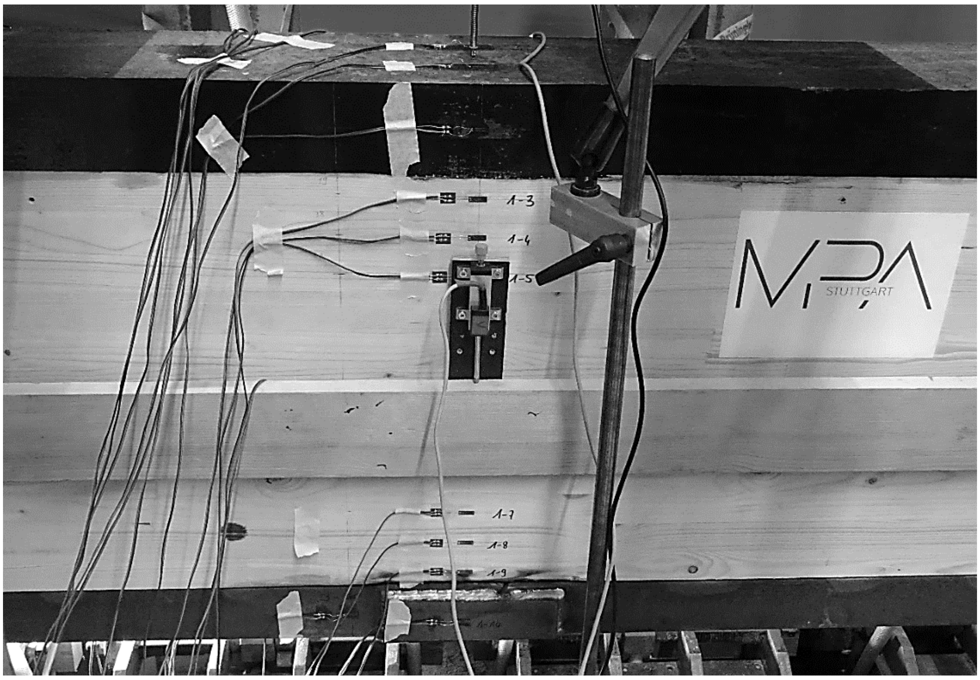
4. BENDING TEST: SET-UP AND PERFORMANCE

The prototype has been tested in a 4-point bending test, symmetric to mid-span. The span between the supports was l_0 = 11700 mm, i.e. 19.5 times the beam depth of H = 600mm. The distance between the support and the next loading point, i.e. the lever arm, was a = 3655 mm. The span between the two loading points (i.e. the zone of constant bending moment) was l_{loads} = 4690 mm. The test set-up sketched in Fig. 4a conforms widely with the standard EN 408 [5]. The global deflection at mid-span and the local deformation in the zone of constant moment were measured with LVDTs according to the provisions of EN 408. In addition to the displacement measurements strain gauges oriented parallel to the beam axis were applied to the beam surface at mid-span. In total 18 strain gauges were positioned at a line over the whole depth starting at the upper narrow face of the beam (at the concrete surface), continued at the side faces of the glulam laminations to the steel profile at the lower narrow face of the beam. Additionally, two strain gauges were positioned at the splice plates welded to the side faces and at the bottom of the U-profile exactly at mid-span. Further, two strain gauges were positioned directly to the side and bottom faces of the U-profile outside the area of the connecting plate, shifted 150 mm from mid-span. A photograph of the installed measurement equipment at or next to mid-span is given in Fig. 4b.

The test was performed in a large servo-hydraulic test machine with variable load frames and pistons.



a)



b)

Fig. 4: Test lay-out of the bending test
a) Sketch of the loading scheme
b) Photo of the measuring equipment installed at mid-span

The ramp load bending tests with stepwise increased loads were performed in deformation control with a constant displacement rate of 20 mm/min. Several load steps with loading / unloading cycles have been performed.

5. BENDING TEST: SET-UP AND PERFORMANCE

Fig. 5 shows the global load-deflection curve (continuous line) of the last load-step until F_{max} = 301 kN, which exhibits an almost completely linear behaviour. Remark: The dashed and thick continuous curves also given in Fig. 5 represent calculation results from the FE-model presented in section 7 of this paper.

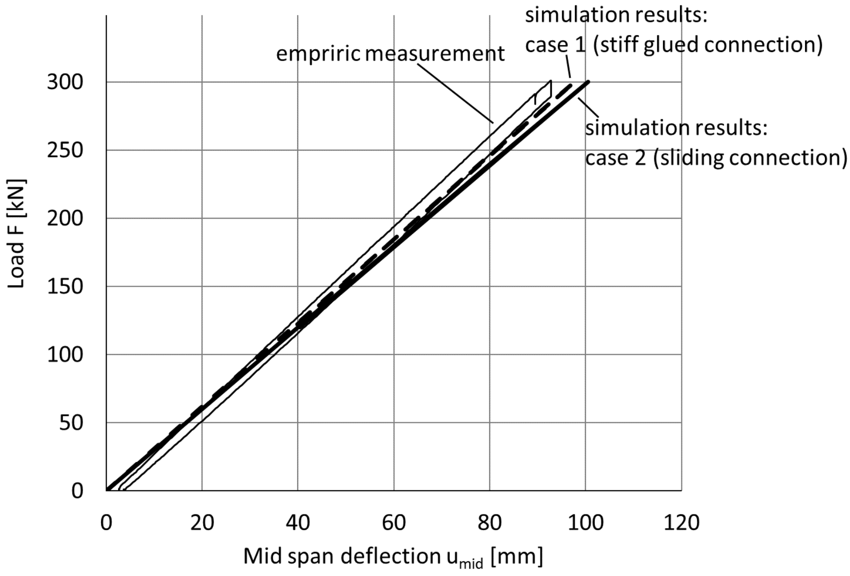
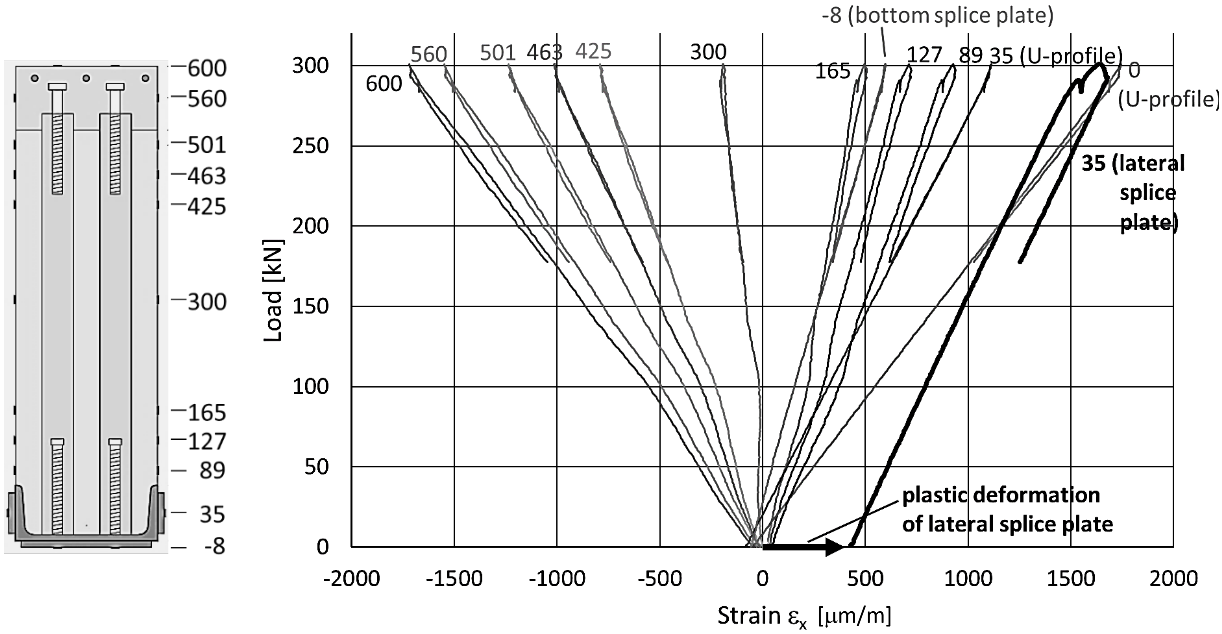


Fig. 5: Global load-deflection curves: experimental data (thin continuous line), modelling results for case 1 (dashed line) and modelling results for case 2 (thick continuous line)

Figs. 6a and b show the position of the strain gauges and the respective load-strain curves for the last load step until 301 kN. Each line represents the mean value of two strain gauges mounted oppositely at both wide faces of the beam or of two positions on the narrow edge of the beam, respectively.

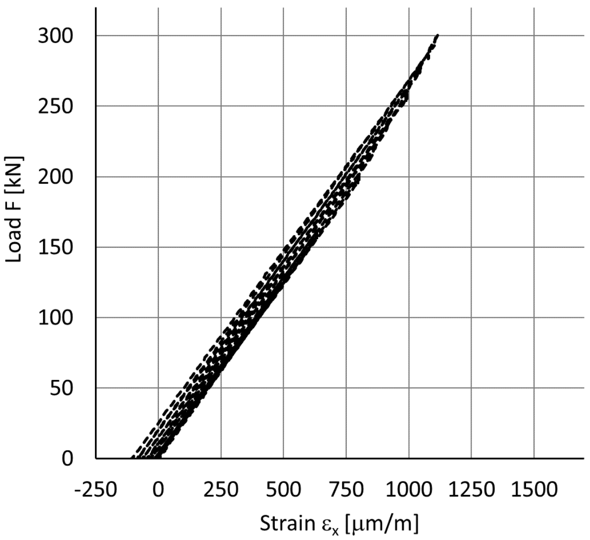


a) b)

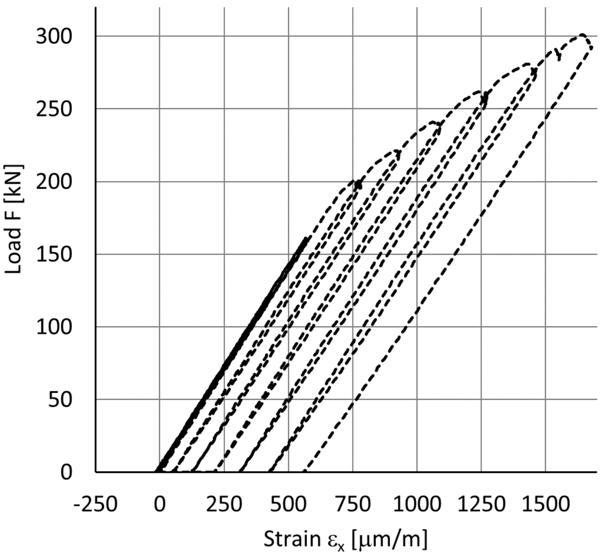
Fig. 6: Strain measurements
a) Location of the strain gauges (cross-section at mid-span)
b) experimental load-strain curves for the last load step (up to 301 kN)

The test was stopped when the measured strains at the lower face of the U-profile were in the range of the expected yield strains. In most of the measured load-strain curves, however, hardly any sign of yielding could be detected with exception of the strain gauges located at the splice plates at mid-span welded to the side faces of the U-profile.

In order to show the completely different behaviour of the strain gauges at the U-profile and at the lateral splice plates located directly at the gap of the two parts of the U-profile (both at the same depth of y = 35 mm) the load-strain curves for different load steps are given in Fig. 7 a and b. The continuous lines in both figures show the linear behaviour for the case of the first step until 160 kN. The dashed curves show the following load steps with ultimate loads of 200 kN, 240 kN, 260 kN and finally 301 kN. In case of the strain gauges at the flanges of the U-profile the behaviour is almost perfectly linear. However, for higher load steps a minor shift of the load-strain curves to smaller strains was encountered. In contrast hereto, the strain measurements on the lateral splice plates located directly at the gap of the two parts of the U-profile show a nonlinear behaviour clearly influenced by yielding. This comparison also explains the shifted position of the respective load-strain curve for the strain gauge on the splice plate in Fig. 6b, which starts at F=0 for the last load-step at a cumulative yielding strain of 460 µm/m.



a)



b)

Fig. 7: Empirical strain results at the side of the steel profile (y = 35mm)
a) Measurement directly at the U-profile (shifted from mid-span)
b) Measurement at the splice plate (exactly at the gap at mid-span)

As the tests were stopped before the U-profile showed pronounced yielding and before the timber and concrete exhibited signs of damage, it was possible to perform additional tests, e.g. in cantilever mode, with the same prototype specimen. The results of these tests are beyond the scope of this paper.

6. FINITE ELEMENT MODELLING: PRINICIPLE APPROACH

Accompanying the experimental investigations, a 3D Finite-Element model has been set up. The model serves two purposes: In a first step the model helps for the quantitative evaluation of the experiments. In a second step the model predictions will in future be used to set up a

i) design concept – not only for bending – and

ii) to support optimisation of the beam lay-up.

The 3D model is implemented into the FE software ABAQUS. The FE mesh represents the different materials PCR-layer, solid wood glulam, beech LVL, steel. The PCR and the wood layers are modelled by means of 3D 8 node solid brick elements (type: C3D8R), whereas the steel profile and the steel splice plates are represented by means of 3D solid shell elements (type: CSS8). As the solid wood glulam and the LVL are bonded according to plan by a thin glue-line this connection was simply modelled by merging the nodes of the elements of the two materials. In the case of the compression zone the character of a possible – but not planned – complete bonding between the wooden web and the concrete layer is not well defined. Therefore, two versions of the model – representing the extreme cases – have been set up and simulations with both versions have been performed:

- "case 1" with complete bonding, i.e. direct connection by means of common nodes between polymer concrete and wood and
- "case 2" with some sliding behaviour of the interface represented by surface-to-surface contact elements between the two materials.

The glued-in threaded bolts have been idealised by means of 1D two-node beam elements (type: B31 with the stiffness of the chosen screw types) connected on one hand to the concrete – representing the cast connection in the concrete – and on the other hand to the wooden web – representing the glued-in connection. In "case 1" the shear load between concrete and wood is transferred simultaneously by the assumed completely rigid bondline and by the glued-in headed threaded bolts, whereas in "case 2" only the glued-in bolts transfer the shear forces.

The connection between steel and wood was modelled in the same manner: Two cases were considered, one with a glued connection between steel and wood and one with sliding behaviour of the interface. The bonded-in threaded bolts were modelled as beam elements directly connected to the wood (i.e. representing the bonding by expoxy adhesive) and stiffly connected to the steel profile, thus representing the welded connection.

Taking advantage of symmetry conditions exclusively one quarter of the beam has been idealised. The supports were modelled as sliding bearings. The load was applied as a single load acting on a support steel plate – similar to the realised test set-up.

The material parameters for the different materials were taken from the respective technical specifications, i.e. values from standard EN 14080 in case of solid wood glulam or from declaration of performance of the different materials, e.g. for the used beech LVL with cross layers. Thus, the assumed values are nominal minimum characteristic values guaranteed by the producers. However, it should be assumed, that the effective parameters are higher than the declared ones.

Deviating from the declared values the yield stress level of the U-profile and of the welded-on steel splice plates, nominally declared as strength class S355, has been chosen to 450 N/mm^2 as a result of the comparison of the simulation results with the empiric strain measurements.

Fig. 8 shows a 3D view the model, whereby the visible front face represents the symmetry plane at mid-span and the visible side face represents the symmetry plane at mid-width. At the bottom of the visible front face at mid-span the modelled gap of the two parts of the steel U-profile and the modelled welded-on splice plates at the side flanges and at the bottom of the U-profile are shown. The black lines represent the beam elements of the threaded bolts.

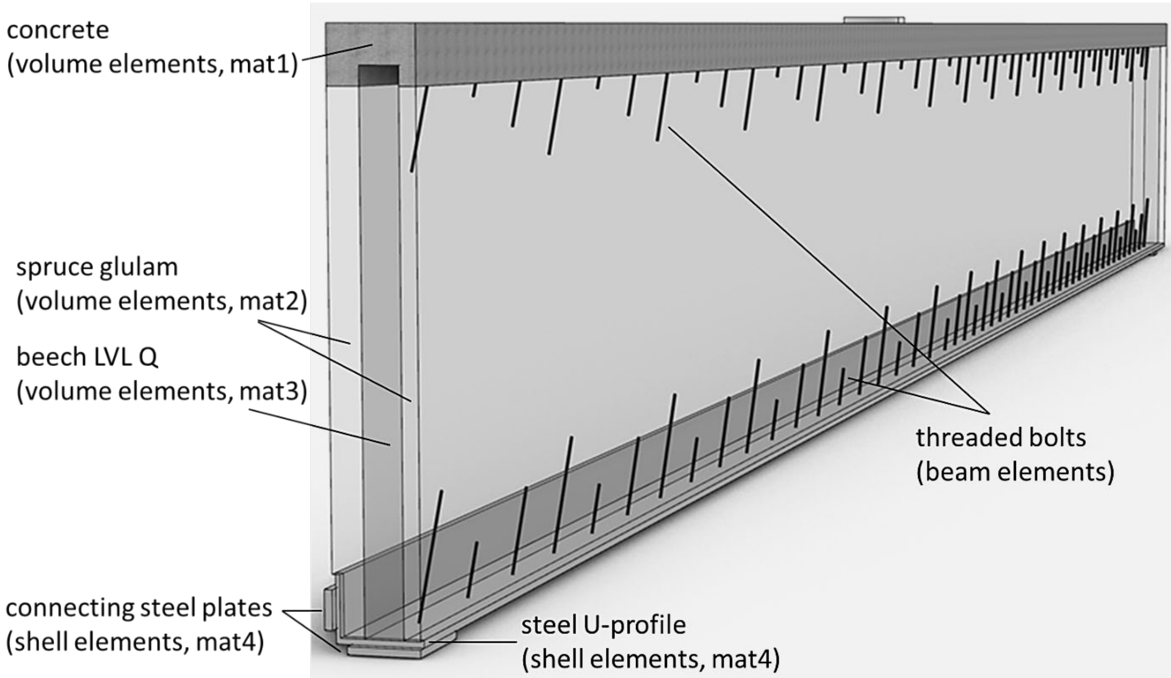


Fig. 8: 3D view of the FE-model of the beam build-up with the different materials / components

7. COMPARISON OF COMPUTATIONAL AND EMPIRIC RESULTS

A comparison of the computational and empiric results regarding global stiffness characterised by the load-deflection curves is given in Fig. 5, see above (section 4 of this paper). In Fig. 5 the continuous thin lines represent the results of the experimental deflection measurement. The thick dashed line gives the calculated FE result for "case 1", i.e. concrete and steel layers bonded stiffly to the wooden web and the thick continuous line represents "case 2", i.e. a) no bonding between steel / concrete and wood and b) connection between the outer layers and the wooden web solely by the glued-in threaded bolts. It can be seen, that the calculated and measured deflections coincide rather well. The two simulation cases do not differ significantly. Not surprisingly, the "case 2" without bonding of layers shows a somewhat smaller stiffness as compared to the completely bonded "case 1".

A comparison of the strain measurements (filled dots) and the results of the simulations (lines) are presented in Figs. 9a and b. The figures give the strain values (x-axis) at a load of 200 kN and 301 kN for different positions along cross-sectional depth (y-axis) exactly a mid-span. The strain results at depths between y = 55 mm to 600 mm refer to the wide faces of the hybrid beam. The strain results at depths between y = 15 mm to 55 mm refer to the outer faces of the connecting steel splice plates welded to the side flanges of the two parts of the steel U-profiles and bridging the gap at mid-span. The results for y = - 8mm represent the lower/outer surface of the connecting splice steel plate welded to the bottom edge of the steel U-profiles also bridging the gap.

Here, only the results of case 2 (glued bolts and sliding contact between steel/concrete and wood) are presented, as the results turned out to be nearly independent of the contact assumptions. The measured and simulated results show qualitatively and to some extent also quantitatively a quite good correlation. Especially the effect of the load concentration and the resulting local yielding of the connecting splice plates at the side faces are correctly predicted by the simulation results. Moreover – not to be expected at first glance – the pronouncedly lower local loading and thereby lack of yielding of the bottom splice plate at the narrow bending tension edge is correctly mirrored, too.

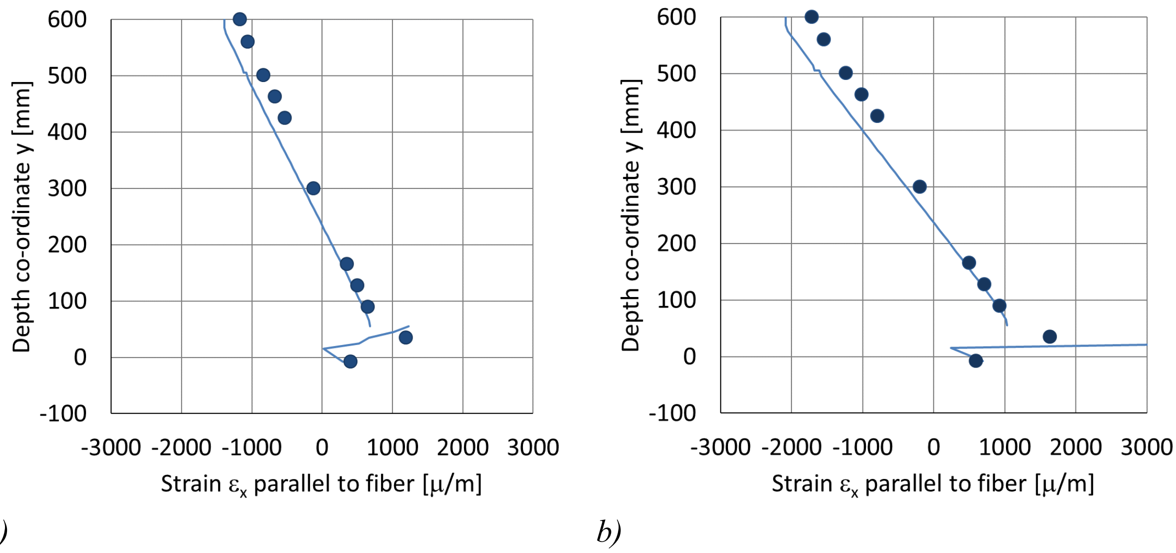


Fig. 9: Comparison of experimentally measured (dots) and calculated (lines) strain distributions $\varepsilon_x(y)$ along beam depth direction for an assumed yield stress level of 450 N/mm^2
a) Results for F/2 = 200 kN, b) Results for F/2 = 301 kN

The different experimental load-strain behaviour of several loading cycles of the strain at the lateral splice plates directly at the gap of the U-profile at mid-width (y = 35 mm) and of the strain at the U-profile shifted from mid-span by 150 mm (also y = 35 mm) is highlighted in Fig. 7. The respective simulation results are given in Fig. 10 for the different contact assumptions “case 1” (dashed line) and “case 2” (continuous line). The experimental observations are – in a qualitative sense – correctly mirrored by the simulations:

- The load-strain curves at the lateral splice plates bridging the gap at mid-span depicted in Fig. 10b show pronounced yielding caused by the local stress concentration at the gap. The yielding starts – as in the experiments – at loads higher than 180 kN. Quantitatively the total yield strains are higher than experimentally observed, which could mean that the elevated yield stress level might be chosen even too low. The contact "case 1" with the assumption of a stiff/glued connection between steel profile and wooden web shows a somewhat better agreement with the empiric data than the "case 2" with an assumed sliding between steel and timber.
- The load-strain curves at the U-profile shifted from mid-span showed (Fig. 10a) – in accordance with the experimental results – no yielding. However, at the unloading branch some shift to lower strain values can be seen, which can be explained by load redistribution or local "unloading" effects caused by the local yielding area at the lateral splice plates.

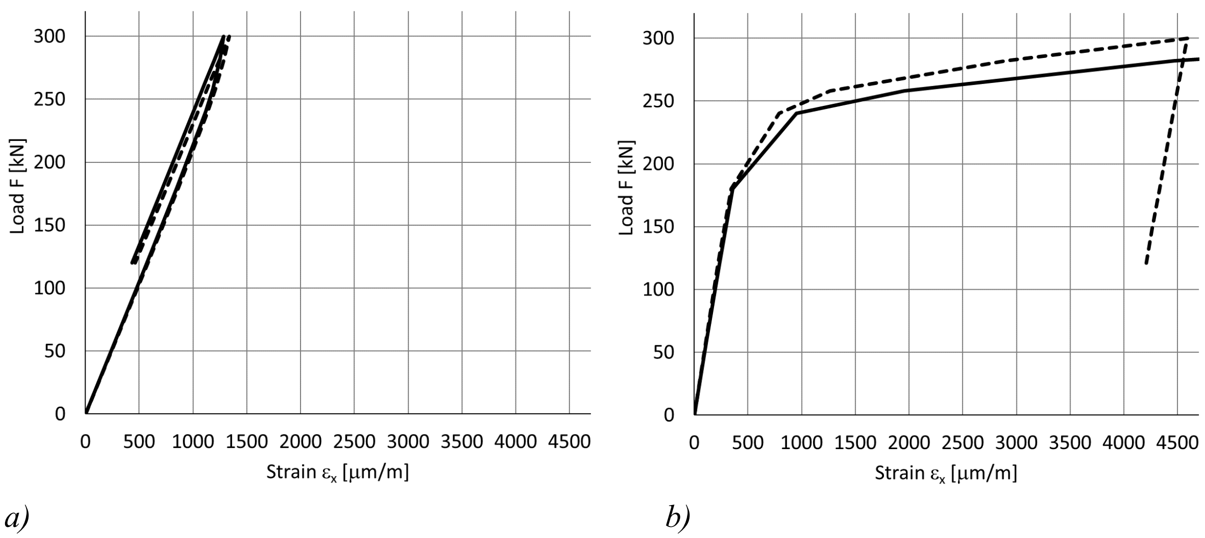


Fig. 10: Calculation results for the strain parallel to beam axis ε_x at depth co-ordinate y = 35mm (steel profile) continuous line: sliding contact between steel and timber dashed line: ideally stiff/glued connection between steel and timber
a) Results directly at the U-profile (outside mid-span)
b) Results at the connection splice plate (directly at mid-span)

Fig. 11 shows the calculated distributions of the normal stress component σ_x parallel to the beam axis as a function of cross-sectional depth co-ordinate y. With respect to length (x-) direction the location of the path for the stress evaluation has been chosen in the zone of constant bending moment, but shifted from mid-span, thus not touching the splice plate. With respect to width (z-) direction the path is located in the LVL layer of the wooden web.

As the results did not differ significantly with respect to the contact assumptions of "case 1" or "case 2", only the results of "case 1" are given here. The continuous line represents the stress distribution at 200 kN and the dashed line the stress distribution for the maximum load of 301 kN.

From the calculation results it can be seen, that at maximum load the assumed elevated yield stress of 450 N/mm^2 has just been reached for the outer face of the steel profile. The bending stresses of the wooden web were far below the characteristic strength values even at maximum load. Further, the stresses at the compression edge did not reach the strength of the special polymer concrete material. Thus, the expected failure behaviour – not observed experimentally due to the early stop of the test – would be a ductile tensile failure in the steel lamella. Consequently, the bending capacity of the beam could obviously be increased significantly by increasing the thickness of the steel profile.

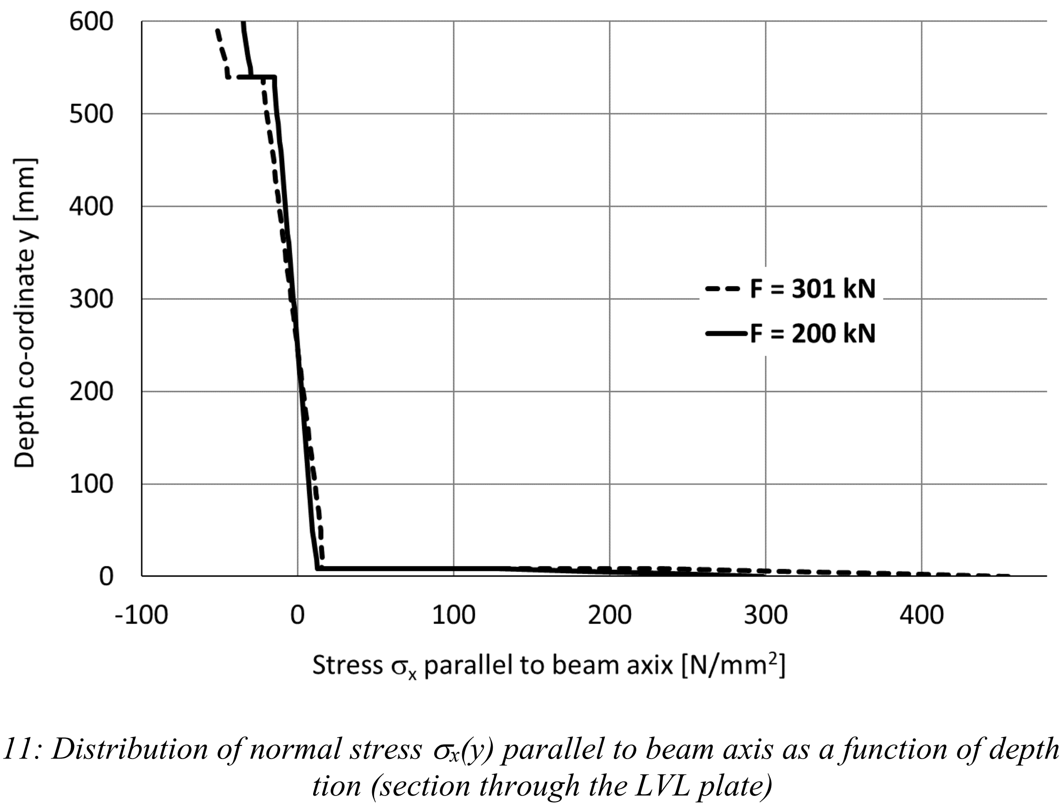


Fig. 11: Distribution of normal stress $\sigma_x(y)$ parallel to beam axis as a function of depth direction (section through the LVL plate)

When relating the achieved maximum bending capacity M_{ult} = 527.5 kNm with a fictitious homogeneous beam of the same gross cross-sectional dimensions, the achieved bending stress is $f_{m,hom}$ = 48.4 N/mm^2, being more than two times larger than that of the standard glulam strength class GL24. In order to judge this result, two aspects have to be considered: On one hand, the full bending strength capacity of the beam has not been yet been determined, as the test was stopped before damage occurred. On the other hand, the obtained single value of the hybrid beam is compared to the declared *characteristic* strength value of the strength class GL24 on a 5% quantile level. Thus, in order to determine the real bending capacity more experimental and/or simulation results are needed.

With regard to stiffness the hybrid beam exhibits – for a fictitious homogeneous cross-section with the same dimensions – a global modulus of elasticity $E_{g,\,hom} = 27360\ N/mm^2$ being 2.4 times higher than the tabled value of $11500\ N/mm^2$ for GL24.

8. CONCLUSIONS AND OUTLOOK

A hybrid beam with a wooden web of more than 85% of the material volume has been developed and a prototype beam was produced by industrial partners of the "ZIM" research project. In a first test the bending characteristics were determined with the full-scale prototype. Although the test has been stopped before any sign of damage, an increase of bending strength capacity of more than a factor of 2 could be determined. The stiffness increased by a factor of 2.4.

An analysis of the bending test by means of a 3D Finite-Element model showed, that the declared yield stress of the steel profile had already been reached. The simulation results have been verified by comparison with the respective empiric deflection and strain measurements during the bending test.

The bending test had been stopped before significant damage, in order to enable a second test aiming at the characterisation of the shear capacity. Thereto, the beam has been tested in a cantilever-type test set-up. The results of this test and the accompanying model calculations are published separately.

The next steps in the ongoing research project deal with an optimisation of the beam lay-up and especially with the simplification of the production process, which turned out to be technically feasible, but rather elaborate and thus would be very costly. It will be studied, to what extent the increased bending strength and stiffness as well as the shear capacity can be preserved also with a reduction of the number of bolts, the absence of an inclined orientation of the glued-in bolts and a screwed instead of welded connection of the bolts to the steel profile in the tensile zone.

9. ACKNOWLEDGEMENTS

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