ECCOMAS:
European Community
on Computational Methods
in Applied Sciences

ECCOMAS Thematic Conference

CompWood 2019

JUNE 17–19, 2019 | VÄXJÖ | SWEDEN

International Conference on
Computational Methods in Wood Mechanics – from Material Properties to Timber Structures

Book of Abstracts

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Preface / Foreword

Dear participants of CompWood 2019,

It is our great pleasure to cordially welcome you at Linnaeus University, on the occasion of the second version of the ECCOMAS Thematic Conference on Computational Methods in Wood Mechanics - from Material Properties to Timber Structures (CompWood 2019).

Växjö – the greenest city in Europe, is both a vision and ambition for the municipality, and building with wood is one major contribution to its sustainability goal. It is an exceptional and highly stimulating environment for all stakeholders in the process of building with wood. The region holds everything that is required for this ambition of a Wooden City, namely the raw material in local forests, forest owners and sawmills, the municipality with a strategic environmental work and a commitment to modern wooden constructions, in combination with strong local business partners developing new products and building systems up to contractors and developers dedicated to sustainable building. Linnaeus University has the central role of generating and disseminating fundamental and applied knowledge related to wood building technology together with the above-mentioned stakeholders in a holistic perspective. This is why we think that Linnaeus University is a predestined place for the second version of the ECCOMAS Thematic conference CompWood.

Building with wood and creating a sustainable built environment through competitive, resource and economically efficient wood-based products and building systems must be based on a strong scientific knowledgebase in order to succeed with this challenge in a long-term perspective. Increased understanding of the unique microstructure of the material, their components and their interaction, gained through the last decades can be exploited in new engineered wood-based products and building systems. Computational methods for the simulation of the mechanical behavior of wood play a key role, not only for an enhanced understanding and predictability, but also for the derivation of engineering design methods for a reliable and safe design of durable wooden structures. Wood is much more than a carbon dioxide storing material, it shows exceptional mechanical properties, not yet fully exploited in engineering applications. Moreover, the increasing competition for this raw material will require innovative methods to ensure efficient utilization of this natural resource.

The objective of the CompWood 2019 ECCOMAS thematic conference is to facilitate the progress in wood mechanics by bringing together scientists focusing on the micro- up to the structural scale. We want to contribute a platform for the dissemination of new methods and technologies. The goal is to present and discuss results of recent research activities, to exchange knowledge, and to discuss new paths for novel future research in order to extend our knowledge base. Computational methods, often in combination with experimental investigations, substantially contribute to explore the anisotropic, hygroscopic, and time dependent properties of wood and to exploit them in engineered wood-based products and structural applications, not limited to the built environment. This is why we aim to bridge length scales; and we are glad that we could attract a strong interest with more than 90 expected presentations, including numerical, experimental, theoretical as well as applied contributions. Five distinguished keynote lecturers will span over the above described conference topics and we are thankful to them for accepting the invitation.

The conference is jointly organized by Linnaeus University and TU Wien. We would like to acknowledge the support of ECCOMAS for providing the possibility to organize the CompWood conference under their auspices. Many thanks also go to the Scientific Advisory Committee for helping advertising the conference.

Finally, we would like to thank you for your contribution to the success of this conference. We hope you find the presentations and the discussions interesting and stimulating, that you have a wonderful time in Växjö and that you will leave the wooden city with a lot of great impressions and new ideas for your research. Enjoy your stay and welcome to Linnaeus University!

Thomas K. Bader
Josef Füssl
Anders Olsson
Chairmen of the 2nd CompWood ECCOMAS thematic conference CompWood

ISBN: 978-91-88898-64-7

Keynote lectures

ISBN: 978-91-88898-64-7

Keynote lecture: Utilizing Experiments and Numerical Models as basis for Structural Engineering

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Building with wood has gained much attention in the last decades mostly due to its favorable footprint with respect to the global heating problem. Wood has found its way into engineered components like glue-laminated beams, cross-laminated plates and veneer-laminated sheets and plates; with rapidly increasing popularity in the building industry. Moreover, these quite complex components also serve as parts in even more complex components like deck-, roof- and façade elements, most likely in composition with other materials.

Complex multiscale numerical models exists, capable to model also wood from maybe the molecular or micro level and all the way up to the up to scale of buildings structures. However, these models are currently not very applicable to the structural engineer due to missing documentation of a vast number of parameters as well as insufficient computational resources available. Another fact is that the structural engineer has no control of the wooden material actually to be used in a building structure; it might even not be harvested at the stage of the design of a building. The structural engineer is left with wooden materials, which have been characterized in certain classes with characteristic properties and statistical distributions. These characterizations, important for strength and stiffness, are based on certain measureable quantities like wave speed, density, deformation under trial load, or some optical methods.

Although wooden materials are classified and class properties are stated, these parameter sets are far from enough for more detailed numerical models of structural behavior e.g. in a connection between two wooden elements. The lack of proper numerical models for structural behavior of both components and connections in wooden structures, hampers the development of improved structural design. In the present keynote lecture, some of the recent research projects at NTNU addressing these topics are discussed.

The role of experiments has been, and is still important for wooden structures. Traditionally, series of experiments were performed to explore the effect of some parameters on strength or deformation, giving enough data to curve-fit the results to some simplified analytical expression, useful for engineering design. However, nowadays the role of experiments are more directed towards calibration or validation of a numerical modelling approach. The latter approach is much more generic in nature, and is more powerful, e.g. in order to develop improved structural detailing.

Basically, all structural FEM models must have data representing the materials, geometry, and internal and external boundary conditions, in order to compute the response to some modelled exposure. The geometry of a continuum is effectively represented by the finite element method. Boundary conditions must be paid sufficient attention as these seldom can be modelled by the basic degrees of freedom in the FEM models. Compared to other material, modelling of wooden material are challenging due to its complexity in any scale and the large variability due to uncertain growth conditions as well as the history of treatment after the harvesting. The natural growth of trees develop branches and results in knots and flaws, together with grain variations. Moreover, the physical behavior is dependent on temperature and moisture content, and the response shows also time dependency. Furthermore, the interaction of wood with other materials like steel needs special attention as too simplified approaches do not give accurate results.

The following topics, relevant for softwood, are addressed in the keynote lecture:

Material modelling of clear wood in meso/macro/component scale by anisotropic models. Effects of varying moisture content and resulting internal stresses. Effects of long term loading in interaction with varying moisture.

Contact properties between wooden parts and wood to metal parts. Threaded rods and screws behavior and modelling.

Localized stresses and fracture development, experiments and models.

Numerical modelling and updating of parameters.

Modelling and development of structural systems.

ISBN: 978-91-88898-64-7

Experimental characterization of material properties for numerical modelling of timber engineering applications

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The focus of this contribution is to give examples of new experimental setups developed at the Chair of Timber Structures and Building Construction (TUM) in order to characterize and quantify material properties which are subsequently included in numerical models of timber engineering applications. These examples mirror the scientific approach taken at the research group which is (1) to provide experimental evidence and (2) to interpolate and (very cautiously extrapolate) from these experimental results using numerical models, in order to receive a wider picture on the investigated topic.

The first example concerns the determination of the axial embedding stiffness k_{ax} of fully threaded self-tapping screws (STS) in order to predict their share in the transfer of relevant stresses when used as reinforcement. A STS is drilled through a timber specimen at desired angle between STS and grain. The STS traverses the timber specimen, projecting on both sides. A compression strain (deformation) is applied to the timber specimen while measuring the relative deformation of the STS which extends through two small holes in the steel plates used for load application. The embedding stiffness between the STS and the wood material is regressively determined with a numerical model [1]-[3].

The second example covers the determination of the axial strain of STS when used as reinforcement of e.g. holes in timber members. This is realized by drilling a hole from the screw head through the core of the screw and applying a glued-in strain gauge in the hole at the location where the highest strains in the screw are expected for the specific reinforcement application. Subsequently the screw is applied as reinforcement in a timber specimen. During the testing of the timber specimen, the strains in the STS are measured. Following this, these are compared to the strains determined by numerical modelling. The results are used to e.g. verify the applied embedding stiffness of the screw [3], [4].

The third example is related to the determination of the angle of stress distribution (stress dispersion) into cross laminated timber (CLT) elements under concentrated loads in plane. The upper flange and web of a specifically designed steel beam (I-section) are slotted into single elements, each featuring 50 mm horizontal length. Strain gauges are applied to both sides of the web-sections of each single element. The CLT element is placed vertically on the steel beam. A concentrated load is applied to the CLT-element in plane direction. During the test, the strains are measured in each steel element. The measured strains are converted into compression stress in the steel, correspondingly the force per unit length in the contact line between steel and CLT, and thus the distribution of stresses in the CLT-element can be estimated [5]. The results are used to e.g. verify numerical results of stress distribution angles for different CLT layups.

The fourth example concerns the continuous monitoring of timber moisture content (MC) and surrounding climate. At each location of measurement, four pairs of teflon-insulated electrodes with varying length are installed to enable the measurement of MC at clearly defined depths of the cross-section. The ram-in electrodes with partly teflon-insulated heads are connected to the moisture meter by custom built, shielded coaxial cables. The moisture meter records the resistance at up to eight channels. Each channel is actuated separately once per hour. Relative humidity and air temperature are recorded via a second data logger installed at the location of MC measurement. In addition to surface temperature, material temperature is recorded at two depths to allow for temperature compensation of the MC [6], [7]. The data (records of up to 3 consecutive years available) can be used to e.g. verify hygro-mechanical numerical models.

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 ISBN: 978-91-88898-64-7

Coupled heat and mass transfer in wood and wood-based products: macroscopic formulation, upscaling and multiscale modelling

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This conference is devoted to multiscale approaches of coupled transfer in lignocellulosic products, with particular attention to configurations leading to the failure of local thermodynamic equilibrium.

The macroscopic formulation of coupled heat and mass transfer in porous media will be presented first. Upscaling methods, such as homogenization or volume averaging, allowed a well-established set of macroscopic equations to be obtained [1]. This classical formulation of coupled heat and mass transfer in porous media assumes what is called the local thermodynamic equilibrium. This ensures important facts at the microscopic level such as a unique and uniform temperature for all phases of the porous medium or the equilibrium between the partial pressure of water and the bound water. Some simulations will be presented to highlight the importance of the anisotropy of wood or the strong heat and mass transfer coupling arising in Low Density Fibreboard.

This macroscopic formulation needs to be supplied by relevant effective parameters. Nowadays, thanks to the spectacular progresses in 3D imaging and High Performance Computing, these effective parameters can be predicted by 3D calculations on real pore morphologies. Recent examples will be presented for transfer properties of wood and bio-based building materials [2].

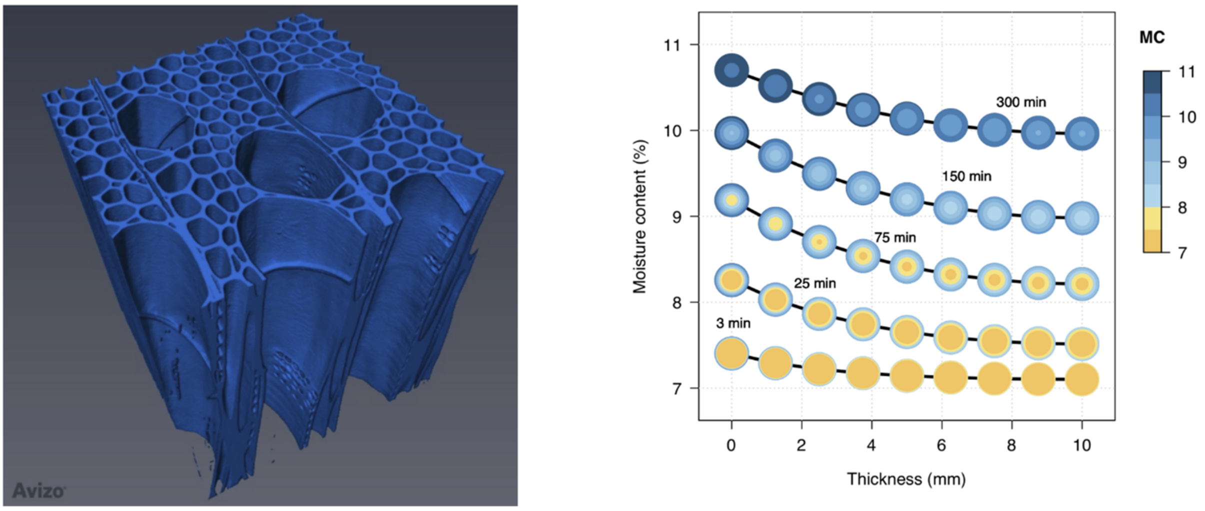

Figure 1: Nano-tomographic scan of poplar (left) and dual-scale modelling of sorption (right)

The remaining part of the conference will be devoted to configurations generating the absence of local thermodynamic equilibrium. Such situations, arising more often than expected, require a comprehensive multiscale approach. In one way or another, the local history of the product must be embedded in the formulation. Two approaches will be discussed here:

- the concept of distributed micromodels, with various assumptions regarding the coupling between scales[3,4],
- the transfer of all the information at the macroscopic level, in which the concept of internal variables allows the history of the microscopic field to be efficiently considered [5].

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ISBN: 978-91-88898-64-7

Cross laminated timber plates with a notch at the support

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The work presents different fracture mechanics approaches to model the crack propagation in notched cross laminated timber (crosslam) plates. The background, see [1], relates to the question of the applicability to crosslam plates of the Eurocode 5 (EC5) design equations for notched members, [1]. This involves the so-called Gustafsson approach [3], one of few design formulae in EC5 with a theoretical basis in fracture mechanics. Figure 1 below defines the basic geometry for a notched member.

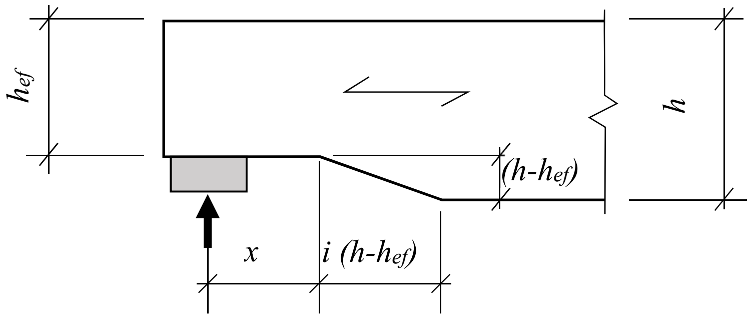

Figure 1: Geometry of a Beam Notched at the Support

In EC5, the shear force capacity of a notched member is given as a function of, *i.a.*, the material shear strength, although the underlying theory does not. This reformulation of the pure fracture mechanics approach of Gustafsson was done in order not to introduce new material parameters into EC5. The EC5 design equation for shear stress is written as:

$$\tau_d = \frac{1{,}5V}{b_{ef}\,h_{ef}} \leq k_v f_{v,d}\ ;\ k_v = \min\left\{\begin{array}{c} 1 \\ \dfrac{k_n\left(1+\frac{1{,}1\,i^{1{,}5}}{\sqrt{h}}\right)}{\sqrt{h}\left(\sqrt{\alpha(1-\alpha)}+0{,}8\frac{x}{h}\sqrt{\frac{1}{\alpha}-\alpha^2}\right)} \end{array}\right. \tag{1}$$

where b_{ef} is the effective width, $f_{v,d}$ is the design shear strength and where k_n is a material (calibration) parameter, $\alpha=h_{ef}/h$ and h, h_{ef} and x are defined in Figure . According to EC5, the material (calibration) parameter should be as follows: k_n = 4.5, 5 and 6.5 for LVL, structural timber and glued laminated timber, respectively. Although not explicitly stated in EC5, the parameter k_n represents the relation between the material parameters $G_{f,I}$ (the Mode I fracture energy), G (the shear modulus), and $f_{v,d}$, and where the factor 0.8 represents the relation between G and E (the modulus of elasticity). Thus the shear capacity is (implicitly) a function of *only* geometry, material stiffness and fracture energy and *not* strength (as expected for a linear elastic fracture mechanics theory).

The presentation discusses the design of notched crosslam plates from a theoretical point of view, including current design approaches as given in European Technical Assessments or Design Handbooks, see *e.g.* [4] and [5]. Those design approaches involve different, more or less straightforward, applications of Eq. 1 although several of the basic assumptions of that equation are not applicable for crosslam plates with a notch. Also, results from finite element analyses, using different theoretical frameworks to model crack propagation, are compared with the design approaches found in [4] and [5] and with experimental results from [6].

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ISBN: 978-91-88898-64-7

Numerical Optimization of Glued Laminated Timber with Mixed Species

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The integration of hard-wood species into structural timber engineering is driven by the development of structures with increased span-width, reduced cross-sections and local load introduction in connections. Mono-species structure either require large cross-sections to avoid excess loads in soft-wood or large volumes of hard-wood – rendering structures less economic. Structural members with mixed species, however, should be designed in a way that hard-wood is applied in the most efficient way to make use of its advantageous properties. Unfortunately, many structures that realize mixed specie concepts show damage in form of laminae cracking or adhesive bond-line delamination [1]. Due to strongly different hygro-mechanical behavior of soft- and hard-wood, considerably increased residual stresses occur, that can accumulate. Not only do those failures pose an aesthetic defect from the architectural point of view, delaminations, in particular at load introduction points, also raise durability safety issues, as they have the potential to grow with seasonal moisture cycles even after decades and should thus be taken into account in design considerations.

The study is motivated by the moisture-induced damage in the “ETH House of Natural Resources” a pilot building erected in Zurich with glulam members of spruce combined with ash with cross-sections of 36x36cm. Since the consequences of moisture induced stresses can only be studied on full scale samples, we perform a combined experimental and numerical study of moisture induced damage evolution in structural sized, instrumented glulam members out of spruce (*Picea abies*) and/or ash (*Fraxinus excelsior*) with varied lamellae thickness and stacking topology at different climatic conditions. Resulting moisture and stress fields are calculated and confronted with experimental observations, before the stress evolution under cyclic loading is predicted, using a hygro-mechanical model for wood. For meaningful simulations of wood components at high stress horizons and changing climatic conditions, the effect of moisture on all mechanical properties needs to be considered, determining the various contributions to the deformation like the elastic response, swelling/shrinking, plastic, viscoelastic, and mechano-sorptive behavior, as well as moisture transport [2].

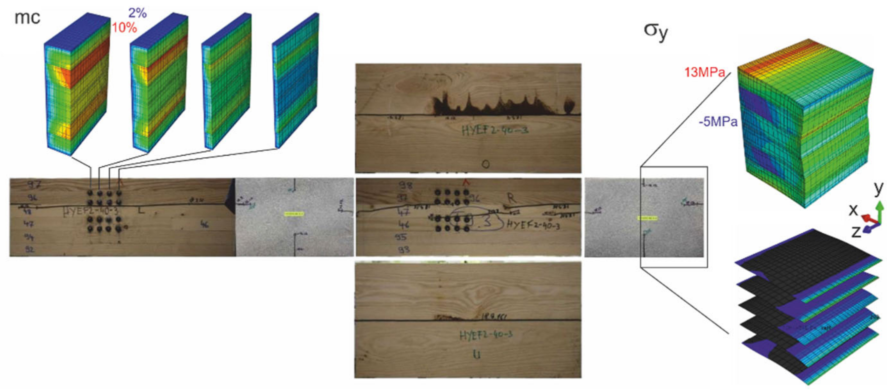

Figure 1: Exemplary ash/spruce sample with calculated moisture and stress fields for low humidity.

The model allows for the simulation of seasonal moisture cycles to assess the risk of damage for different configurations. We want to find the optimal configuration of ash/spruce lamella with respect to a required bending modulus and minimal risk of delamination under cyclic moisture-induced stress buildup. Since a single run can take up to several hours, gradient-based optimization methods are unfeasible. We demonstrate that surrogate models can efficiently be employed to find an optimal topology of a multi-species glulam beam with respect to multi-criteria objective functions. The method described reaches beyond todays stress based design and can be applied to systematically reduce moisture induced stress buildup, increase dimensional stability and hence durability of timber structures.

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Computational methods for connections and structures

ISBN: 978-91-88898-64-7

Wood crushing modelling for timber joint engineer problems

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Numerical modelling of wood failure in very localized areas which happens very often in timber structure joints, remains complex. Locally, the properties of wood can vary very highly due to natural growth, which make the mechanical problem deviate from continuous mechanical assumptions. Another problem appears when the stress reaches the material limits, leading to microcracking as well as densification in these areas. The definition of the densification paths are then difficult to predict because of the densification and the difficulty to model due to numerous numbers of parameters to be identify.

In order to produce a simple and reliable predictive modelling approach for engineers, it is chosen to idealize wood as a structure [1] that is composed of an isotropic foam base, reinforced by beams in order to bring the orthotropic nature of the material [2] [3]. In this approach, the plastic flow of the structure is modelled by the collapse of the foam-beam structure. This approach makes it possible to maintain elementary behaviours in the two components which are the beams and the foam. The identification of the most parts of the parameters could be obtained with confined compression tests.

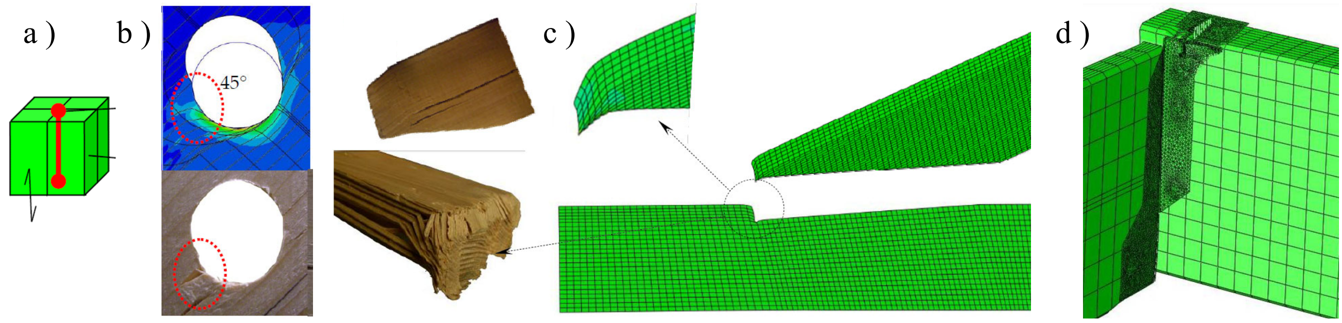

Figure 1: (a) Elementary cubic foam beam elements (b) Embedment test (c) Timber-to-timber joint (d) Folded metal anchors modelling.

After the realization of mesh adapted according to the type of problem, it is possible to approach several problems in a very promising way [4] such as the embedment of circular dowel according to the grain directions, the behaviour of joints with timber-to-timber contact (carpentry joints), the metal folded anchor type assembly behaviour fixed with nails or screws.

In order to perfect the modelling, it is then important to continue the research to master the predominant shear and transverse compression interaction on many aspects in the phenomenology encountered.

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ISBN: 978-91-88898-64-7

Numerical modeling of dowel-type connections in soft- and hardwoods including the rope effect

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In laterally loaded dowel-type connections, forces are not only transferred perpendicular to the fastener axis via contact forces (so-called embedment forces) and shear forces in the fastener, but also by means of forces evoked by displacement along the fastener axis and frictional forces within shear planes. The latter load transfer, the so-called *rope effect*, is often neglected or simplified in numerical models for laterally loaded connections, but considerably increases strength in case of large bending deformations of the fastener. In partially threaded screws, the rope effect is a result of the withdrawal behavior of the threaded part in combination with the axial resistance of the head of the fasteners. The tensile force along the axis of the fastener causes compression between connected members and frictional forces within the shear planes of the connection, which increase the load bearing capacity [1]. Consideration of the rope effect in numerical models is decisive for valid and suitable prediction of the load-deformation behavior and discussed in this presentation.

Different kind of numerical models, with different levels of complexity have been proposed for the simulation of dowel-type connections [2]. Herein, calculations by means of the beam-on-nonlinear foundation (BOF) method will be presented (see Figure 1). Compared to conventional foundation models, interaction elements that account for increased lateral connection strength due to withdrawal strength and the rope effect of the connection are considered. The behavior was implemented by means of axial springs that encompass a withdrawal force-relative displacement relationship, similar to the lateral springs considering the embedment behavior. In addition, friction between the connected timber members was considered by a frictional force as a consequence of the force component perpendicular to the shear plane. An elasto-plastic material model for steel accounted for possible failure of the steel fastener in shear and/or bending in connections.

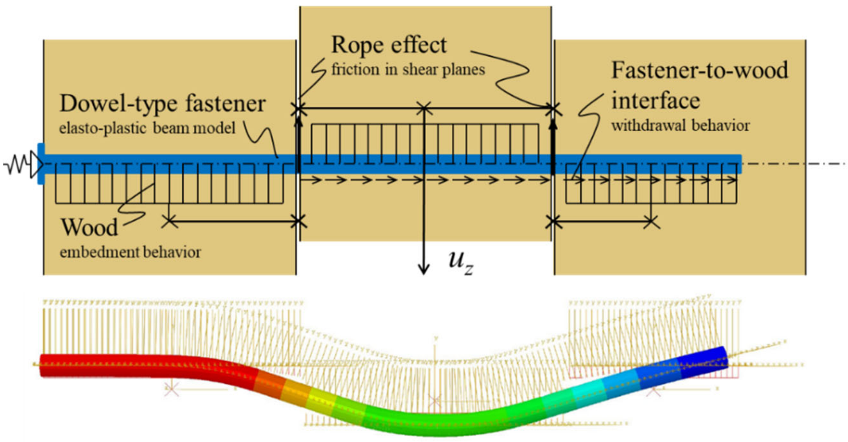

Figure 1: Beam-on-nonlinear foundation model for a partially threaded screw in a double-shear timber-to-timber connection in its undeformed and deformed state.

Simulations were carried out for connections in softwood and hardwood, applying different kind of dowel-type fasteners, including connectors with smooth shanks and screws. Model predictions were in good agreement with results from experimental studies. This raises confidence for application of the beam-on-foundation model for the engineering design of dowel-type connections and for reliable prediction of the structural behavior [3].

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 ISBN: 978-91-88898-64-7

Test-analyses comparisons of a stabilizing glulam truss for a tall building

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Nowadays, timber buildings are rising and their wind induced dynamical loads are thereby increasing. Tall and light buildings become sensitive to wind loads. Adding extra mass, damping or both constitute solutions [1]. In a recent study, the dynamic behavior of a large glulam truss with slotted-in steel plates and dowels connection types found from vibrational tests and finite element (FE) models were compared [2]. From forced vibration tests (FVTs) on the truss, used as a stabilizing element in a tall timber building, five significant eigenmodes, i.e. the natural frequencies, the damping values and the mode shapes with scaling, were estimated and compared to results from calibrated FE models, named A-models in Table 1. Two aspects for the modelling of a timber truss structure subjected to wind-induced loads were highlighted. First, the importance of including the finite stiffness in the connection instead of modeling the connection as being either pinned or clamped. Second, the accuracy of the Eurocode model aimed for estimates of the slip moduli for groups of fasteners in steel-to-timber connections. In this new study, Euler-Bernoulli beam element models have been developed by use of two FE codes: RFEM which is commonly used by structural engineers and the more advanced MSC NASTRAN. The geometry, density and stiffness properties of the glulam elements were set according to the production drawings, but the weight of the connections was ignored. Similar methods and analyses as the ones presented in [2] were performed to compare the natural frequencies and the mode shapes from the FE models with the experimental results. The RFEM and NASTRAN models had the same number of linear beam elements (1D).

Table 1: Experimental and analytical natural frequencies with MAC values comparing measured and FE eigenvectors.

Data set \ Mode nr.	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5
Experimental results	9.0 Hz	25.2 Hz	42.6 Hz	61.6 Hz	92.8 Hz
Calibrated FE model A1 (NASTRAN clamped-joints)	9.2 Hz MAC = 1.00	26.3 Hz MAC = 0.99	47.9 Hz MAC = 0.98	70.0 Hz MAC = 0.93	105.9 Hz MAC = 0.76
Calibrated FE model A2 (NASTRAN pinned-joints)	9.1 Hz MAC = 1.00	22.6 Hz MAC = 0.99	50.3 Hz MAC = 0.97	75.2 Hz MAC = 0.57	92.6 Hz MAC = 0.76
Calibrated FE model A3 (NASTRAN spring-joints)	9.0 Hz MAC = 1.00	24.2 Hz MAC = 0.99	42.1 Hz MAC = 0.99	60.0 Hz MAC = 0.79	84.7 Hz MAC = 0.92
FE model B1 (RFEM clamped-joints)	9.8 Hz MAC = 0.99	37.9 Hz MAC = 0.97	51.7 Hz MAC = 0.81	84.2 Hz MAC = 0.53	107.1 Hz MAC = 0.69
FE model B2 (RFEM pinned-joints)	9.8 Hz MAC = 0.99	33.2 Hz MAC = 0.95	57.7 Hz MAC = 0.95	80.4 Hz MAC = 0.52	98.6 Hz MAC = 0.59
FE model B3 (RFEM spring-joints)	9.6 Hz MAC = 0.99	34.7 Hz MAC = 0.97	58.2 Hz MAC = 0.84	69.4 Hz MAC = 0.77	89.5 Hz MAC = 0.65
FE model C1 (NASTRAN clamped-joints)	9.8 Hz MAC = 0.99	37.9 Hz MAC = 0.97	51.7 Hz MAC = 0.80	84.2 Hz MAC = 0.53	107.1 Hz MAC = 0.70
FE model C2 (NASTRAN pinned-joints)	9.8 Hz MAC = 0.99	33.3 Hz MAC = 0.95	57.7 Hz MAC = 0.95	80.3 Hz MAC = 0.52	98.6 Hz MAC = 0.59
FE model C3 (NASTRAN spring-joints)	9.6 Hz MAC = 0.99	34.8 Hz MAC = 0.97	58.2 Hz MAC = 0.84	69.4 Hz MAC = 0.77	89.5 Hz MAC = 0.65

Table 1 presents the old and the new results. On one hand, the natural frequencies and mode shapes from the models named B, which were calculated using RFEM, and the models denoted C, which were calculated using MSC NASTRAN, are identical. On the other hand, the new models predict higher natural frequencies than the calibrated models. In the new numerical models, the glulam members are of quality GL30c according to EN 14080:2013 and they have a Young´s modulus of 13 GPa which is 18.5 % higher than for four of the tested beams ($E_{mean,test}$ = 11 GPa and $E_{CoV,test}$ = 1.9 %). The total weight of the modelled truss is 3 880 kg which is 9.4 % lower than the tested one. Natural frequencies of vibrating structures are proportional to the square root of the modal stiffness to modal mass ratios. The deviations in mass and stiffness explain the numerical models' higher natural frequencies. Finally, the rotational and translational springs in the model 3 seem to be valuable parameters when evaluating natural frequencies. Using correct masses and stiffnesses for both structural elements and fasteners in FE-models is crucial to mimic the real structure's behavior.

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ISBN: 978-91-88898-64-7

Numerical and experimental study on light-frame test-modules for modular-based timber structures

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Today Sweden has a significant housing shortage. The authority "National Board of Housing" has stated that 700,000 flats need to be built by 2025. Building with prefabricated light-frame volume modules is an existing and innovative construction method for low and mid-rise timber buildings. Compared to traditionally site-built constructions this method is very advantageous due to its high prefabrication level and the fast on-site assembly of the modules. The volume modules are also manufactured in a well-controlled factory climate, where the wood material can retain its good quality. This work presents results from two ongoing research projects concerning numerical and experimental study of modular-based timber buildings. The final aim of this work is to develop an efficient three dimensional finite-element model to analyse both the global and detailed structural behaviour of these types of buildings.

To study the overall shear stiffness of the volume modules, eight different test-modules are to be tested. The main focus is to study (and optimise) the global shear stiffness of the test-modules. In addition, the shear stiffness of the mechanical (or friction based) connections between the modules will be tested. Regarding structural safety, connection design is an important task that needs to be numerically studied and experimentally verified. The test results will be used to calibrate the numerical model, see [1]. Figure 1 shows some results from the experimental study and the simulation.

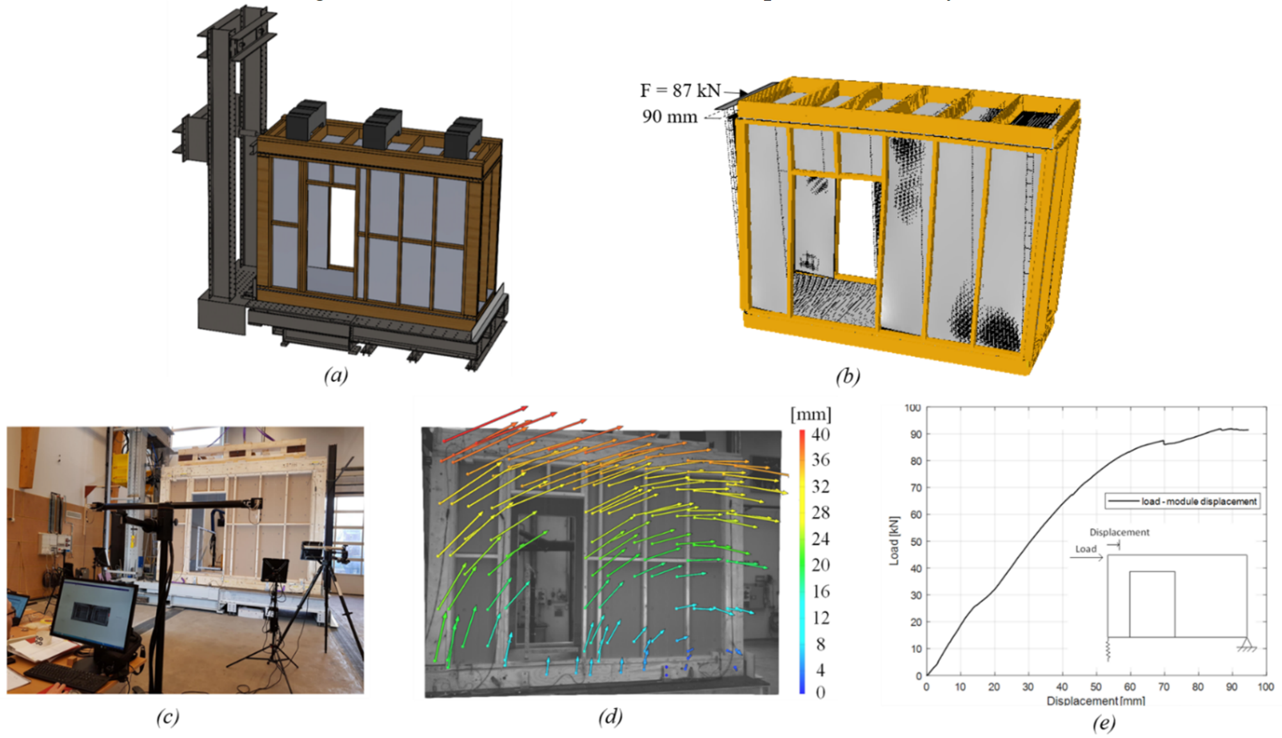

Figure1: Experimental and simulation results, *(a)* 3D drawing of the experimental facility used, *(b)* simulated deformations of the test-module studied, *(c)* the test setup with the DIC system and the potentiometers, *(d)* the displacement vectors from the DIC system, *(e)* the global load-displacement curve for the test-volume.

The experimental result for the first module showed the module to be both stiffer and stronger than expected. The mechanical connections worked very well and the global load-displacement curve in Figure 1*(e)* shows linear variation up to a high load level. The different test results were used to validate the simulated deformations of the test-module shown in Figure 1*(b)*. The experimentally-based stiffness values found for the mechanical connections shows clearly the importance of the different connection stiffness on the global module behaviour.

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 ISBN: 978-91-88898-64-7

Approximation of stresses in multi-span CLT beams based on refined zigzag theory

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In the recent years Cross Laminated Timber (CLT) has become an indispensable engineered wood product, especially within Europe. Due to its shear-flexible transverse layers, the calculation method has to capture these effects. Most commonly used approximation methods in practice are the Timoshenko beam theory, the gamma-method and the shear analogy method. Each method has its advantages and shortcomings. A comprehensive study on uniformly loaded multi-span beams using the mentioned methods was conducted in [1]. It was shown, that at the middle supports (local force introduction) the normal stresses (Timoshenko and gamma method) deviate from the reference solution (2D FE solution). The shear analogy method was able to capture these local effects better than the other employed methods, but with a higher modelling effort (two coupled beam elements). Higher beam theories are a possibility to reduce the modelling and discretization effort and reach a suitable approximation of normal stresses within CLT at middle support areas.

This contribution deals with the application of the Refined Zigzag Theory (RZT) for calculating stresses in CLT elements under local load introduction (eg middle supports in multi-span beams). RZT is a robust displacement theory, well suited for beam and plate finite elements, that has been developed recently [2]. Refined zigzag theory makes use of Timoshenko as its baseline and a product of the zigzag function $\psi^{(k)}(z)$ and the amplitude of the zigzag displacement $\phi(x)$ is added. The zigzag function is predefined through the thickness and the shear modulus of the lamellas. Adding one additional degree of freedom in case of an 1D beam.

Within this contribution an analytic solution of RZT [3], as well as RZT-FE-beam elements [4], are employed to calculate normal and shear stresses of different multi-span CLT beams. The results are compared with a 2d-FE solution and the mostly used methods in practice.

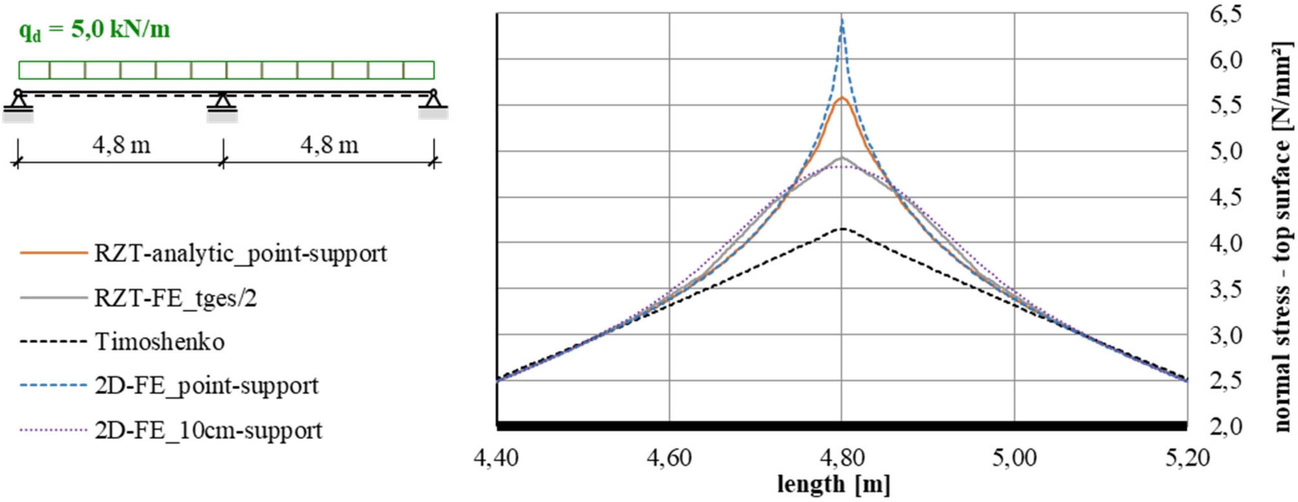

Figure 1: normal stresses on top surface of 5-layer CLT element at the middle support

Results show, that the refined zigzag theory is able to approximate the stresses within CLT elements at middle supports. An additional advantage of RZT is, that no shear correction factors are needed to calculate the deflections. Furthermore, the implementation of the employed linear FE-beam-elements based on RZT are presented and discussed.

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ISBN: 978-91-88898-64-7

Numerical study on de elastic buckling of CLT Walls subjected to compressive loads

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The use of Cross Laminated Timber (CLT) as a building material has been growing for the last 20 years. This impulse has been driven by the lower environmental impact of timber construction compared to concrete and steel, good structural and thermal performance and cost competitiveness [1]. Moreover, in recent years the use of CLT in mid-rise buildings has been consistently increasing, with examples present in Austria, Canada and the UK among other countries. This surge in the use of CLT as a construction material has fuelled research in CLT structural mechanics, leading to deeper understanding of its structural response [2-4]. Despite the considerable progress made on the study of CLT structural response, little have been done to fully understand their buckling behaviour.

In this work, a numerical-experimental approach is used to study the elastic buckling of CLT panels [5]. First, a finite element-based multi-scale model is developed to study the linear elastic buckling behaviour of CLT panels [4,5]. The model incorporates wood's most relevant microstructural features, such as the volume fraction of hemicellulose, lignin and cellulose, their mechanical and physical properties, microfibril angle, etc.; which are crucial to capture the inherent orthotropic nature of wood observed at the macroscopic level. Furthermore, the values of key microstructural parameters are determined through a parameter identification procedure, in which experimentally measured values of density and longitudinal Young's modulus of radiata pine grown in Chile are used as target values. The model is successfully validated with results from buckling tests performed on CLT panels' specimens with different thickness and slenderness ratios. The validation clearly shows the capability of the model to predict the buckling response of CLT panels (Figure 1). Finally, the model is used illustrate how parameters such as wood density and panel number of layers influence the buckling response of CLT panels.

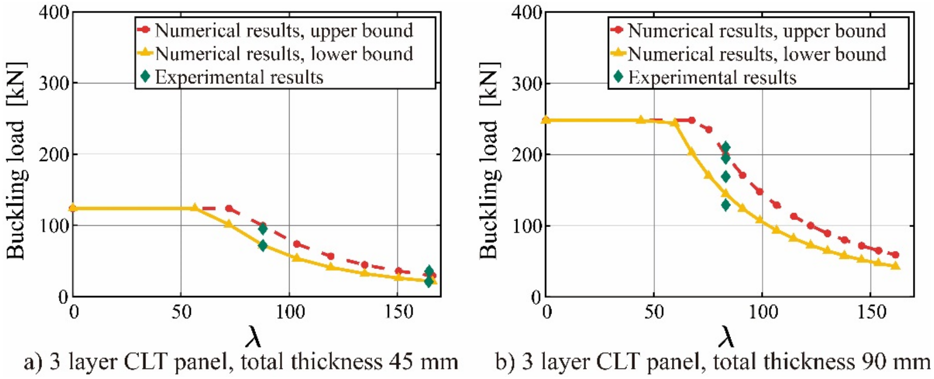

Figure 1: Numerical predictions vs experimental results: a) CLT panel with 45 mm total thickness, b) CLT panel with 90 mm total thickness [5].

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 ISBN: 978-91-88898-64-7

Computational and experimental methods for wood materials

ISBN: 978-91-88898-64-7

Computational Wood Mechanics using the Material Point Method

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The material point method (MPM) [1] has developed over the past 25 years into a robust tool for computational mechanics. It has numerical features that recommend it for modeling of wood. For example, it can model complex, realistic structures, deal with realistic constitutive laws, model cracks and contact, and model wood-adhesive interactions. This talk will cover some recent developments in MPM and their application to problems in wood products.

Because MPM is a particle method, it is relatively straight forward to translate 3D X-Ray CT data directly into a realistic model for wood by converting voxels in the X-Ray data into material points in the computer model. A past project at Oregon State University [2] recorded X-Ray CT data for wood-adhesive bonds. The adhesives were tagged to allow segmentation of wood cell wall from adhesive thereby providing 3D data for adhesive penetration into wood cells. The previous project used MPM to model stresses and strains in those structures [2]. We have now also modeled penetration of the adhesive into wood cells using fluid mechanics. In brief, we started with X-Ray data, removed the adhesive, separated the two wood adherends, and restored the adhesive as a slab between the adherends. This entire structure was them compressed in an MPM model. The MPM predictions of penetration were compared to experimental results and showed good agreement. Note that the X-Ray data provided both a realistic structure for input to the model and experimental observations to validate the modeling. Figure 1 shows the initial MPM model (1A) and final state for the MPM model (1B and 1C; the top half is remove in 1B to see adhesive penetration). The modeling was for penetration of phenol formaldehyde (PF) resin into hybrid poplar [3].

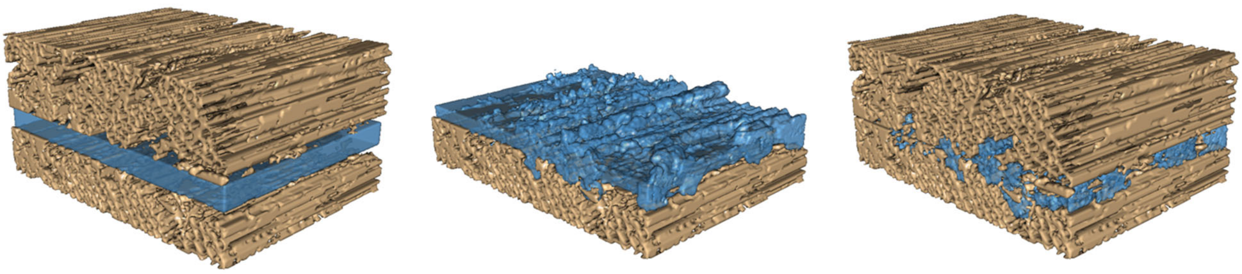

Figure 1: A. Initial model B Final result (top removed) C. Final result (full)

In addition to modeling adhesive penetration, MPM can model subsequent failure of the wood-adhesive bond using newly developed methods for anisotropic damage mechanics of an initially anisotropic material [4]. The damage mechanics methods have applications in many other wood failure problems such as modeling the tendency of cross-laminated timber (CLT) to develop cracks in all layers after relatively minor changes in moisture content [5]. Other wood applications of MPM include modeling of cutting [6] and failure of wood-working joints.

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 ISBN: 978-91-88898-64-7

Phase field method-based modeling of fracture in wood

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Wood, as a naturally grown material, exhibits an inhomogeneous material structure as well as a quite complex material behavior. For these reasons, the mechanical modelling of fracture processes in wood is a challenging task and requires a careful selection of numerical methods. Promising approaches like limit analysis [1] or the extended finite element method (XFEM) in combination with microstructure materials models [2] deliver good but not yet satisfying results. Particularly the latter approach, including XFEM, has severe difficulties with crack paths in regions with complex morphology, mainly around knots. Therefore, in this work, focus is laid on the recently emerging and very popular phase field method [3]. Especially geometric compatibility issues that limit the use of XFEM can be avoided, as the crack is not discretely modeled but smeared over multiple elements. This allows the formation of complex crack patterns, defined by the underlying differential equations and boundary conditions but not restricted by the mesh geometry.

The present implementation contains a micro-mechanical model-based tsai-wu failure criterion [2] and allows considering orthotropic material behavior through a structural tensor which scales the phase field's length scale parameter [4]. For solving the system of differential equations, a staggered approach is used where the phase field equation and deformation problem are solved separately. The staggered approach is enhanced with an additional Newton-Raphson loop that ensures convergence [5].

The developed algorithm was tested on various problems. Compared to XFEM more computation time was needed as the phase field method requires a finer discretization. However, crack patterns, including branching and merging, could be modeled very stable and accurately, even in the vicinity of knots where the material structure of wood is particularly complex and interface zones exist (see Figure 1).

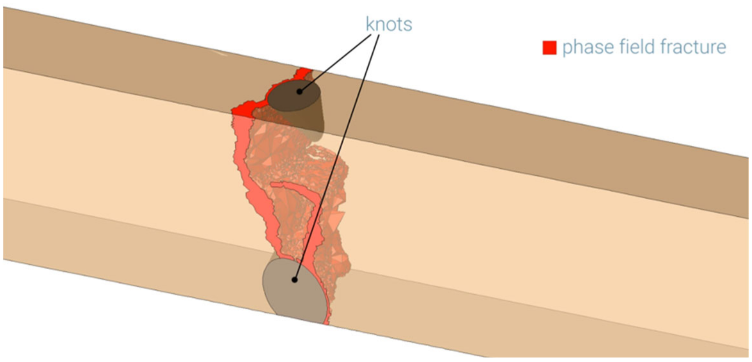

Figure 1: Cracking between two knots under uniaxial tension modeled with phase field approach

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ISBN: 978-91-88898-64-7

Experimental study on the creep response of Chilean Radiata Pine wood

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Wood has long been an accessible material, at hand for different type of applications. It has recently undergone a renaissance thanks to exceptional properties. For instance, as a renewable material source, it has a small environmental impact, a favorable seismic performance and it can self-protect against fire. In Chile, Radiata pine has been promoted as a construction material to take advantage of its wide availability. In 2017, near the 60% of the total wood exports were of this type of pine. Despite the increasing interest on the material, experimental mechanical properties are scarcely available or not very well known [1]. Macroscopically, a common way to characterize the material is the bending test. Nevertheless, they are often performed at ambient conditions, with humidity and temperature kept constant.

For this reason, the scope if this work is to study the relaxation phenomenon in the Radiata pine under flexion i.e., the decrease of the tension or load over time while keeping a constant deformation rate. The purpose of this research is to determine which factor has a greater influence on the relaxation of the bending behavior. An ambient chamber was designed so to keep the temperature and relative humidity in the inside within a defined range. Bending relaxation tests were carried out on wood specimens inside the chamber. The proposed methodology leads to satisfactory results. It is shown that both the displacement and the humidity play an important role on the relaxation behavior. Temperature, on the other hand, has a negligible effect on the relaxation. Recommendations are indicated for the improvement of the environment chamber.

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 ISBN: 978-91-88898-64-7

Improvement of ductility and toughness of wood polypropylene-composites

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Given the wide range applications, wood-polymer composites (WPC) are occupying an undeniable position in the industrial composite market. However, WPCs suffer from low ductility and weak toughness even when only small fiber proportions are used. Fibers in thermoplastic polymers promote the propagation of cracks as well as the fracture of beams. In addition, wood beam assembly and fixation by screw or nail could become difficult due to the limited deformation behavior. Thus, the main objective of this study was to assess the enhancement of toughness and ductility of wood-polypropylene composites. The specific objectives were: (i) to establish a comparative analysis on the ductile and tenacious behavior of WPC according to the type of fiber and (ii) to investigate the effect of the addition of glycerol and elastomer on the mechanical behavior. WPCs were produced through two types of fiber (kraft and white birch), polypropylene (PP), maleated styrene-ethylene/butylene-styrene (SEBS-MA) and maleated polypropylene (MAPP). The composites were manufacturing using a two-step process, pellets extrusion using a twin-screw extruder and injection of test samples using an injection-molding machine. WPCs (Table 1) were characterized by mechanical (tensile, three point bending and impact) and thermal (differential scanning calorimetry and thermogravimetry) tests.

Table 1 : Composition of the various PP/WF composites

Composite code	Composite compositions for polymers (%-by weight), impact modifiers and compatibilizer					
	PP	White birch fiber	Kraft fiber	Kraft treated with glycerol	MAPP	SEBS-MA
1	67	30			3	
2	67		30		3	
3	67			30	3	
4	57		30		3	10
5	47		30		3	20
6	47			30	3	20

Results showed that the incorporation of wood fiber reduced the elongation at break of the PP from 600% to 12.9 % for PP/kraft fiber WPC and to 5.4% for PP/white birch fiber. Treated kraft fibers with glycerol solution reduced deformation and ductility of composites, the corresponding elongation at break was equal to 7.21%. Adding 10% and 20% of SEBS-MA to the WPC formulation increased the elongation at break by 29.3% and 61.8, respectively (Figure 1)%. Adding SEBS-MA to WPC made with glycerol treated kraft fibers led to a substantial improvement of elongation at break (95.1%) Similarly, SEBS-MA substantially improved the WPC impact strength. Adding 10% and 20 of this elastomer improved the impact strength by almost 100% and 380%, respectively (Figure 2). WPC made with glycerol treated kraft fibers showed a 685% improvement of the impact energy. The improvement of toughness and ductility is explained by the encapsulation of filler by the SEBS-MA [1-2].

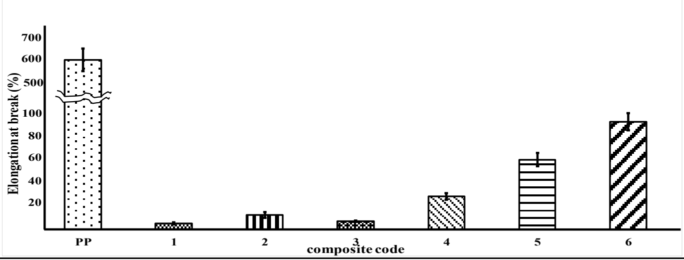

Figure 1 : Elongation at break for different composites

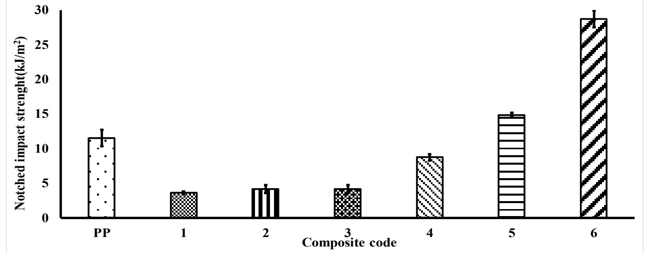

Figure 2 :Notched Impact strength for different composites

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A numerical and experimental methodology to investigate morphological changes in wood exposed to fire temperatures

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The response of wood under fire temperatures is strongly dependent on its material microstructure and chemical composition [1]. Due to its non-homogeneous porous structure and hygroscopicity, the hygro-thermal behaviour of wood is characterized by several coupled processes. For timber components of buildings the moisture content inside wood remains below the fiber saturation point and is constrained in the wood cell walls as bound water, while the water vapour is distributed in the lumens, rays and canals in addition to the other gaseous components. The two phases of water are coupled through sorption. Furthermore, the heat of sorption includes the latent heat of condensation/evaporation due to the change between the two water phases. Finally, in the presence of fire temperatures, wood undergoes pyrolysis, a phenomenon of thermal degradation. This is characterized by weight loss due to moisture and pyrolysis products leaving the material that can highly affect the morphology of wood microstructure [2]. In this context, numerical models can integrate the experimental tests to deeply understand the effects of these complex phenomena in order to develop better methods to enhance wood durability under fire temperatures. Most of the numerical models developed in the nineties for wood under high temperatures were mainly aimed to simulate wood drying processes. In the last decade, there was an increasing interest in developing hygro-thermal models including also pyrolysis (see references in [1]) but the topic is still challenging above the charring temperatures.

In the present paper, the responses of 0.5×0.5×1cm spruce samples heated in a furnace under helium flow are investigated. A heating rate of 5 K/min from room temperature until final temperatures in the range of 250-550^{0}C is used. To understand the effects of different charring temperatures on the material microstructure of wood, SEM pictures are taken from cuts made by a razorblade on perpendicular to grain wood sections (Figure 1, left). The morphological variations of vessels and wood cell layers are observed and discussed in relation to the corresponding increments of temperature and mass losses. These increments are evaluated by means of a multiphase hygro-thermal model including pyrolysis, as illustrated in Figure 1 (middle). Most relevant morphological changes are found for temperatures above 300 ^{0}C. In addition, to observe morphological changes for temperatures below 300 ^{0}C, microstructural finite element analyses (µFEM) [3] are carried out by taking into account thermal expansion and mass loss of cell walls (calculated with the hygro-thermal-pyrolysis model), and the decreasing values of elastic moduli measured by tensile tests. The µFEM analyses of representative microstructures (Figure 1, right) allow to identify the strain and stress distribution in wood cells before charring that could explain the reason of the successive morphological changes. The proposed methodology represents a further step for the future development of more complex hygro-thermo-chemo-mechanical models for wood exposed to fire.

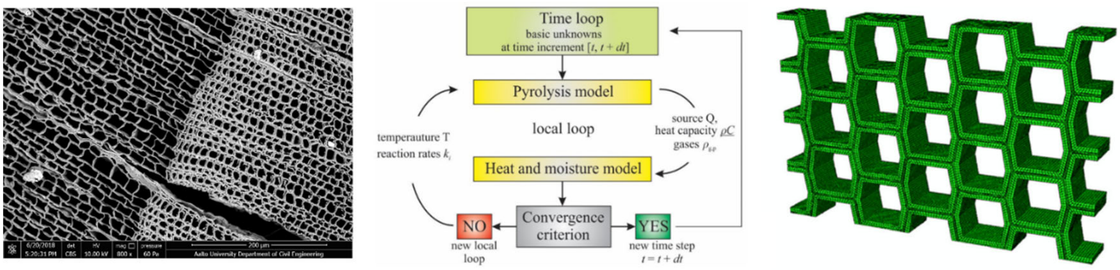

Figure 1: Left: SEM picture from wood sample heated until 400 ^{0}C. Middle: scheme of the proposed hygro-thermal-pyrolysis model. Right: meshed representative microstructure for the µFEM analysis.

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ISBN: 978-91-88898-64-7

Modeling of wood under combination of normal stresses with rolling shear stress

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Variation of material properties and strength of wood in its three main directions is large and mainly depends on the cellular structure of wood, the geometry of wood cells [1] and their orientation. The anisotropic behavior leads to several different failure mechanisms already under uniaxial stress. Multiaxial stress analysis becomes important when dealing with modeling and design of certain engineering details and the local material behavior. Research regarding material behavior of wood under stress combination, especially interaction of shear stress with normal stresses in the transverse plane has however attracted less attention. Even, the current Euro-code EC5 also lacks [2] of such a design model to account for the combination of stresses perpendicular to the grain with shear. The Swiss standard for the design of timber structure, SIA 265:2003 [3] however includes design criteria for stress perpendicular to the grain in combination with shear, which was developed for longitudinal shear. The work presented herein aims at defining a material model for wood under compression perpendicular to the grain with rolling shear interaction by means of finite element (FE) modeling. It considers geometrical nonlinearity, which makes the model suitable for analysis on the structural scale to have an in-depth insight of local stress distribution. The FE material model was compared with experiments by modeling a biaxial test setup and comparing numerically determined stresses with strain fields measured in experiments.

Experiments were realized in biaxial test frame with a setup, consisted of steel plate with mechanical grip to hold and connect the specimens with the test frame. Displacements in two perpendicular directions were prescribed for testing under different stress combinations. Experiments were carried out on two series of dog- bone shaped, Norway spruce specimens. In addition to force and displacement measured by the internal actuator of the biaxial test frame, a digital image correlation (DIC) technique (Aramis, GOM) was used to measure strain fields on one surface of the specimens by continuously capturing images during the experiments.

FE modeling was analyzed in Abaqus (SIMULIA™ by Dassault Systemes®) for compression, shear and different stress combinations of compression and shear stresses in radial-tangential plane. A 3-D model was considered with symmetry in one plane to reduce the number of elements and computational cost. Orthotropic material behavior was considered by defining elastic and plastic properties of wood in three principal directions. Material behavior was defined by means of a user-defined subroutine considering elastic and plastic material model with hardening and densification regions under compression orthogonal to the fiber direction.

The numerical model shows suitable correlation with experiments under uniaxial compressive and pure shear loading. Influence of annual ring orientations and differences in earlywood and latewood material response are observed in experiments in all loading cases. The biaxial experiments confirmed the positive influence [2] of the interaction of rolling shear with compressive stress in the failure of materials. However, in the FE model, the beneficial effect of shear interaction in case of combined loading is not reflected as well as in experiments. Consideration of one global hardening parameter to define the material behavior under compression and shear in different anatomical directions could be the reason. Consideration of annual rings, its curvature, and variations of material behavior i.e., earlywood and latewood region could improve the results in the biaxial case. However, consideration of different hardening parameters or fitting the failure criteria with experiments under such stress combination is an extremely demanding task.

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ISBN: 978-91-88898-64-7

Connections and timber structures

ISBN: 978-91-88898-64-7

Close-up strain measurement along the mechanical interface of self-tapping screws joined with timber by means of electronic speckle pattern interferometry

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Inserting self-tapping screws in timber is done for example to reinforce wood material, in order to guarantee the required mechanical strength of this common timber engineering application. In favor, to understand the interface behavior of the "timber-screw" system, it is of main importance to describe the stress-strain distribution in the vicinity of the screw. However, only few researchers have dealt so far with the investigation of the stress-strain distribution at the screw – wood interphase that relies on non-contact optical gauging techniques [1, 2] like for example electronic speckle pattern interferometry (ESPI). Therefore, in the present study the in-plane strain distribution around the joint area "timber-screw" was measured by means of ESPI. In the experiments solid spruce wood (*Picea abies* (L.) Karst.) and self-tapping screws with a slenderness of $\lambda \approx 11$ (l_{ef}/d) were used. Timber elements were cut into two pieces, clamped together and the screws were inserted in the cut surface. Before performing screw pull-out tests, one timber piece was removed, so that the interphase section "timber-screw" was observed directly. Based on the idealization of wood as orthotropic material with linear-elastic material properties, Hooke's law was applied to estimate the related in-plane stresses. The established maps implied where stress-strain concentrations can be expected, and further what timber material region is strained and stressed (cf. Fig. 1). Providing fundamental insight for future screw-joints developments and material simulation, is the aim of this investigation.

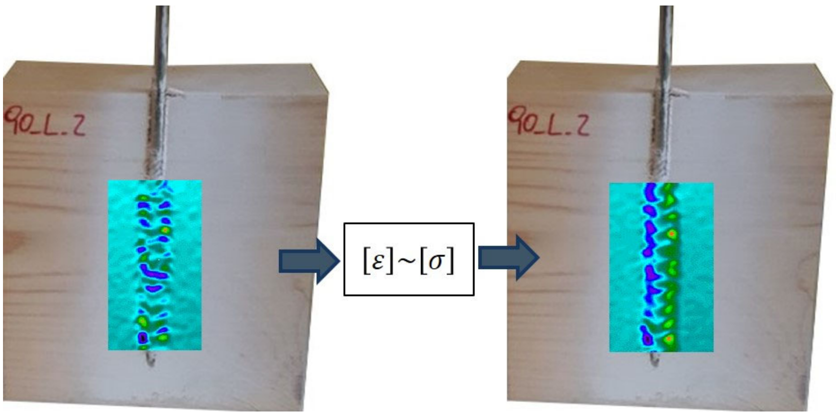

Figure 1: Illustration of strain (left) and stress map (right) of a tested specimen

Applying ESPI technique on the designed pull-out experiment, proved that full-field non-contact optical gauging techniques are suitable to establish stress-strain maps of a highly heterogenous, combined material region accurately [3]. Nevertheless, for more profound understanding and validation of the strain-stress behavior along the interface section of the "timber-screw" specimens finite-element modeling (FEM) is suggested. The aim of this investigation, however, which was generating basic knowledge considering the stress-strain mechanism between screw and timber has been fulfilled.

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Acknowledgements: this research was conducted within the project "WoodC.A.R: Wood – Computer Aided Research" which received funding by the Austrian Research Promotion Agency (Österreichische Forschungsförderungsgesellschaft, FFG) under the Project Number 861.421.

Reliability analyses using finite element models of trussed timber structures with dowelled connections

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Current design of timber structures is mostly based on element-by-element approaches, in which the reliability of the whole structure is unknown, but assumed not to be much smaller than the reliability of each member or connection. A recently started research project at ETH Zurich and Empa, in Switzerland, aims at investigating the global behaviour and reliability of medium and large-span trussed timber structures with connections with dowel type fasteners, taking into account the variability of mechanical properties and of the load-displacement behaviour of its connections. A multi-scale modelling approach will be followed, from the behaviour of a single fastener, to the load-deformation of a connection and behaviour of a complete structure, which will allow assessing the most important material and geometrical parameters at different levels.

Most of the past reliability studies in timber engineering were conducted with analytical models [1]. For such models FORM or SORM analyses have often been accurate enough. However, crude Monte-Carlo simulations (MCS), which require 10^5 to 10^8 simulations (depending on the order of magnitude of the probability of failure of interest), are also feasible with analytical models. The use of computationally-heavy non-linear finite element simulations is, in general, not viable due to the extremely long total running time. Using alternative methods, such as importance sampling or subset simulations, the number of simulations needed can be reduced by some orders of magnitude, depending on the boundary conditions. One of the latest developments in the field of reliability analyses for structural safety are adaptive Kriging MCS methods, which only require hundreds to thousands of model runs [2]. The increase in performance provided by these methods makes reliability analyses with computationally heavier FE-models more feasible, at least for research purposes.

For this investigation two complementary modelling frameworks are being developed: at the connection scale a semi-analytical method (as described by Schweigler et al [3]), based on a beam-on-springs models (similar to that of Lemaître et al [4]) is used for the reliability analyses; at the system level, a finite element approach with 1-D elements and non-linear springs representing the connections is used. The input for the models is based on probabilistic data described in JCSS [5] and Leijten et al. [6] and benchmarked against test data from Jorissen [7].

The proposed contribution will summarise previous studies on the use of simplified models for timber connections and structural reliability analyses of timber structures. It will focus on which analysis methods and modelling strategies are best suited for connections with dowel type fasteners, and trussed timber structures. The overall multi-scale modelling framework will be presented and discussed.

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ISBN: 978-91-88898-64-7

4D self-shaping mechanisms for achieving double-curved wooden structures

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The hygroscopic nature of wood results in anisotropic dimensional changes in function of ambient climate. Employing smart structural design, we explore how this inherent capacity of wood can be used beneficially for generating large deformations and self-shaping mechanisms. A commonly known set-up for generating structures with single curvature is the bilayer composite lay-up where temperature or moisture responsive materials bend or twist upon changes in ambient climate. The self-shaping of wooden bilayers was previously characterized [1,2] and is suitable for large scale building applications [3]. The next milestone towards complex-shaped timber structures by self-shaping is the generation of double curvature from an initial flat configuration. Change in Gaussian curvature, i.e. the product of two principal curvatures along the two axes of 2D surfaces in 3D space, however, is mathematically impossible. Physically, however, this is possible by using a volume changing material such as wood and by designing adequate structures.

Here, we present different self-shaping mechanisms, which involve a 4D manufacturing approach, to generate double-curved wooden structures starting from an initial flat shape. For each mechanism, the optimal design of the structure is found by parametric studies on numerical models using the Finite Element Method. The material wood is hereby represented by a complex rheological constitutive material model featuring specific deformation mechanisms such as elasto-plasticity, visco-elsaticity, mechano-sorption, and hygro-expansion [4]. The different structure-specific self-shaping mechanisms include the arrangement of bilayer strips to a grid-shell configuration (Fig. 1A), a controlled buckling by anisotropic growth (Fig. 1B), and a structure composed of densified wooden wedges that display set-recovery when wetted (Fig. 1C).

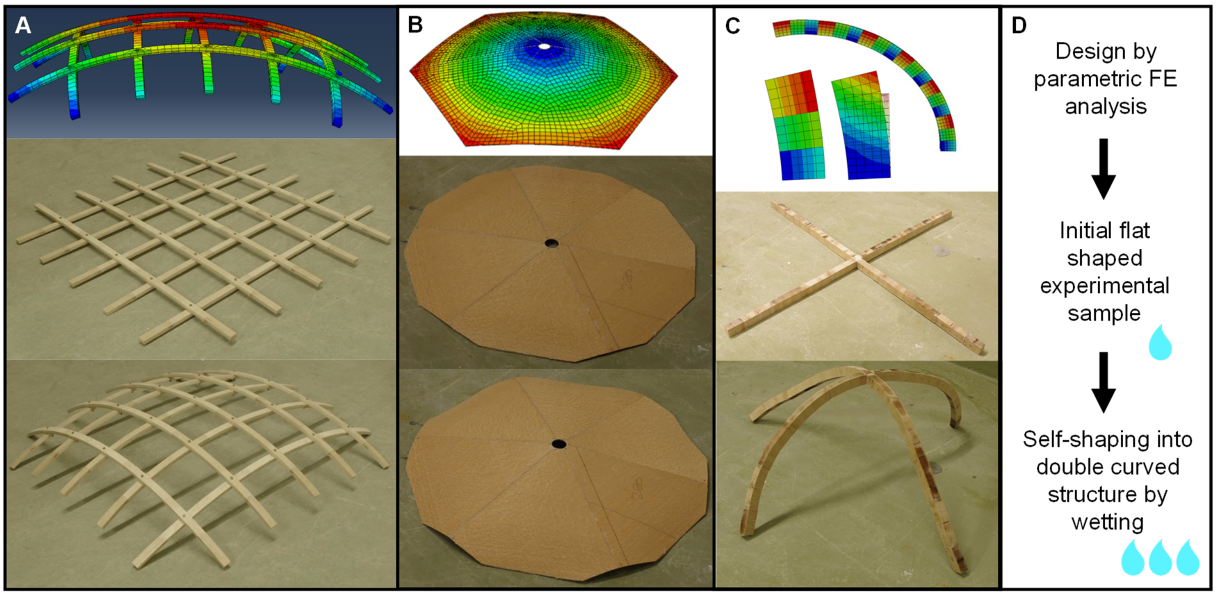

Figure 1: Self-shaping mechanisms for double-curved structures. (A) Bilayer grid-shell. (B) Anisotropic growth buckling. (C) Set-recovery of densified wood wedges. (D) 4D approach.

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ISBN: 978-91-88898-64-7

A New Macro Modeling Approach in Structural Analysis of Integrally-Attached Timber Plate Structures

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The recent advancements in the robotic fabrication of engineered timber products are used to re-consider the oldest known method of wood-wood joinery, apply it in modern architecture, and provide an integrated design framework in free-form spatial timber plate structures (Figure 1) [1]. The structures are adaptable to a wide range of large-scaled 3D forms; nevertheless, there have been few systematic investigations of their mechanical characteristics.

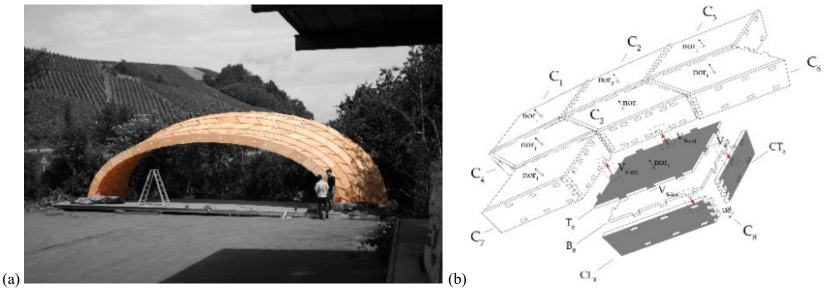

Figure 1: Integrally-Attached Timber Plate Structures; (a) Prototype, (b) Assembly Logic

Providing an efficient and practical-oriented mechanical models seems inevitable. In light of this, through avoiding plasticity governed shell and solid meshes, a novel modeling approach is proposed, where series of beam-column elements are used. This approach, which is referred to as the “macro models” remarkably enhances the efficiency of structural computations. Burton et al. [2] provided the application of such model in timber frames. Through the kinematic realization (Figure 2a-b), the macro models versus mesh-based shell FE models are shown in Figure 2c.

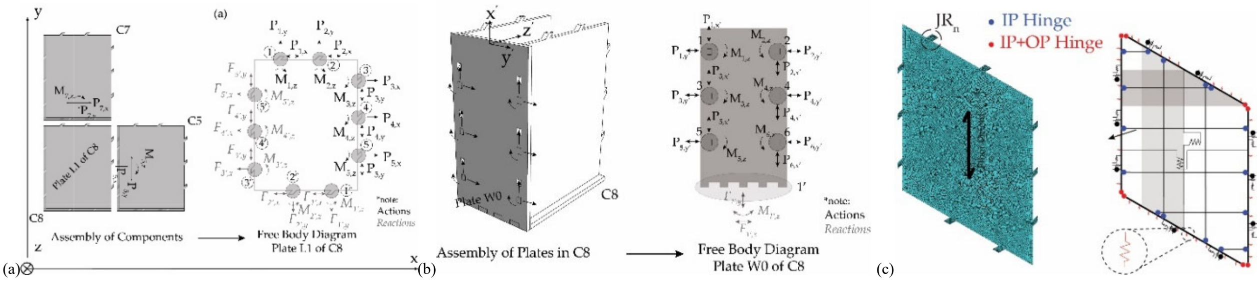

Figure 2: Free Body Diagram, Macro vs. FE models

The performance of an integrally-attached timber plate under In-Plane (IP) and Out-of-Plane (OP) load cases are demonstrated in Figure 3. The results are in line with the FE mesh-based solutions. The mode of deformation is well approximated in the macro model and it is close to the deformed shape simulated by the FE model.

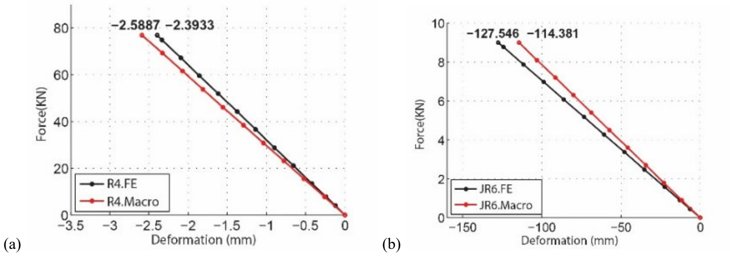

Figure 3: IP and OP behavior of Macro and FE models

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 ISBN: 978-91-88898-64-7

Structural design methods for tall timber towers with large wind turbine

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As wind power towers grows taller and wider, transportation issues of large parts to the site have appeared due to restriction on the roads. The company Modvion developed a modular design of wind power towers made of wood products and their concept provides advantages concerning transportation and assembly. Such a concept makes it possible to build tall towers with lower costs and reduced CO2 emission [1]. But technically, the challenges for tall timber towers are multiple and the durable resistance to strong winds must be carefully studied during the design state of the product. In [2], three different analytical methods have been implemented and compared to evaluate the loads effects on the wood structure for ultimate limit states. Description of the methods and the main outcomes are presented in this abstract.

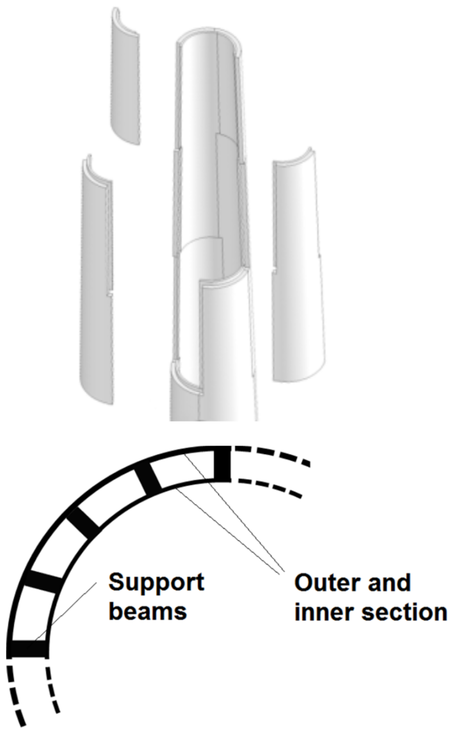

Figure 1: The modular design and the conceptual cross-section of the tower

The objective was to carry out a strength and functional analysis to verify the structural design of a 120 m tower in ultimate limit state. The work included: 1. an analytical model based on Euler Bernoulli beam theory, 2. a finite element model with shell elements and static loads and 3. a dynamic model including wind simulation in the time domain and interference from a 5 MW turbine installed at the top of the tower. The loads and the material strength verifications were applied according to the Eurocodes. The dynamic analysis was performed to verify that the turbine rotor speed is not operating in or close to the towers' first natural frequency and finally an elementary check of the risk from vortex shedding was studied. The structure of the tower was made of several timber modules with conical shape. The wall consisted of an outer and an inner panel made of LVL panels connected with glulam beams. A conceptual sketch of the modular system and the cross section is shown in Figure 1. The analytical beam model was implemented on MATLAB, the FE-model was developed with ANSYS and the dynamic analysis was done using FAST. The wind profile for the dynamic analysis was generated with TurbSim, which is a stochastic turbulent wind simulator, with characteristic mean wind speed of 12 m/s and turbulence factors according to the IEC standard for wind turbine.

The developed analytical beam model shows to be a useful and straight forward tool for an initial structural design of the tower. The results correspond well with the advanced FE-model. The dynamic model implemented with the FAST tools shows that the analytically determined loads are conservatively estimated. Comparable thrust is 25 % and 9 % smaller in the turbulent and uniform wind field respectively. The analytically calculated torque is closer to results from the dynamic analysis, which shows a 6 % difference in the turbulent and uniform condition. Thus, the simulation in the uniform wind field yields a more similar result to the analytical calculation than the turbulent wind field simulation. Moreover, the towers' structural materials and dimensions are fulfilling several requirements for strength resistance and functionality at ultimate limit state. Finally, the results imply that the different analytical methods can be implemented to design taller timber towers for greener and cheaper wind power production. However, there are still more conditional effects and scenarios that must be studied and analyzed in detail. Among others, the shear strength in the timber wall modules should be investigated and the fatigue strength of the connections is critical matter [3]. Fatigue strength of timber connections is not well known and will be tackled through series of cyclic tests.

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ISBN: 978-91-88898-64-7

Creep – Transfer of complex rheological behaviour into timber engineering

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Following the basic principles of the timber design code EC5 [1], not only displacements due to short term loading, but also additional displacements, such as according to creep, due to quasi permanent loading have to be taken into account for both serviceability limit state (SLS) and ultimate limit state (ULS) design. Although scientific research on this topic is challenging and still work in progress, its implementation in everyday timber engineering might be much less complicated, if it is well organised. Before presenting and discussing the aspects of an appropriate implementation of creep in both design codes and engineering software, a few basic considerations are made first.

In general, the **evolution of creep strains is a time dependent** phenomena and only related to stress components from permanent loading [2]. Such time dependent relationships could already be successfully derived for different building materials like concrete. It is well known from testing standards [3,4], that the **structural stiffness is only slightly reduced by creep** (maximum 10 % reduction of the initial stiffness). That is why assessment of dynamic behaviour - even at time infinity – is still to be performed with the initial stiffness. Typical for the anisotropy of wood, the evolution of **creep strains is dependent on the type of loading** [2]. Creep strains due to permanent shear stress is about three times higher than creep strains due to tensile stress parallel to the fibre. Therefore, it is obvious, that for a single span solid wood beam, the contribution of shear forces to the global displacements in comparison to the contribution of bending moments will be much more significant at time infinity than at time zero. The evolution of **creep strains also depends on the height of the corresponding stress component** from permanent loading. For the case of linear elastic distribution of bending stress over the beam height, it might happen, that creep strains are growing faster at the outer fibres than in the interior of the beam. Consequently, a redistribution of bending stress and reduction of the corresponding effective internal lever arm of the resultants of stress components might be the reason for a fictitious reduction of the nominal bending strength only valid for short-term loading conditions. Since the evolution of **creep is also significantly affected by the moisture content**, the knowledge about the distribution of moisture content across the cross section of a beam should be helpful for a more accurate estimation of the extent of creep strain evolution.

Following these basic mechanical observations, it seems natural to follow this track also within the domain of structural modelling and approval. Nearly every structural engineering software is capable to handle induced strains originated from loading by temperature. Therefore, a proposal [5] for **modelling of creep strains** could consist of a) calculating elastic strains from only one load combination dedicated to permanent loading, b) scaling these elastic strains or curvatures possibly by individual values of k_{def} and c) final application as separate load case to all load combinations at time infinity. For the case of heterogeneous cross sections with different creep behaviour of the subsections, the **consequence of eigenstress** and related curvature should be processed as well according to best practice with temperature, shrinkage or swelling. This procedure has also to be repeated for connections in terms of relative displacements, which again are well known from influence lines for internal forces of beam elements and only have to be reused this time as external loading for sake of creep. The good news are, that a) **structural stiffness is constant across all load cases** and b) at least for 1D and 2D structural elements, these procedures have already been activated in the software from Dlubal [6].

Concluding, the case sensitivity with respect to stages of approval, distributions of creep factors, order of analysis and stiffness parameters, as actually implemented in EC5, could easily be substituted by correct mechanics and complemented even within existing structural engineering software due to the fact of the already existing and working basic computational features. Therefore, EC5 could become slimmer with better interoperability to other design standards and partial safety factors could even be reduced due to more transparency of mechanics and realistic structural modelling.

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 ISBN: 978-91-88898-64-7

Cross-laminated timber

ISBN: 978-91-88898-64-7

Modelling principles of glued-in rods in cross laminated timber

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An experimental study of glued-in rods in cross laminated timber (CLT) was recently presented by some of the authors [1]. As a continuation, the experimental results are used for calibrating numerical models to study the effect of different parameters on the global load-bearing capacity and stiffness of glued-in rods in CLT. The numerical simulations are based on 3D finite element (FE) analysis using a cohesive zone modelling approach for the bond-lines between the timber laminations and the bond-line along the rod, to be able to capture the many different failure modes found in the experiments (Fig. 1). The numerical parameter studies concern influence of glued-in length, rod diameter and the direction of the rod axis with respect to the grain direction of the lamination within which the rod is glued.

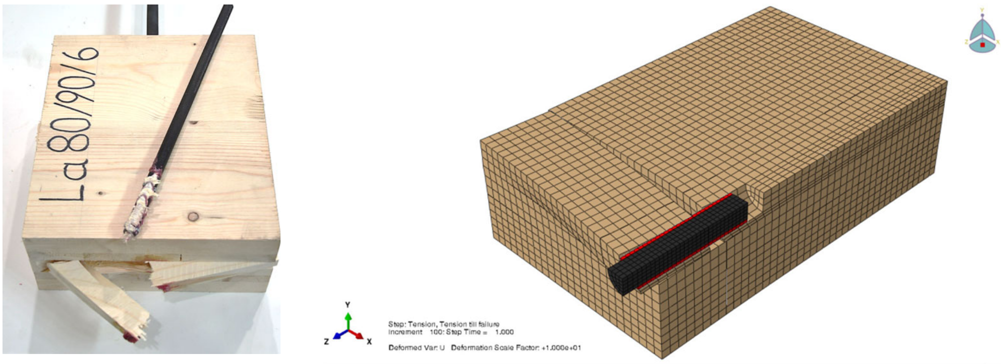

Figure 1: Comparison of experimental (left) and numerical (right) pull-out failure for the connection with the rod perpendicular to the grain and bonded-in length 80 mm.

In this contribution, the principles of the numerical modelling are presented in detail with focus on the calibration of the parameters for the cohesive zone model and on the comparison of experimental and numerical results (Figure 2).

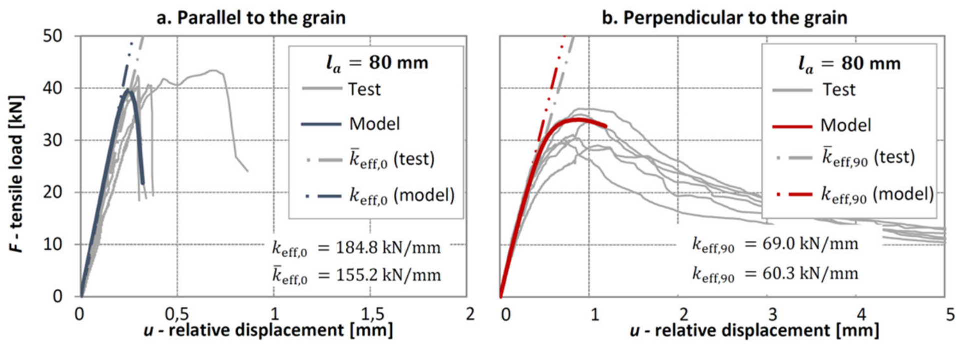

Figure 2: Load vs. displacement response of the numerical models (coloured curves) and the experimental tests (grayscale curves).

Furthermore, the results are compared to empirical and semi-empirical design equations for estimating the pull-out strength of glued-in rods in structural timber and glulam. The comparison shows that most of the existing equations overestimate the ultimate tension loads for connections with the rod placed parallel to the grain of the core CLT layer and underestimate the ultimate tension loads for connections with the rod placed perpendicular to the grain.

Glued-in rod connections have high potential for use in CLT structures, which was confirmed also within this study. The numerical results show that the load-bearing capacity generally increases with the glued-in length and with the rod diameter, which agrees well with the experiments. The numerical results further show partly different mechanical behaviour for different rod-to-grain angles, especially regarding modes of failure. Further research is however needed for holistic evaluation of load-bearing capacity and stiffness of glued-in rod connections in CLT.

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ISBN: 978-91-88898-64-7

A calibrated model for experimental hysteretic results of wall joints in CLT panels

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During the last twenty years, the use of cross laminated timber (CLT) in mid- and high-rise buildings has increased in many industrialized countries in Europe, North America and Australia. In South America, Chile has demonstrated a great interest in CLT technology due to the vast forest resource it can be found in this region [1]. However, the seismic hazard and earthquake occurrence rates in Chile confirm the strong necessity to know the dynamic behavior of CLT panels used in buildings. The typical approach in practice, and sometimes the option prescribed by the standards, is the use of a linear analysis with the characteristic design response spectrum. In such a situation, it is necessary to pay attention to the hold-downs and brackets [2,3].

In this work, a calibrated model for wall joints in CLT panels is presented and used to study the dynamic behavior of the connection. To obtain this result, an experimental characterization of three different configurations of wall joint connections (i.e. parallel walls, perpendicular walls (T) and corner walls (L)), is performed from tests subjected to cyclic loading protocols as described in DIN EN 12512 [4]. From these results, the hysteretic models are calibrated to obtain a comprehensive understanding of the viscous damping, ductility and energy dissipation capacity of the connection. The model is successfully validated with results from the hysteric tests performed on CLT panels' specimens. The validation clearly shows the capability of the model to predict the dynamic response of wall joints in CLT panels.

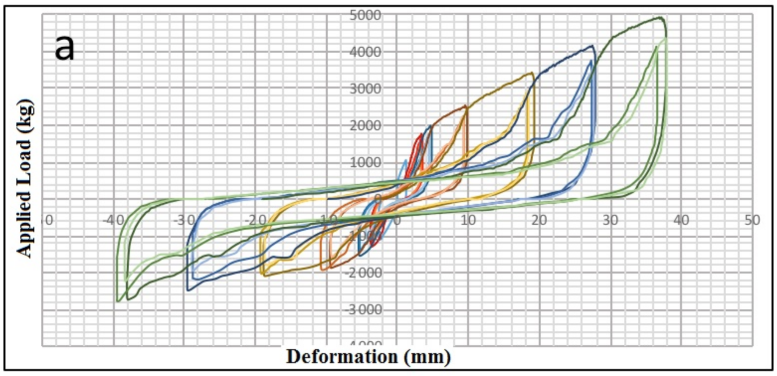

Figure 1: Hysteresis Curve of a parallel wall [5]

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ISBN: 978-91-88898-64-7

Strengthening of Cross-Laminated Timber by adding aluminium plates

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Wood is commonly judged as orthotropic with three material directions: longitudinal, radial and tangential. Shear stress and strain can occur in different directions on surfaces with different directions and shear stiffness is commonly described by three shear moduli. Of those, the weakest shear modulus is called the rolling shear modulus [1].

Cross laminated timber (CLT) is a rather recent and innovative engineered wood product with properties that can be improved and which still requires research. The benefits of using wood in buildings and construction are far from being maximized [2]. During recent years, timber has been used for constructing higher buildings. It has been seen that previous small and acceptable movements of the building are magnified, which can create discomfort for the occupants [3]. In these cases, the problem is the low in-plane shear stiffness of the CLT panel. One way to increase the in-plane shear stiffness is to build CLT mixed with other materials, with high modulus of shear, and by that increase the in-plane shear stiffness of the CLT panel. A practical test and finite element analysis (FEA) of the shear modulus was performed on 3-layer samples reinforced with aluminium plates, see Figure 1.

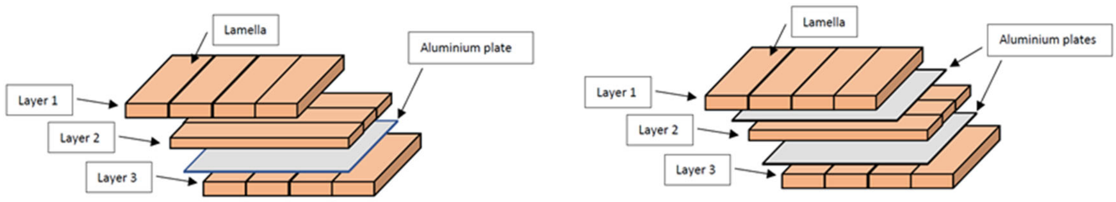

Figure 1: Constructed CLT plates reinforced with one (left)/two (right) aluminium plates.

The panels were built by three layer of wooden lamellas and the aluminium plate was added between the first and second and/or second and third layer of boards. Two different thicknesses of the aluminium plate were used, 1 mm and 1.5 mm. Also, panels without aluminium plates were used as reference. Diagonal compression test was performed on the CLT panels, see Figure 2, where the modulus of shear could be calculated. The diagonal compression method was performed based on experience from Andreolli [5].

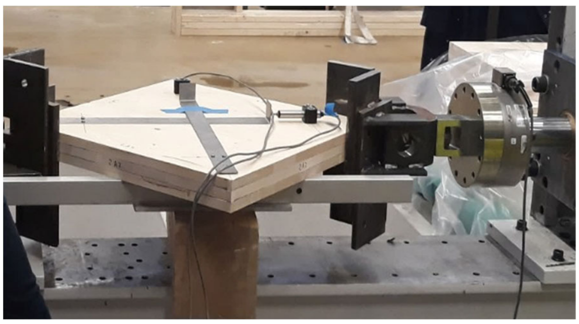

Figure 2: Practical diagonal compression test of a CLT panel

The panels containing aluminium plates had a higher shear modulus than panels without aluminium plates. This was concluded in both the practical testing and FEA.

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ISBN: 978-91-88898-64-7

Simulation of Alternative Load Paths After a Wall Removal in a Platform-Framed Cross-Laminated Timber Building

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An increasing number of multi-storey timber buildings use cross-laminated timber (CLT) for their bearing structure. Platform-framed CLT buildings consist of vertical repetitions of floors resting upon one-storey tall walls, squeezing-in the floor panels between the walls. Tall buildings need to be structurally robust because many lives would be at stake in case of a disproportionate collapse. Robustness is the ability of a system to survive the loss of components. For collapse resistance, it poses the last line of defence, after an unforeseen exposure (e.g. accident, terrorism) has already occurred and after the exposed components could not resist failure. A robust building offers alternative load paths (ALPs) which come into action when a part of the bearing structure has been removed [1].

Many alternative load path analyses (ALPA) have been conducted for tall concrete and steel buildings using the finite element method (FEM), but for timber, ALPA are still scarce. ALPs depend on the behaviour of the connections after a loss [1]. Studies on timber so far have accounted for connections in a simplified manner by lumping their aggregate behaviour into single points. Our goal is to elicit the ALPs after a wall removal in a platform-framed CLT building, study their development and quantify their capacity, to determine whether they can prevent a collapse.

We investigated a corner bay of an 8-storey platform-framed CLT building (see Figure 1) and removed a wall at the bottom storey. We studied the ALPs of each storey by pushing down the walls above the gap in a non-linear quasi-static analysis in the FE software Abaqus. We accounted for contact and friction, considered plastic timber crushing, and accounted for brittle cracking in the panels. We modelled single fasteners with connector elements which simulated the elastic, plastic, damage and rupture behaviour. We recorded the force-displacement curves, i.e. *pushdown* curves, for each storey and used them to conduct a dynamic analysis of the entire bay in a simplified model, as suggested by [2].

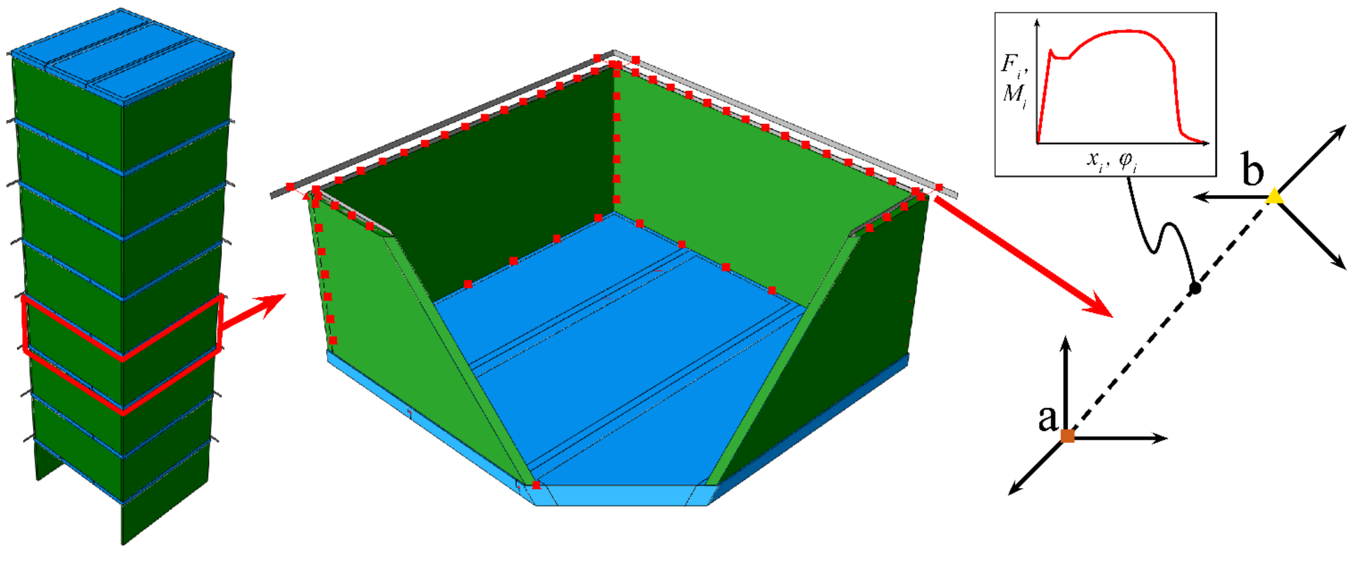

Figure 1: Model of the bay, a single storey and a single connector element

The results show that the structure could engage the following ALPs after a wall removal: I) arching action in the outer floor panels, II) arching action of the walls, III) quasi-catenary action in the floor panels, and IV) hanging action from the roof panels. The ALPs were limited by various parameters, but they sufficed to resist a collapse of the bay. We observed that the inter-storey stiffness influenced the load-sharing among storeys, which affected the structural robustness. In the compressed connections, friction, and not the fasteners, transferred most of the horizontal loads. Future research should test the squeezed-in platform joint experimentally, to quantify its capacity for transverse shear loads. We also advise to assess the inter-storey stiffness to estimate the capacity for load-sharing among storeys.

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Experimental Analysis and Numerical Modelling of Post-Tensioned CLT Shear Walls with Energy Dissipators

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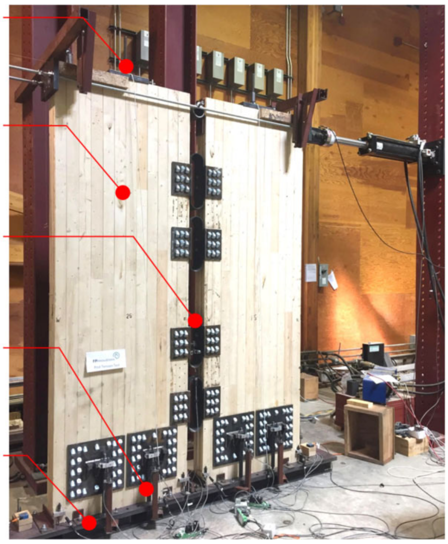
The prestressed-laminated timber (Pres-Lam) system, developed in New Zealand, is a low damage innovative wood-hybrid system that utilizes post-tensioned (PT) mass timber components along with various types of energy dissipators. For implementation of Pres-Lam system in North America, the in-plane lateral-load response of PT only and Pres-Lam Cross Laminated Timber (CLT) shear walls was investigated at FPInnovations [1].

A total of 14 different PT only and Pres-Lam CLT walls in four configurations were tested under monotonic and reversed cyclic loading. A coupled Pres-Lam CLT shear wall with axial energy dissipators (fuses) at the bottom and U-shaped flexural plates (UFPs) between the two CLT panels is shown in Figure 1a. PT only and Pres-Lam CLT shear walls had nonlinear behaviour with a decompression point corresponded to the state when the PT force started to increase. Pres-Lam CLT shear walls had higher resistance, maximum lateral drift and energy dissipation compared to PT only walls. The test results also showed that the behaviour of the Pre-Lam CLT shear walls can be de-coupled and a "superposition rule" can be applied to estimate the stiffness and resistance of such systems. Yielding and buckling of the fuses occurred at the early stage of loading as designed, and localized crushing of wood at the bottom ends of panels happened when the lateral drift was at or beyond 2.5%.



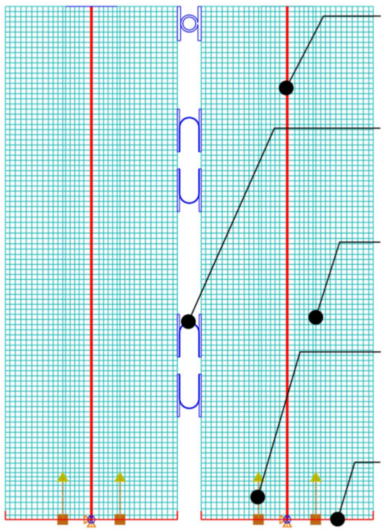


Figure 1: Coupled Pres-Lam CLT Shear Wall with Fuses and UFPs: (a) Tested Specimen; and (b) FE model

Specific numerical models of PT only and Pres-Lam CLT walls were developed in general purpose finite element (FE) software package, ABAQUS. The CLT panels were modelled using shell elements with adequate strength and stiffness properties in each orthogonal direction. The steel post-tensioning cables and the UFPs were modeled using truss and shell elements, respectively, with the strength and the stiffness properties of the steel used in the testing program. The fuses were modelled using connector elements with the hysteresis loops of fuses obtained in the testing program. The foundation was modeled using rigid elements. "Softened" contact with friction was adopted in the interaction between the CLT panels and the foundation. The fuses were connected to the CLT panels and the foundation using multiple point constraints (MPC) technique. Using MPC, UFPs were also connected to the CLT panels in the coupled wall configuration (Figure 1b). The PT force applied to the wall was achieved by lowering the temperature of the PT bar which will shorten correspondingly. The response of the developed models agreed well with the experimental results of the tested wall configurations. The influence of the PT force level, the aspect ratio of the wall panels, the spacing and number of energy dissipating devices on the response of the system were investigated using the verified models. The testing and modelling results gave a valuable insight on the behaviour of the PT only and Pres-Lam CLT shear walls under in-plane lateral loads.

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ISBN: 978-91-88898-64-7

A Finite Element Approach to Investigating the Influence of Knots on Cross Laminated Timber

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Cross laminated timber (CLT) typically consists of an odd number of laminated orthogonal layers of high quality dimensional lumber. High quality species have limitations on naturally occurring defects resulting in smaller variabilities in material properties when compared to low-value species. Knots are naturally occurring defects which reduce the strength and stiffness of wood by interrupting the direction and continuity of the wood fibers; however, the composite nature of CLT provides an opportunity to utilize low-value species typically considered inadequate for structural purposes. The cross lamination of the wood boards allows for an averaging effect of the stress concentrations in the panels and reduces local stress concentrations around knots and other defects. This research investigates the influence of knots on the strength and stiffness of wood at two different scales: within dimensional lumber and within a full CLT panel. Three dimensional finite element models of knots in clear wood are presented at these two scales and the influence of knots on the effective material properties are analyzed within the elastic and strength cases. The models are validated through full scale experimental testing.

The costs associated with fabricating and testing CLT panels provide significant limitations when investigating the influence of knots on the stiffness and strength of CLT fabricated from low-value woods. Thus, a stochastic model for the distribution of knots in dimensional lumber is applied to create synthetic geometry for the finite element analysis models of the CLT layups. This allows for a preliminary reliability analysis of low-value woods used as constituents in CLT.

The development of an accurate and efficient three dimensional finite element model of cross laminated timber panels provides significant opportunity for growth and expansion of CLT construction. Finding applications for underutilized species creates potential for a promising market for low-value wood species that are abundant in the United States. Additionally, finding commercial markets for low-value woods supports national forest management strategies to improve forest health, while giving rise to more sustainable building practices and increased job opportunities in rural areas of the United States.

ISBN: 978-91-88898-64-7

Modeling and testing of connections

ISBN: 978-91-88898-64-7

Simplified mechanical models for timber connections in fire

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Only few numerical models have been developed to study the fire behaviour of timber connections, even though experimental studies are also difficult to perform. Two main approaches have been followed: 3-D thermal finite-element (FE) models combined with simplified numerical mechanical models of single fasteners; and 3-D thermal-mechanical FE models. In the first approach, a 3-D heat transfer FE analysis is performed to determine the temperature fields in the connection and a simplified mechanical model is then used to estimate the mechanical behaviour at each time step. An advantage of this approach is that, since no 3-D FE stress analysis is performed, the overall number of model parameters, including the assumptions about their temperature dependency, is immensely reduced. In addition, simplified models can be very informative and allow assessing which parameters are relevant to describe selected aspects of the connections' mechanical behaviour under fire. Since they are also much faster, they allow studying how sensitive these parameters are to various temperature-dependency models.

The proposed paper presents the results of a study on the influence of various temperature-dependency models (Figure 1) on two simplified mechanical models for timber connections in fire, namely based on Johansen's yield models [1] and beam-on-elastic-foundation models [2] (Figure 2). The complexity of the load-carrying model should be adjusted to each specific study: Johansen-type models are very simple and adequate to estimate the load-carrying capacity, but do not take directly into account load redistribution between fasteners, which might be very important in some situations; beam-on-springs models allow accounting for load redistribution between fasteners and establish a time-displacement curve, as well as assessing the stiffness of the connection, but require more temperature-dependent parameters that also have to be calibrated (stiffness and resistance of the springs). Nevertheless, even though both modelling approaches rely on very few parameters, they are able to capture the load-carrying capacity after a given period of fire exposure, in the case of Johansen's yield models, and the time-displacement behaviour, in the case of the beam-on-springs models (Figure 3). The study revealed that some of the temperature-reduction factor relationships do not seem to influence the mechanical behaviour of the connections (e.g. plateau between 20 and 50 °C), whereas others have a significant influence (e.g. residual load-carrying capacity above 300 °C).

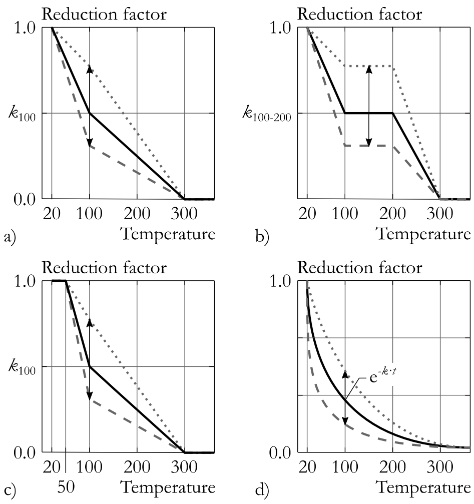

Figure 1: Examples of single-parameter temperature-dependent reduction factors.

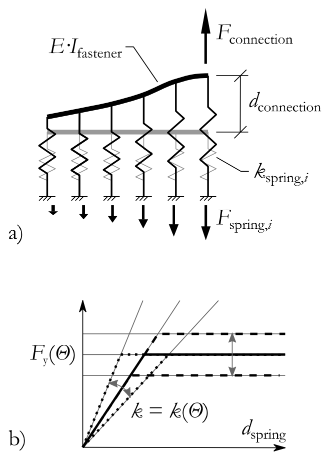

Figure 2: Beam-on-springs model: a) single fastener of a steel-to-timber connection; b) load- displacement behaviour of a spring.

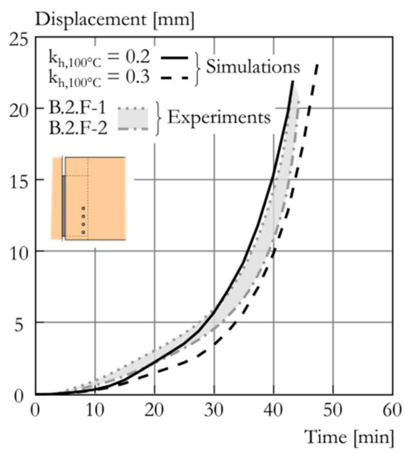

Figure 3: Experimental and simulated displacements (beam-on-springs model and two different spring stiffness-temperature relationships).

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Numerical Modelling and Experimental Investigation of Compressed Wood Dowel Connected Laminated Timber Members

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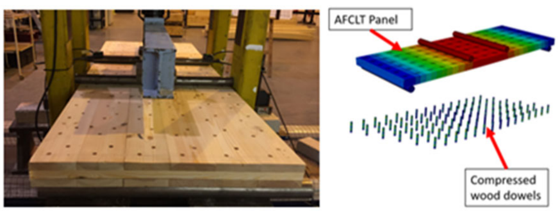
The use of Engineered Wood Products (EWPs) in the construction industry is continuously growing due to a number of reasons, which include their availability in large-sized structural sections, low embodied energy and carbon, as well as more consistent mechanical properties [1]. However, the prevalent use of petrochemical adhesives in EWPs raises safety and environmental issues (e.g. release of harmful gases) and has led to increased research interests around the globe to develop more environmental-friendly EWPs [2]. Hence, the development, experimental investigation and numerical modelling of novel and sustainable EWPs offer new prospects for the efficient utilisation of timber materials, which also contributes to a low carbon economy. This work thereby describes an investigation into the structural response of Adhesive Free Laminated Timber (AFLT) beams and Adhesive Free Cross Laminated Timber (AFCLT) panels. Figure 1 and Figure 2 show the experimental setup and finite element modelling of the AFLT beams and AFCLT panels, respectively. These structural members are made solely from timber lamellas and connected with highly compressed wood dowel fasteners. This is because highly densified wood materials have beneficial spring-back and greater mechanical properties that make them suitable as durable fasteners in structural connections.

Material characterisation of the uncompressed and compressed wood dowels was carried out to determine their mechanical properties. Following that, experimental tests were carried out on the large-sized AFLT beams and AFCLT panels. By using the mechanical and geometric properties of the timber lamellas and compressed wood dowels, the finite element (FE) models (using a commercial code, ABAQUS) were developed to supplement the experimental work. The FE model also incorporates custom written subroutines that allow the representation of the unique properties of highly compressed timber. Thereafter, design optimisation of the dowel laminated timber members is investigated by varying the patterns, sizes, and configurations of the dowels. In summary, this work demonstrates the development and characterisation of innovative adhesive free laminated timber members alongside useful structural analyses, all of which support the delivery of more sustainable buildings and green construction.

(a) (b)

Figure 1: AFLT beam: (a) Experimental setup and (b) Finite element modelling



(a) (b)

Figure 2: AFLCT panel: (a) Experimental setup and (b) Finite element modelling

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ISBN: 978-91-88898-64-7

Numerical simulation of full-culm bamboo structural member connections

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Bamboo is an inexpensive, functionally graded material, that in many cases possesses a strength-to-weight ratio comparable to that of steel [1]. In addition to that, as a grass species, it is essentially fast-growing and environmentally friendly. These remarkable characteristics suggest that it can potentially be a very advantageous alternative to conventional construction materials, which is also showcased by recent developments in architecture [2], [3]. One main issue that needs to be overcome in order to facilitate a more widespread use of bamboo is the lack of reliable bamboo structural member connections. Herein we examine the failure modes of a bolted bamboo-to-steel connection that allows the utilization of more than one culms [4]. To that end, a detailed finite element model of such a connection is developed, that simulates all the pertinent joint components (gusset plate, steel bolts, bamboo culm) (Figure 1). The model incorporates failure laws for both materials. The failure law for bamboo specifically is calibrated using experimental data. Particular emphasis is given on capturing crack initiation and propagation using the eXtended Finite Element Method (XFEM, Figure 2) [5]. The orthotropic nature of the material and the high deformations that take place due to its relatively low Young's modulus are also taken into account. The analysis results are compared with pertinent experimental results and the model is calibrated accordingly until an acceptable error margin is achieved. The final outcome is a reliable, experimentally calibrated FEM model, that can be used to predict how the examined connection system is going to fail and, subsequently, be implemented to further improve its efficiency.

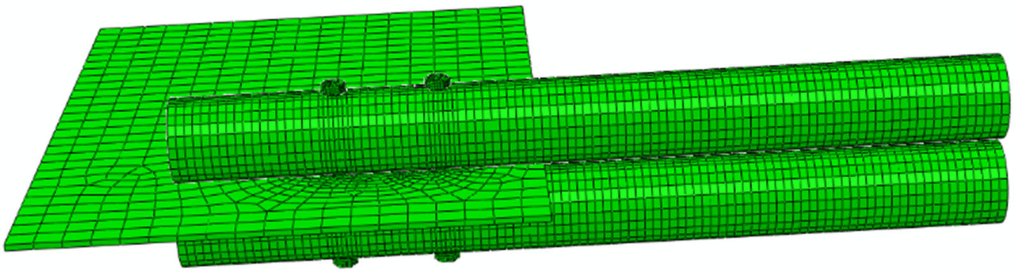

Figure 1: Two-culm member connection

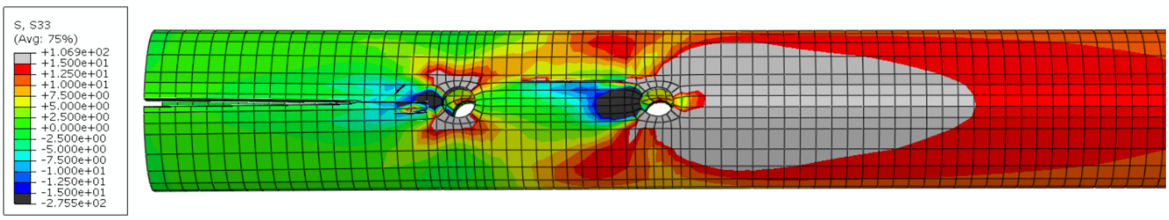

Figure 2: Axial stress (legend in MPa) and longitudinal splitting of the culm at displacement=2.5mm

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ISBN: 978-91-88898-64-7

Long-term finite element analysis of timber-steel composite joint

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The long-term behaviour of timber structures under service loading conditions is significantly influenced by timber creep which depends on fluctuations of relative humidity (RH), temperature and level of service load. Accordingly, accurate deflection control and analysis of timber, timber-concrete and timber-steel composite members under service load is not possible, unless the coupling of hygroscopic behaviour, temperature effect and mechanical loads on timber behaviour are adequately considered. The material models used for predicting long-term behaviour of timber elements are mostly formulated in the framework of visco-elasticity assuming that the stress in timber elements subject to long-term service load remains well within the elastic range of timber behaviour [1,2]. The visco-elastic material models have been incorporated into 1D beam [3] or 2D/3D continuum-based [4,5] finite element models to predict the global and local behaviour of timber/timber-concrete composite beams and connections, respectively. The 1D beam finite element models are effective in predicting the global long-term behaviour, but their accuracy heavy rely on the empirical models adopted for predicting the complex time-moisture-temperature dependent behaviour of the mechanical connectors. However, the 3D finite element models can predict the long-term behaviour of the mechanical connectors embedded in timber.

The results of long-term test (over 18 months period in uncontrolled in-door conditions) conducted on cross laminated timber (CLT) to steel connections with coach screws are briefly discussed. A 3D orthotropic visco-elastic model of predicting creep deformation developed and implemented [4,5] in ABAQUS finite element software is employed to predict the long-term behaviour of CLT-steel composite joints. In the finite element simulations, the elastic deformation, plastic deformation, hygroexpansion deformation, visco-elastic deformation and mechano-sorptive deformation were taken into account and the variation of moisture and temperature were also considered. It is shown that the adopted finite element model can predict the slip versus timber response of the composite joints with reasonable accuracy.

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ISBN: 978-91-88898-64-7

Tensile loading tests steel plated inserted joint with drift pin on CLT

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CLT is the latest wooden structural material member that will be applied to wall, floor and other uses in large scale wooden construction worldwide. For wooden architecture, its joint system is the most important factor for its structural performance. There are many types of connection methods and different types of fasteners, bolts, drift-pins, nails etc. are used in its construction [1]. For structural design, we have to analyze and clarify the resistant mechanisms and mechanical performance of joints, but the study of connections for CLT members is still in its early stages.

In this paper, we conducted tensile loading tests of plate-inserted connections with drift-pins in the CLT. For the test parameters, we changed the drift-pin diameter, end-distance (e_1) and edge-distance (e_2). CLT members would be composed with 5ply Japanese cedar, its thickness was 150mm, graded Mx60A-5-5 in Japanese Agricultural Standards[2]. The test set up is indicated in Figure 1, the test parameters are shown in Table 1. As the ratio (b/d) of wood thickness (b) to drift-pin diameter (d) is large, we can see the large bending deformation of the drift-pin. On the other hand, if the ratio (b/d) is small, we can't see bending deformation or fracture of the wood member.

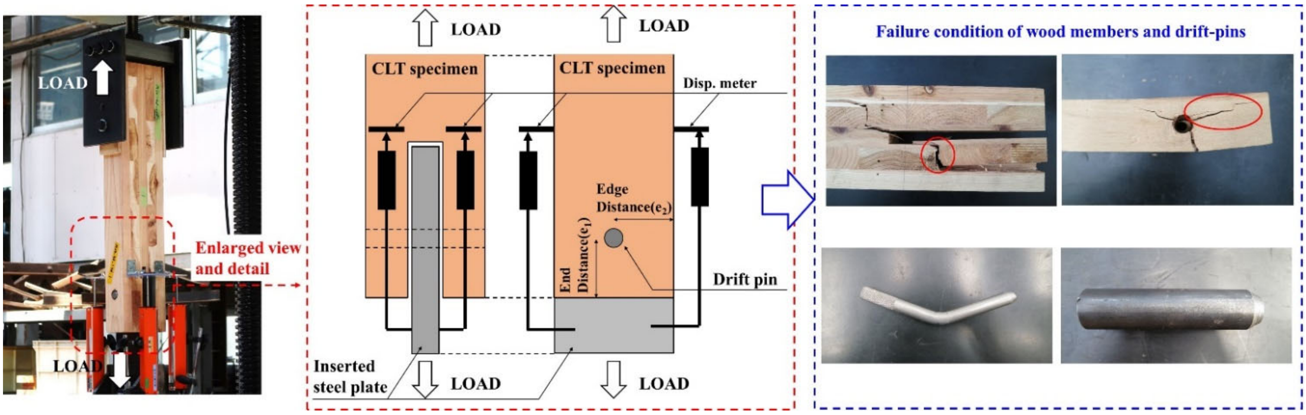

Figure 1: Test and failure condition of tensile loading with inserted plate connection with drift pin for CLT

Table 1 : Test parameter

Diameter of drift-pin	End distance (e_1)	Edge distance (e_2)
12[mm]	4d	1.5d
24[mm]	5d	3d
36[mm]	7d	

Figure 2 shows examples of the relationship with maximum load, yield strength and stiffness to the pin-diameter. As shown the figures below, we can see a strong correlation between performance and pin-diameter.

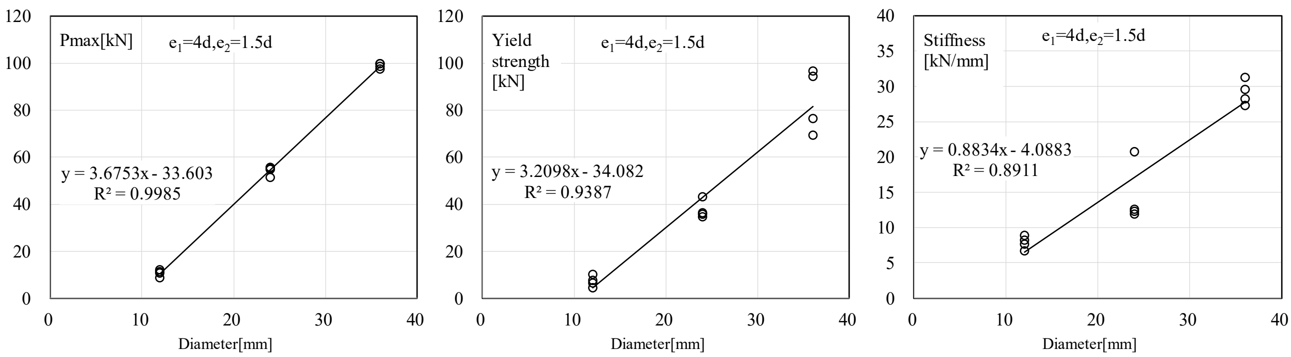

Figure 2: Relationship with mechanical performance and pin diameter (e_1=4d and e_2=1.5d)

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Nonlinear 1D component based and 3D continuum-based finite element analysis of hybrid timber-steel beam to column connections

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Current study deals with numerical simulation of steel-timber hybrid frames. A 3D continuum finite element model developed in ABAQUS is proposed to simulate the behaviour of steel-timber composite beam to column connections with double web angles under negative bending moments. Inelastic behaviour of timber is captured by a multi-surface continuum damage mechanics constitutive law implemented in a UMAT subroutine. The adopted timber material model used in the current study considers the anisotropic behaviour of timber and accounts for the hardening behaviour of timber in compressions perpendicular to the grain direction, i.e. radial and tangential directions, as well as the brittle failure of timber under shear and tension. For the sake of simplicity and reduction of computational costs, timber is treated as a transversely isotropic material. Crack band model in conjunction with a specific meshing strategy are used to alleviate mesh sensitivity of the problem. Also, contact and geometrical nonlinearities are considered along with the nonlinear response of the steel profiles, bolts and connectors. In addition, a component-based 1D finite element model developed in OpenSees is utilised to capture the nonlinear response of joints, where action of each component is modelled by using a zero-length spring. Results of simulation for both finite element models show a good agreement with results of experiments conducted on joints. Sample results of simulations and their correlation with the experimental results are shown in Fig. 1.

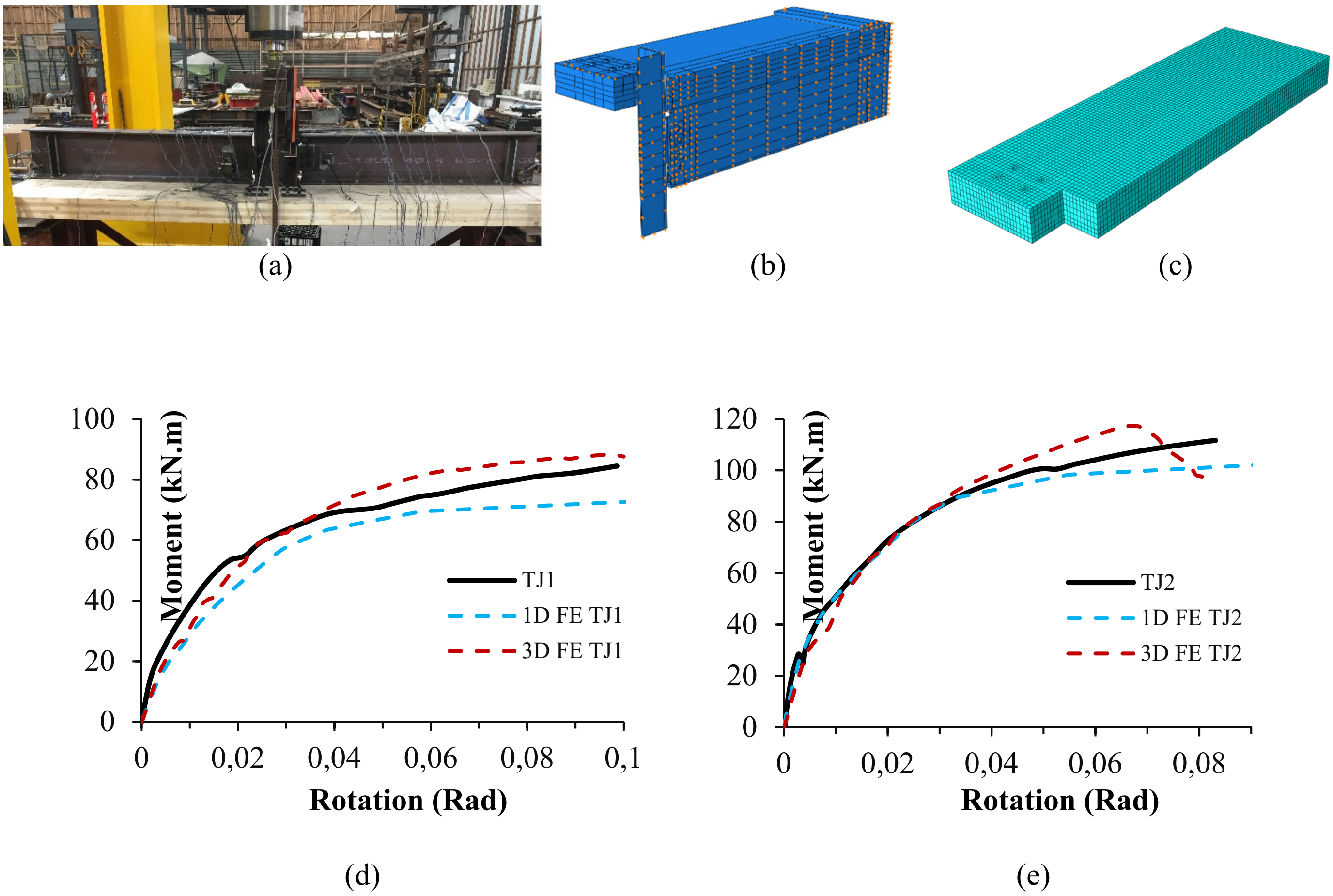

Figure 1: FE modelling (a) tested specimen (b) 3D Continuum model (c) meshing outline of CLT slab (d) results of FE simulation TJ1 (d) results of FE simulation TJ2

ISBN: 978-91-88898-64-7

Simulation and testing of materials and structures

ISBN: 978-91-88898-64-7

In-plane Elastic Behavior of Transparent Wood Composite Measured with Digital Image Correlation

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Wood has been used as a load-bearing material in applications such as buildings and constructions for years, due to its renewable nature, easy manufacturing ability and excellent mechanical performance. The advancement of nanotechnology enables modifications within the wood hierarchical structure such as lumen and cell wall, providing new possible functionalities when impregnated in a poly methyl methacrylate (PMMA) matrix. Particularly, the concept of transparent wood composite stands out since it has interesting optical properties while preserving the load bearing capacity and microstructure that wood naturally has [1]. The reinforcement effects from a delignified, nanoporous wood template are unknown, and are compared with native wood veneer as the reinforcement. It has earlier been shown that transparent wood composites are possible to laminate for desired mechanical performance, which expands the usability for the material as it can be customized for applications [2]. However, characterization of the in-plane elastic parameters from a single ply is then a necessity as they form the basis of lamination theory of orthotropic materials. Within this work the interest is to determine the four elastic parameters. The in-plane strain field is also investigated in detail during deformation, and reinforcement effects are analysed. To resolve the matters a set of tensile experiments were done and a non-contact deformation measurement technique (digital image correlation) was used to capture the strain fields during quasi-static loading. The focus has been on small strip samples from veneers with material axes along the woods tangential (T) and longitudinal (L) axes.

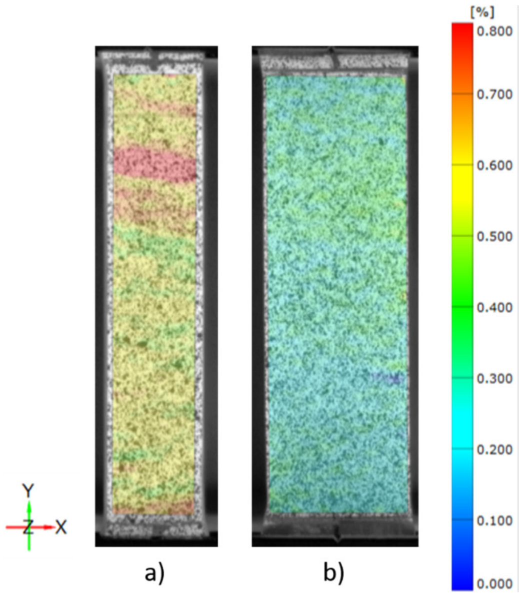

Figure 1: Strain field at average Strain 0.002 in vertical Y-axis along the tangential material axis of a) Native Wood and b) transparent wood composite.

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ISBN: 978-91-88898-64-7

Experimental and Computational Models of Bamboo Reinforcement

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Bamboo is a naturally available material that has high mechanical strength [1]. Bamboo reinforcement has emerged as a possible alternate material to steel reinforcement in the concrete element. However, bio-based materials utilized for the construction activity are normally questionable under their sensitivity to moisture conditions and durability. In order to understand and compare the long-term behaviour between bamboo reinforcement and standard industrial reinforcement, we conduct the present experimental study. Various methodologies of treatment were evaluated with the same physical and mechanical properties to assess their effectiveness in the treatment method. First, to improve their durability bamboo samples were treated to decrease the sensitivity to moisture. Different conditions of treatment process were tested: heating of bamboo, with and without chemical treatment. Then, a comparative experimental investigation was conducted on bamboo reinforced concrete beams, cubes and cylinders in order to find out the flexural strength, split tensile strength and bond strength. Results obtained from experimental investigation showed that some treatment methods could increase the compressive strength and durability with and without treatment of bamboo. The present experimental campaign will be used to calibrate current computational models under development. This is part of an ongoing research.

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ISBN: 978-91-88898-64-7

Multi-Objective and Multi-Criteria Approach for Value-Driven Design in Industrialized Residential Multi-Storey Timber-Building

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Serviceability in terms of springiness, vibration and deflection [1], as well as sustainability in terms of climate impact and costs [2] have been identified as the most important aspects for appropriate functioning in residential multi-storey timber-buildings. Thus, the aim of this study is focused on product development of a timber-concrete composite (TCC) floor system by 1) enhancing serviceability performances of the floor for larger spans (above 6 m) in terms of stiffness and dynamic response, and 2) reducing both climate impact (CO_2-emissions) and costs, by optimizing material usage.

As the case study a timber structure of a residential multi-storey building, including concrete ground floor and shaft, with the overall dimensions $L \times W \times H = 30 \times 11 \times 14\ [m^3]$ has been studied. The geometry of the load bearing structural elements has been modelled using finite element programs. As serviceability criteria for the floors, the deflection due to a point load was chosen. The deflections were related to comfort classes given in [3] and transverse load distribution was taken into account according to [4]. The deflection and effective bending stiffness (EI_{ef} in EC5 Annex B) were chosen as objective functions, while thickness of concrete slab and shear stiffness of the connection between glulam beam and concrete slab were chosen as design variables in a multi-objective optimization. The relationship between connection stiffness and height of the concrete slab for comfort class B can be seen in Figure 1. In the figure the cross-section of the TCC floor structure, with a span of 7.5 m, is also depicted.

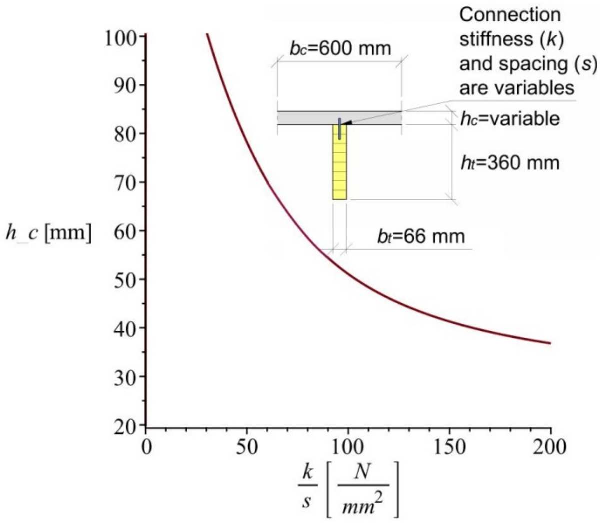

Figure 1: Connection stiffness-concrete thickness relationship and cross-section for the TCC floor.

After optimization, a multi-criteria analysis was applied to select a design solution from the Pareto optimal front, satisfying some subjective preferences of the stakeholders for value-driven design. The results in this study integrates serviceability, environmental and economic performances for value-driven design and supports decision making in the early phases of a project, where various alternatives can be analyzed and evaluated.

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 ISBN: 978-91-88898-64-7

Analytical evaluation of bond models for glued-in rods in timber

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The experimental investigation of glued-in rod connections has been the subject of many studies. However, to date the modelling of these connections and the understanding of their mechanics has been limited in literature [1]. The aim of this study is to evaluate known bond models against experimental data of glued-in CFRP and GFRP rods in block laminated timber as reported in [2]. The CFRP and GFRP rods were tested in a pull-compression method with a 50 mm bonded length corresponding to 5*D*, where *D* is the core diameter of the rod. Despite pull-pull test method being commonly applied for studying glued-in rods in timber, the pull-compression test method allows experimental recording of the slip values at both the loaded end (end where the load is directly applied) and free end of the specimens. Therefore, bond stress slip models can be analytically studied following an energy based method according to Equation (1) [3]

$$W_{int} = {}^{1}\!/_{2}\,{}^{A_r}\!/_{E_r}\,\sigma^2 = \iint_{s_f}^{s_l} \tau(s)\,\mathrm{d}s\,\mathrm{d}\Omega = W_{ext} \tag{1}$$

where W_{int} and W_{ext} are the internal energy and external work done respectively, A_r, E_r are the rod's cross sectional area and elastic Young's modulus accordingly, σ is the axial normal stress, $\tau(s)$ is the bond stress slip model, s_l and s_f are the loaded end and free end slip values and Ω is the surface area of the rod.

Moreover, specimens with 4 strain gauges attached on the rods and equally distributed along the bonded length (Figure 1a) and an additional specimen cut in half and analysed with Image Processing technique (GeoPIV) [4] (Figure 1b and c) were prepared to understand the bond stress transfer mechanism during the pulling out of the rods and any induced stress concentrations due to the adopted test method.

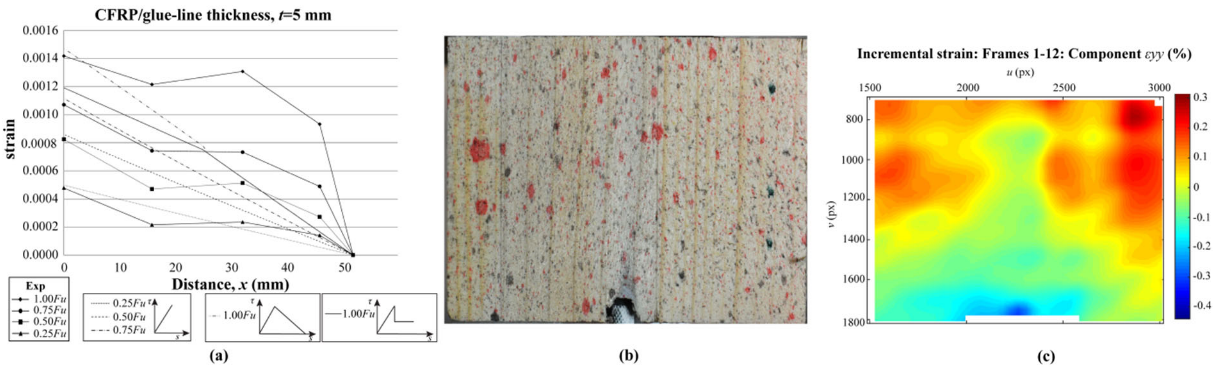

Figure 1: (a) strain distribution of a CFRP rod glued-in block laminated timber, (b) GeoPIV specimen and (c) strain analysis εyy.

A linear and the m.B.E.P ascending bond stress-slip models have been chosen and their constant values are derived following the Nedler optimization algorithm using a Matlab script such that the error function is minimized. To assess a non-uniform bond stress distribution scenario along the bonded length, bilinear models are also adopted. The aforementioned models neglect the mechanical properties of the adhesive layer and might be suitable for very thin glue-line thicknesses. For ease of comparison, the Volkersen model and a new model adapted from [5] that account for the mechanical properties of the adhesive layer are also studied.

The indentification of a suitable bond model relies on the bond failure mode of the glued-in rod connections and can enable their accurate modelling in a detailed analysis stage. The analytical prediction of the slip values allows the calculation of the bond stiffness at each loading stage and consequently the overall stiffness performance of glued-in rods in timber under tension.

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ISBN: 978-91-88898-64-7

Determination of shear modulus (G_{LR}) for seven boreal species using a bending test and non-destructive methods (ultrasound and torsional resonance methods)

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In general, a static bending test is used to determine either the modulus of elasticity (MOE) or the modulus of rupture (MOR) of materials. In the case of wood, this test is used to evaluate the shear modulus. This is mainly due to the complexity of performing a static shear test on wood. To overcome this complexity, empirical approaches and non-destructive testing could be used such as ultrasound and torsional resonance methods. This study aims to (i) present an alternative methodology for determining the shear modulus in the longitudinal-radial plane (G_{LR}) of wood using the three-point flexural test ($G_{f,LR}$) (ii) investigate the relationship between $G_{f,LR}$ and MOE as well as between $G_{f,LR}$ and MOR (iii) and predict the static shear modulus from dynamic modulus determined by the ultrasonic ($G_{u,LR}$) or the torsional resonance ($G_{r,LR}$) methods. Seven boreal species were studied: white spruce, white birch, hybrid poplar, trembling aspen, jack pine, eastern larch and eastern white cedar. The results showed a moderate correlation between $G_{f,LR}$ and MOE, with a correlation coefficient (R^2) of 0.50. A stronger correlation was found between $G_{f, LR}$ and MOR (R^2=0.66). These results proves the effectiveness of the static bending test to determine the shear modulus. An average correlation between $G_{f,LR}$ and $G_{u,LR}$ (R^2=0.55) is similar to that obtained between $G_{f,LR}$ and $G_{r,LR}$ (R^2=0.50). However, the torsional resonance method is the one that provides values closer to the static values of $G_{f,LR}$. These results show the usefulness of non-destructive methods for estimating $G_{f,LR}$, with an emphasis on the reliability of the torsional resonance method.

Keywords: Shear modulus, non-destructive methods, ultrasound, resonance method, static bending test

ISBN: 978-91-88898-64-7

Modelling moisture in wood

ISBN: 978-91-88898-64-7

Heat and Mass Transfer Model for Wood under real climate conditions

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Knowledge about the wood moisture condition in a timber component is essential to predict its mechanical behavior. Not only stiffness and strength properties are highly dependent on the wood moisture content but also diffusion coefficients, density, specific heat capacity and the thermal conductivity. Therefore, modern prediction tools, which are able to describe these effects, can be of great benefit for the development of new wood-based products. Especially if these products exhibit complex geometries and are made of materials with different moisture characteristics. Such products have to be tested in varying climate conditions. Different and direction-dependent coefficients of expansion of wood may lead to critical stresses due to deformation. The stress levels depend on geometric and climatic conditions.

Transport mechanisms below the fiber saturation point were developed by Krabbenhoft and Damkilde [1] and Fortino et al. [2]. Three coupled differential equations describe bound water, water vapor and energy conservation. Free water exists above the fiber saturation point with the corresponding transport mechanisms described in Perré and Turner [3]. Values of the free water content can be much higher than those of bound water and water vapor. Thus, within the areas, where the switch from the transport mechanisms below the fiber saturation point to those above occur, high gradients can exist. To deal with these high gradients in terms of the finite element method different procedures like upstreaming and mass lumping described in Eriksson [4] were used. A three-dimensional Abaqus User Element Subroutine was developed to describe these coupled equations. This model was validated with results from Frandsen [5] below the fiber saturation point and with results from Eriksson [6] during drying from a moisture content above the fiber saturation point to dry conditions.

Linear elastic stress calculations were conducted based on the obtained moisture content and temperature fields. Expansion due to swelling and shrinking as well as temperature were considered. Furthermore, a multi-surface failure criterion [7,8] was applied which leads to moisture induced failure.

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ISBN: 978-91-88898-64-7

Finite-Element-Modelling of moisture-induced cracks in wood and wooden structures

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Wood is characterized by heterogeneity at different length scales. Knots or growth-related annual rings determine the material-specific characteristics at the macroscopic level. Lignin, hemicellulose and cellulose are the main components of the microscopic cell structure of wood. Cellulose fibers form the cell walls and are enclosed by hemicellulose and lignin as the matrix material, see [1].

Deformation of wooden structures and stresses in wood materials might be caused by changes of moisture content. Moisture transport as well as moisture inclusion mainly occurs in the inter-micellar and inter-fibrillar void system. Especially the water at the microscopic pore system, bounded due to chemical sorption, adsorption and capillary condensation, causes changes of the microscopic structure of wood. These changes induce swelling and shrinkage deformation, which might be accompanied by stress formation in wooden materials. If tensile stresses exceed the appropriate tensile strength, brittle failure occurs, compare [1]. At compressive stresses beyond the yield strengths, wood shows ductile failure modes.

The contribution at hand presents an approach to model moisture-induced cracks in wooden structures by the Finite-Element-Method (FEM). The displacement, moisture and temperature dependent constitutive description of wood is taken from [2] and is used as basis for evaluating the load dependent stress state. The crack initiation criterion developed captures brittle failure at tensile stresses above the yield level. A node duplication algorithm within the FEM-framework is formulated and implemented in order to model crack propagation within wooden materials. The formulation captures the modelling of load dependent crack paths as well as possible crack branching.

The numerical verification of the crack modelling approach is carried out by convergence studies with respect to mesh dependency and numerical behaviour of the solution of the global finite element equations. Appropriate examples, based on experiments from literature, are presented, see e.g. Figure 1.

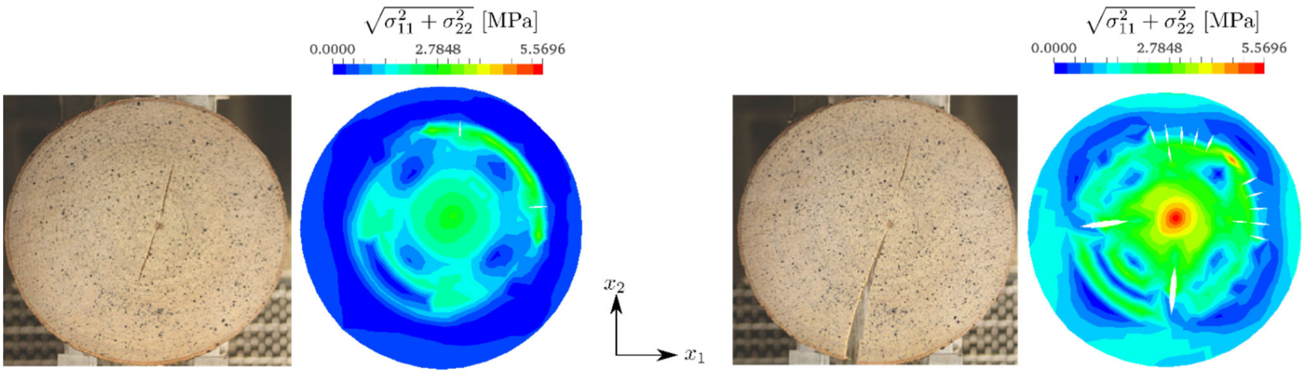

Figure 1: Comparison of numerical simulation and experimental investigation (taken from [3]) of a 20-year-old Norway spruce (*Picea Abies*) tree slice at drying process

The drying experiment in Figure 1 is carried out in a climate chamber for Norway spruce, see [3] for details. Figure 1 shows simulation results, which are computed by the contribution at hand. As can be seen, a qualitatively agreement between the position of the crack initiation as well as the crack propagation steps can be observed. The discrepancies between simulation and experiments are caused by the uncertain material properties and the applied initial conditions, used in the simulation, as well as the differences in the annual ring positions.

A further validation of the fracture modelling is presented and carried out by drying experiments for beech wood (*Fagus Sylvatica*), in cooperation with the Chair of Timber Engineering and Construction Design of Technische Universität Dresden.

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ISBN: 978-91-88898-64-7

A multi-phase coupled transient heat and moisture transport model in wood based on the hybrid mixture theory

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Building materials most commonly consist of porous structures involving high inhomogeneity in relation to structural formulations, material properties and constituents. Wood in general is one example, whose porous fibers exhibits high affinity towards moisture. The behavior of such porous materials when exposed to varying environmental temperature and moisture conditions is of special interest in the construction industry. This is due to the high effects and variations of the wooden material structure, strength and stiffness when moisture interacts with its fibers.

Moisture flow and distribution within a porous media involves additionally complex processes, where at standard ambient conditions of use the pores in wood usually contain moist air and water molecules are bonded to the fibers (bound water). Sorption processes of water to and from the fibers at different temperatures takes place and are a function of the equilibrium sorption isotherms relating the relative humidity and temperature to moisture content.

This study presents the application of the hybrid mixture theory (HMT), applying the balance principles in the realm of continuum mechanics governed by the principles of thermodynamics to describe a multi-phase multi-constituent heat and mass (moisture) flow process in wood. A tri-phase model consisting of the wood fibers, liquid moisture and gas is developed and implemented into a two dimensional 2-D test example in a non-linear, coupled finite element code. This study contributes to the stringent understanding of transient moisture transport in porous media through application of a sophisticated thermodynamically consistent model. Related studies involving the HMT have been successfully applied to a variety of other materials including: clay soils [1], biopolymers [2] and paperboards [3,4].

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ISBN: 978-91-88898-64-7

Brittle failure of wood

Investigations of crack formation and delamination in bonded wooden elements in variable climatic conditions in the interior

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In wood construction, but also in interior construction, increasingly glued wood is used. Glued laminated timber (GLT) or cross laminated timber (CLT) are often used in timber constructions, while solid wood panels or multi-layer parquet are typical products for the interior.

Especially in the winter season, when buildings are heated, low indoor air humidity levels of 35% or significantly lower occur. This drives to the formation of moisture profiles in the timber components and subsequently cause tensions. In particular shrinkage stresses lead by the low maximum breaking strain of wood to a failure. As a result, cracks in wood but also in the bonding line occur. The tension influencing factor are

a) the structure of the material. Layer thickness, properties of the layers (type of wood, coating), moisture difference of the layers during bonding, arrangement of annual rings, adhesive type, adhesion quality, surface treatment
b) the ambient climate (relative humidity, temperature, air circulation).

The tensions resulting from changing climate conditions can be measured (free cutting, analogous to wood drying) and also calculated. Comprehensive material characteristics are required for computation (e.g., wood, adhesive, coating material). Relatively few research approache exist about the aging of the bonded components and there is considerable more research needed. Aging effect can occur quite soon after the wood-adhesive-composite is built in. In case of parquet, delamination can take place only after years. In case of cross laminated timber, cracking and delamination are often recognizable after just a few months. The damage rarely leads to a reduction in the load-bearing capacity of glued laminated timber and cross-laminated timber. In cross-laminated lumber or multi-ply solid wood panels (see Figure 1), the risk of cracking is higher than in glulam. This is due to the considerable orthotropy in the swelling and shrinkage behavior of the wood. Technological (moisture content during production, pressure during pressing, wood elaboration) and structural influencing factors (lamella properties, lamella thickness ratio) are presented as well as selected cases of damages and positive examples.


a) shrinking cracks in solid wood panel (middle layer)   b) delamination in glulam

Fig 1: Moisture induced cracks and delamination in glued wooden elements under dry conditions

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ISBN: 978-91-88898-64-7

A review of computational methods to describe the strength and failure behavior of wood and wood-based products and their embedment into a holistic design approach

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Increased use of wood has led to complex timber constructions and new types of wood-based products. In simulations, however, mainly simplified models are used to describe this material with strongly varying properties and a complex mechanical behavior. Wood, as a naturally-grown material, exhibits a highly anisotropic and inhomogeneous material structure, with a complex wood fiber distribution influenced by randomly occurring knots. Thus, for the prediction of effective strength properties of wood, advanced computational tools are required, which are able to predict as well as consider multidimensional strength information at different scales of observation.

Therefore, we developed a multi-surface failure criterion [1], which is able to describe brittle and ductile failure mechanisms of wood, based on simulations on several length scales. Combined with a geometric reconstruction algorithm for knots [2], such a tool can be used to determine effective strength properties of knot sections. Due to the highly orthotropic failure behavior of wood and the strong variations of material directions close to knots, this task is very challenging.

The extended finite element method is a powerful technique that allows for a very realistic description of strength-governing processes. Nevertheless, its complexity and high computational effort prevent widespread use in the engineering field, and it is limited by frequently occurring geometric incompatibility issues. Plastic limit analysis and elastic limit approaches [3,4], however, show good predictive performance compared with the extended finite element method, coupled with excellent efficiency and stability. In this study, it is found that together, the latter two approaches are able to enclose the experimentally-obtained failure regions for clear wood almost perfectly, while also delivering new insights with respect to the ductile failure potential of wood and wood-based products. Furthermore, the – in this field – relatively new phase field method may represent a solution to often encountered problems with respect to stability, reliability, and description accuracy.

Finally, strength properties of wooden boards are condensed into so-called strength profiles. By applying this approach to a large set of wooden boards, probabilistic material models can be developed and used in simulations of wood-based products [5]. In such a framework, sensitivity analysis and robust design optimization becomes possible and could lay the basis for a more holistic and, thus, also efficient design of timber structures.

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ISBN: 978-91-88898-64-7

Experimental evaluation of fracture properties and cohesion law of wood-adhesive bond-line in mode II using end-notched flexure

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The use of European beech (*Fagus sylvatica* L.) has a high potential in timber construction as it represents a considerably large growing stock in European forests. In general, the bondability of European beech compared to Norway spruce (*Picea abies* L.) can be more complex due to its structure and material characteristics. To increase its usability as a construction material, the characterization of bonding properties is of fundamental importance. Therefore, this study aims to evaluate these bonding properties through fracture analysis of the adhesive bond-line system in mode II using several different construction adhesives: emulsion polymer isocyanate (Epi), phenol-resorcinol formaldehyde (Prf), melamin-urea formaldehyde (Mu), and polyurethane (Pur). This analysis will provide material data to build a finite element (FE) model of a wood adhesive bond-line loaded in shear mode. After conditioning, clear beech boards were planned. Waxed paper was inserted between two boards to reduce the friction in the crack along the fiber direction and to create an initial crack of 182 mm [1]. The quantity of adhesive and bonding pressure applied on the beech boards was defined according to the manufacturers' recommendations. After curing, base samples were cut to the final dimensions of b × h × l = 18 × 20 × 500 mm. A three-point end-notched flexure (3ENF) test in mode II was performed with all the specimens. A universal testing machine (Zwick/Roell Z050) equipped with 50 kN load cell was used. Position control was used at a constant rate of 5 mm/min. To derive the cohesive law model for particular bond line systems, surface displacements must be measured [2]. To achieve this a stochastic pattern was applied to the specimen surface for optical measurement of displacements using the digital image correlation (DIC) technique. Displacements were measured using a pair of 23 mm lenses and Aramis GOM system. Images were captured once per second until ultimate failure. The force-deflection data were then used to calculate the strain energy release rate (G_{II}) [3]. The optically measured displacement slip together with the G_{II}, enabled the development of the cohesive law model for all adhesive-wood systems.

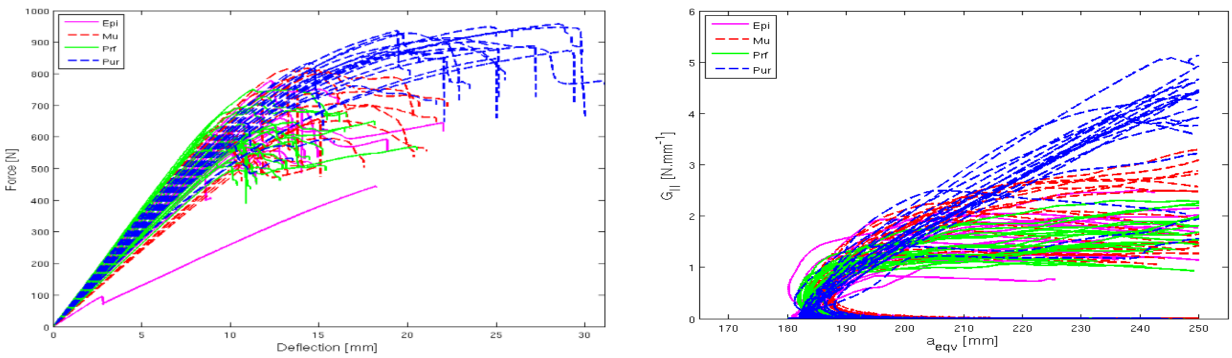

Figure 1: Force-deflection diagram (left) and strain energy release rate in dependence of equivalent crack length (right)

Preliminary results show the highest strength achieved with the Pur adhesive and the lowest with the epoxy adhesive (Fig. 1, left). The Pur adhesive exhibited a high portion of plasticity until it reached the ultimate load. The other adhesives exhibited more brittle behavior than Pur. As seen in Fig.1, the wood-adhesive systems show high variation in strength, stiffness, and consequently in G_{II}. The variability of test results is accounted for by the natural variability of wood, variability in adhesion quality, and adhesive surface coverage for individual specimens after the solidification. G_{II} values are highly dependent upon researcher routines in data processing and identification of crack initialization.

Acknowledgments
The authors gratefully acknowledge the European Commission for funding the InnoRenew CoE project (Grant Agreement #739574) under the H2020 Widespread-Teaming Programme and the Republic of Slovenia (Investment funding of the Republic of Slovenia and the European Union of the European Regional Development Fund).

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ISBN: 978-91-88898-64-7

Numerical modeling of wood-adhesive bond-line in mode II for beech wood glued by various adhesives

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Bond-line creates an interface between two glued surfaces and, therefore, it brings additional complexity into the mechanical behavior of glued components, especially around the bond-line region, because adhesive itself has a very different response to mechanical stress than wood. From this perspective, the bond-line influences the total mechanical response of glued components by both its cohesive and adhesive behavior at the wood-adhesive interface. For timber constructions, there are many adhesives one can use, and each of them has different mechanical characteristics, advantages, and disadvantages.

The goal of this work was to create numerical finite element (FE) models applicable for analysis of fracture problems in mode II in constructional elements. The models were developed for the various adhesives (PUR, EPI, MU, PRF) that are often applied in timber constructions and wooden materials. The FE models include 2D geometry of the bond-line and cohesive law fitted on the outputs of the experimental measurement. The experimental data for developing the numerical models were obtained using three-point end-notched flexure (3ENF) tests with the compliance-based beam method (CBBM), coupled with digital image correlation in order to obtain displacement slip needed for the development of the FE models [1, 2]. Furthermore, within the FE analysis, wood was modeled as an orthotropic material, including both elastic and plastic regions of deformation [3]. The FE models were developed in an Ansys computational system. The specific objectives of the work were to: 1) create cohesive zone models based on experimental data; 2) develop a parametric 2D model of the bond-line reflecting experimental data; 3) validate the FE models based on the experiments; 4) exploit the FE model in analysis of friction in the 3ENF setup; and 5) analyze plastic imprint into the specimens for European beech and Norway spruce.

Implementation of the cohesive law models of various wood-adhesive systems into the FE analysis was successful. The FE analysis provided a force-deflection response that was validated by experiments. The output of the analysis in terms of von Mises stress is shown in Fig. 1a. Despite it is scalar stress, it represents the location where the stress tensor experiences the highest intensity in total. The FE model showed that the influence of friction on the simulated force may be up to 5% of the maximal force, which in some cases cannot be neglected. The effect of friction may be seen via contact stress in Fig. 1b. The FE models further showed that higher imprint into the specimens from the supports and loading head is higher for the spruce wood, which should be taken into an account during the measurement. The imprint also depends on a size of the specimen, the slenderness ratio eventually.

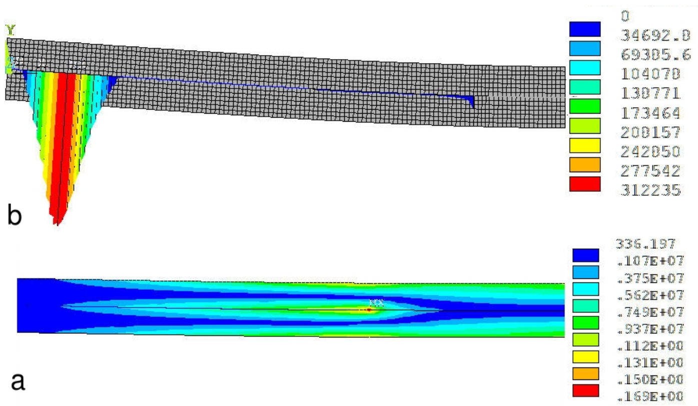

Figure 1: Two-dimensional FE model of the 3ENF: a) von Mises stress [Pa], b) contact stress [Pa].

Acknowledgement: European Commission for funding the InnoRenew CoE project under the Horizon2020 Widespread-Teaming program [Grant Agreement #739574].

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Strength and fracture analysis of shear mode III in cross laminated timber

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At in-plane shear loading of cross laminated timber (CLT), three failure modes (FM) are generally considered in design: gross shear failure (FM I), net shear failure (FM II) and shear failure in the crossing areas (FM III). FM III relates to shear stresses acting in the crossing areas between flat-wise bonded laminations, belonging to adjacent layers and being oriented orthogonally to one another. Relative translations and relative rotation between the two bonded laminations give rise to shear stresses acting in the plane of the bonded area, i.e. the *xy*-plane according to Figure 1.

For design with respect to FM III, considering relative translation and rotation, the following stress interaction failure criteria have been proposed in e.g. [1] and [2]

$$\frac{\tau_{tor}}{f_{v,tor}} + \frac{\tau_{zx}}{f_{v,R}} \leq 1.0 \quad \text{and} \quad \frac{\tau_{tor}}{f_{v,tor}} + \frac{\tau_{zy}}{f_{v,R}} \leq 1.0 \qquad \text{(1a), (1b)}$$

where τ_{tor} is a torsional shear stress due to the torsional moment M_{tor} while τ_{zx} and τ_{zy} are the shear stresses due to shear forces F_x and F_y, respectively, see Figure 1. The torsional shear stress τ_{tor} is assumed to be equal to the maximum shear stress at the mid-points of the edges of the crossing area, as calculated from linear elastic theory using the polar moment of inertia of the bonded area. The corresponding shear strength, $f_{v,tor}$, refers to the strength at pure torsional loading. Shear stress components τ_{zx} and τ_{zy} are in design commonly assumed to be uniformly distributed and the corresponding strength value, $f_{v,R}$, refers to the rolling shear strength of the laminations.

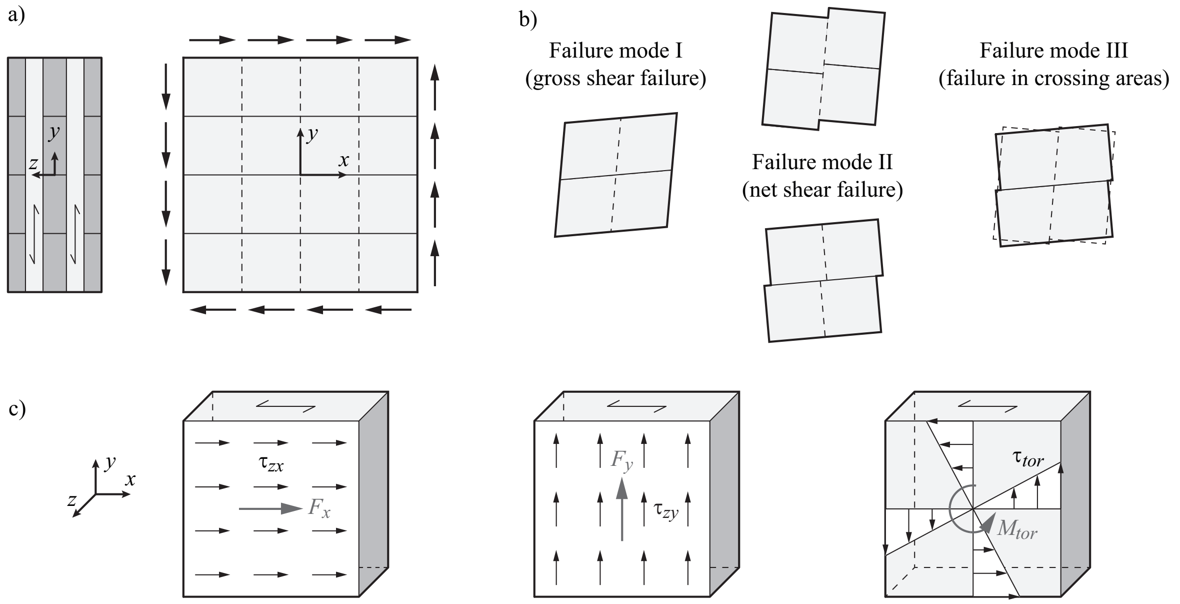

Figure 1: Illustration of a) cross laminated timber at in-plane shear loading b) corresponding shear failure modes I, II and III and c) decomposition of shear stress components τ_{zx}, τ_{zy} and τ_{tor} and their assumed distributions within a crossing area.

3D non-linear finite element (FE) analyses have been performed in order to investigate the strength and fracture course for shear failure mode III, as proposed in [3]. A single node, consisting of two orthogonally bonded laminations, was modelled. The bonding between the laminations was modelled using a cohesive zone approach, including strain softening behaviour after reaching the local material strength. Loading conditions representing pure shear force, pure torsion and mixed modes were considered and the global load-bearing capacity and the fracture course was studied. The presentation discusses the results of the FE-analyses in relation to experimental test results, considering the failure criteria in Equations (1a) and (1b).

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 ISBN: 978-91-88898-64-7

An experimental and numerical investigation of fracture characteristics of acetylated Scots Pine

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Modification methods have proven successful in limiting the hygroscopic nature of wood, and are less harmful to the environment than toxic preservatives. Acetylation is a modification method based on a chemical reaction between acetic anhydride and the cell wall polymers. Extensive research over the past decades has demonstrated its success in improving both dimensional stability and durability of various wood species. The change of chemical composition of the cell wall affects mechanical properties as well. Numerous studies of basic mechanical properties, e.g. bending stiffness and strength, have been conducted [1]. However, only a few studies concerning the influence of acetylation on fracture characteristics have been performed. A study conducted in 2002 [2] indicates a significant decrease of both the fracture energy and the critical stress intensity factor for acetylated spruce. In structural applications fracture characteristics are of importance due to the occurrence of holes, notches, moisture gradients etc., inducing large stress concentrations which may lead to crack propagation. In designing mechanical joints, they are decisive for avoiding brittle failure modes [3].

Experimental tests on fracture characteristics were carried out at Lund University, during the fall 2018. The fracture energy was determined for mode I in tension perpendicular to the grain. Single edge notched three-point-bending (SEN-TPB) tests were performed according to the test standard described in NT-BUILD 422 [4]. Scots Pine (*Pinus Sylvestris*) originating from young stands was examined and evaluated based on four test groups: unmodified sapwood; acetylated sapwood; unmodified heartwood; acetylated heartwood. The results demonstrate a significant decrease of the fracture energy for acetylated specimens, where the mean value decreased with approximately 50% for sapwood and 30% for heartwood. Additionally, the modulus of elasticity in compression parallel to the grain and tensile strength perpendicular to the grain were examined. No significant influence related to these properties could be concluded.

The SEN-TPB tests will be further examined through finite element analyses. The nonlinear fracture behavior will be modelled by a fictitious crack approach. This will be done by introducing a cohesive zone along a predefined crack path, aligned with the initial notch. In order to calibrate proper material models, different strain-softening properties will be evaluated in relation to experimentally achieved load-displacement curves. The aim is to establish appropriate material models to, further on, be able to implement these in the analysis of structural elements and evaluation of joints. Additionally, finite element analyses will be used to estimate the critical stress intensity factor, i.e. evaluate the effects of acetylation on notch sensitivity.

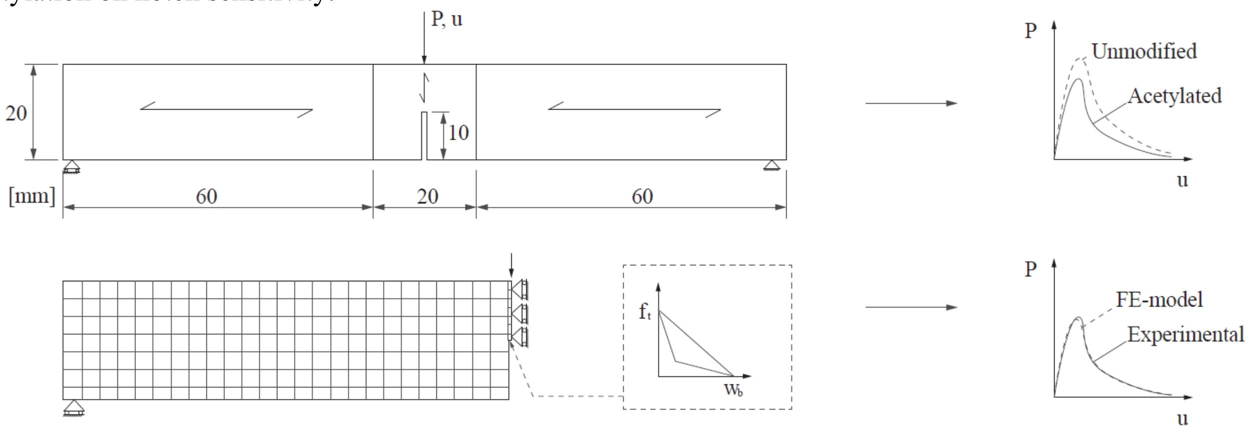

Figure 1: Illustration of SEN-TPB test set-up (a), typical load-displacement curves from experiments (b), FE-model with strain-softening properties for the cohesive zone (c) and numerical versus experimentally found load-displacement curves (d).

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ISBN: 978-91-88898-64-7

Fiber orientation – modeling and grading of wood

ISBN: 978-91-88898-64-7

Assessment of the error of fiber orientation measurement obtained by laser scanning on several European hardwood and softwood species

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Scanner measurements allow knot detection and machine strength grading on sawn timber, which answers the construction and sawmilling industry needs at high production rates. In particular, laser scanners measure fiber orientation by means of the so-called tracheid effect explained by the fact that the wood fibers conduct concentrated light better in the direction of fibers than across. Schlotzhauer *et al.* [1] showed that a red laser scanner (660 nm, 100 mW) is very effective on softwood species like spruce but ineffective on many hardwood species like oak. However, the LaBoMaP has recently developed a fiber orientation scanner effective on hardwood species, especially oak [2]. It is, to the best of authors' knowledge, the first to use a laser wavelength of 1064 nm and a modulable power (up to 1 W). Assessing the fiber orientation measurement error is important for strength grading purposes. Hu *et al.* [3] did this work for sugi and Japanese beech species. To the best of authors' knowledge, there is no such study focusing on European species as oak, beech, Douglas fir, spruce, etc. Furthermore, [3] and other similar studies focused only on the angle of the major axis of the elliptically scattered laser light, while other geometric parameters as the size or minor to major axis ratio of this ellipse are interesting to study. They also did not discuss the intensity of the tracheid effect between species, nor study the influence of laser power.

In the present work, not only the fiber orientation measurement error is assessed on several European hardwood and softwood species, but a method to measure the intensity of the tracheid effect according to the species and wood cutting plane (radial-longitudinal or tangential-longitudinal) is also proposed, and several underlying artefacts of the tracheid effect are discussed. To achieve this, clear wood test pieces were first scanned at a 0° orientation (figure 1a). Paying attention to the power of the lasers to be able to make comparison between species, ellipse geometrical parameters were compared to that obtained on an isotropic surface. In a second time, test pieces were scanned at different orientations from 0° to 90°. The measurement error was defined as the difference of mean scanned fiber angle on each iteration and test piece angle between these same iterations.

Results showed that fiber orientation measurement was possible on hardwood species with the LaBoMaP laser scanner. Indeed, ellipses axis ratio observed were distinct enough from that of an isotropic surface. However, for certain species like oak the axis ratio can be highly affected by the parenchymal cells when being in the radial-longitudinal plane (figure 1c). The second part of the experiment showed that measurement error can be less than one degree. The comparisons of the results between species and cutting planes will be presented at the conference.

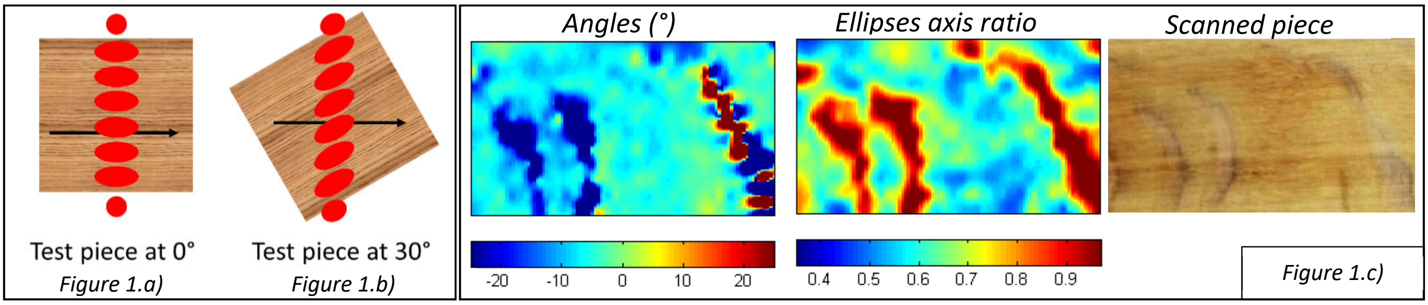

Figure 1: a) & b) laser scanning on the same piece at 0° and 30°. c) Example of angles and axis ratio maps for oak quarter sawn

Acknowledgement

The authors thank the regional council of Bourgogne Franche-Comté and Ducerf Groupe.

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ISBN: 978-91-88898-64-7

Prediction of tensile strength in sawn timber by means of surface laser scanning and dynamic excitation

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Machine strength grading of sawn timber is based on the relationship between so-called *indicating properties* (IPs) and *grade determining properties* (GDPs). The former are calculated using board properties measured non-destructively whereas the latter are determined by destructive tests. For T-classes, which are used for glulam lamellae, the GDPs are tensile strength ($f_{t,0}$), *modulus of elasticity* (MOE) in tension and density.

The aims of this study were to develop an IP, similar to the one given in [1], for prediction of tensile strength and to calculate yield in different T-classes using this IP. Non-destructive and destructive measurements were made on a total number of 967 boards of Norway spruce with varying dimensions originating from Finland, Norway and Sweden. The non-destructive measurements included in this study were surface laser scanning, X-ray scanning and dynamic excitation, and were carried out both before and after planing of the boards. Results from X-ray scanning were used to calculated board density whereas surface laser scanning and dynamic excitation were used to determine in-plane fibre directions at longitudinal surfaces and axial resonance frequency, respectively. The destructive tests were made after planing.

The IP used for prediction of $f_{t,0}$, herein denoted $IP_{E,b}$, was based on a local MOE calculated by means of observed fibre directions and dynamic MOE (E_{dyn}). Coefficients of determination between $f_{t,0}$ and $IP_{E,b}$ of 0.65 and 0.66 (linear regression) were obtained using measurement results before and after planing, respectively, see Figures 1a and 1b. Applying E_{dyn}, which is used by several grading machine as IP for prediction of $f_{t,0}$, resulted in a coefficient of determination (r^2) of 0.46 for $f_{t,0}$, both before and after planing.

Table 1 gives calculated yield in three different strength classes using the suggested grading method applying $IP_{E,b}$ for prediction of $f_{t,0}$. This table also includes the calculated yield for a grading machine applying E_{dyn} for prediction of $f_{t,0}$. The yield obtained using $IP_{E,b}$ rather than E_{dyn} for prediction of $f_{t,0}$ is much higher, particularly for high strength classes.

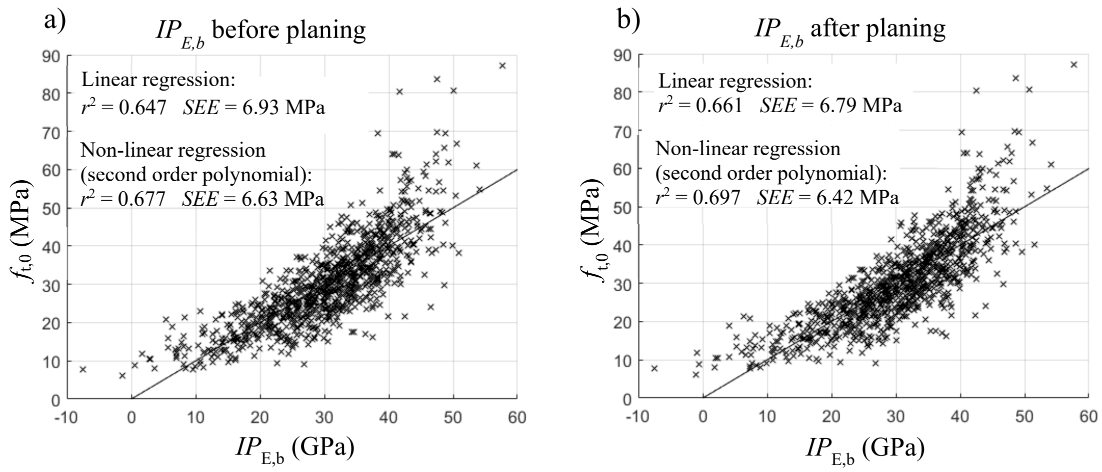

Figure 1: Scatters between $f_{t,0}$ and $IP_{E,b}$. Included in each scatter are r^2 and *standard error of estimate* (SEE).

Table 1 : Yield in strength classes (single grade).

Grade	Suggested grading method (before planing)	Suggested grading method (after planing)	Grading method applying dynamic MOE for prediction of strength
T15	95.3 %	96.3 %	95.5 %
T22	60.9 %	59.0 %	42.9 %
T26	33.4 %	33.5 %	15.9 %

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ISBN: 978-91-88898-64-7

Determination of Global Modulus of Elasticity of Timber by Using Fiber Orientation and Proportion of Latewood

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Mechanical properties of timber depend on the fiber orientation, the percentage of latewood, the distribution of knot, etc. [1-4]. However, the fiber orientation and the modulus of elasticity (MOE) may be affected by the presence of knots. Therefore, the tracheid effect and proportion of latewood were used in this paper to measure the fiber orientation and MOE of timber, respectively. Since Japanese Cedar (*Cryptomeria Japonica*) is a popular species in both Taiwan and Japan, its timber was used as the test specimen.

By using the tracheid effect, the fiber orientation can be determined from the ellipse formed by the scattering pattern [5]. The in-plane fiber orientation can be determined by the orientation of the ellipse. On the other hand, the out-of-plane fiber orientation angle can be determined from the ratio of the major and minor axis of the ellipse.

In the past, majority of research work used the statistical average material properties obtained from a number of experiments. Since wood is a composite material in nature, based on the rule of mixtures [6], material properties of timber can be calculated by the proportion of latewood and MOEs of earlywood and latewood. Digital image analysis (DIA) technique first reported in [2] was employed to determine the proportion of latewood and earlywood of the radial cross-section.

Functions of digital image processing of MATLAB [7] were employed to determine the major axis, the minor axis, and the orientation of the ellipse formed by the scattering pattern. The stiffness matrix was calculated by the compliance matrix and the fiber orientation matrix. The fiber orientation matrix was combined with the compliance matrix to calculate the global MOE [8]. A four-point bending test was also conducted in this paper to measure the global MOE and the three-dimensional digital image correlation (DIC) method software, VIC-3D [9], was used to analyze the surface deformation during the experiment.

Based on the tracheid effect, an optical setup was proposed in this paper to analyze the ellipse formed by the scattering pattern to determine in-plane and out-of-plane fiber orientation angles. Moreover, the digital image processing technique was adopted to measure the proportion of latewood and earlywood of the radial cross-section. Finally, the global MOE of timber obtained from calculation and experiment were compared and discussed.

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ISBN: 978-91-88898-64-7

Automatic detection of pith location along boards of Norway spruce on the basis of data from optical scanning of longitudinal surfaces

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Different mechanical and physical properties of wood are related to the location of pith. Norway spruce wood from the centre of logs, close to the pith, is characterized by lower longitudinal MOE, larger spiral grain angle, and larger longitudinal shrinkage coefficient than what wood farther away from the pith is [1]. Thus, knowledge of pith location along timber boards may play an important role in both appearance grading and in assessment of mechanical properties such as strength [2]. The current work aims to develop an algorithm which is capable of automatically estimating the pith location of Norway spruce boards, along the boards' length direction, by utilizing optical scanning of longitudinal surfaces. The initial step of the algorithm is to identify defect free sections along the timber board. This is done by utilizing data from tracheid effect scanning of the four sides of the timber board. Thereafter, a continuous wavelet transform (CWT), similar to fast Fourier transform, is applied on grey scale images from scanning, to analyse the variation of light intensity across the four surfaces at selected positions along the board. Obtained local frequencies correspond to the local annular ring pattern on surfaces. Then, assuming that annular growth rings are concentric circles with the pith in the centre, *detected* local annular ring wavelengths (using CWT) and *artificial* annual ring wavelengths corresponding to different hypothetical locations of pith are compared, and an optimization procedure is used to identify the location of pith that *minimizes* the discrepancy between the detected and artificial sets of annular ring wavelengths. Figure 1 shows grey scale images of short segments of longitudinal surfaces, graphs of the detected local annual ring widths, and a photograph of the board cross section where the determined location of pith is marked out. Preliminary results reveal that data from optical scanners and the suggested method allow for accurate detection of annular ring width and location of pith along boards.

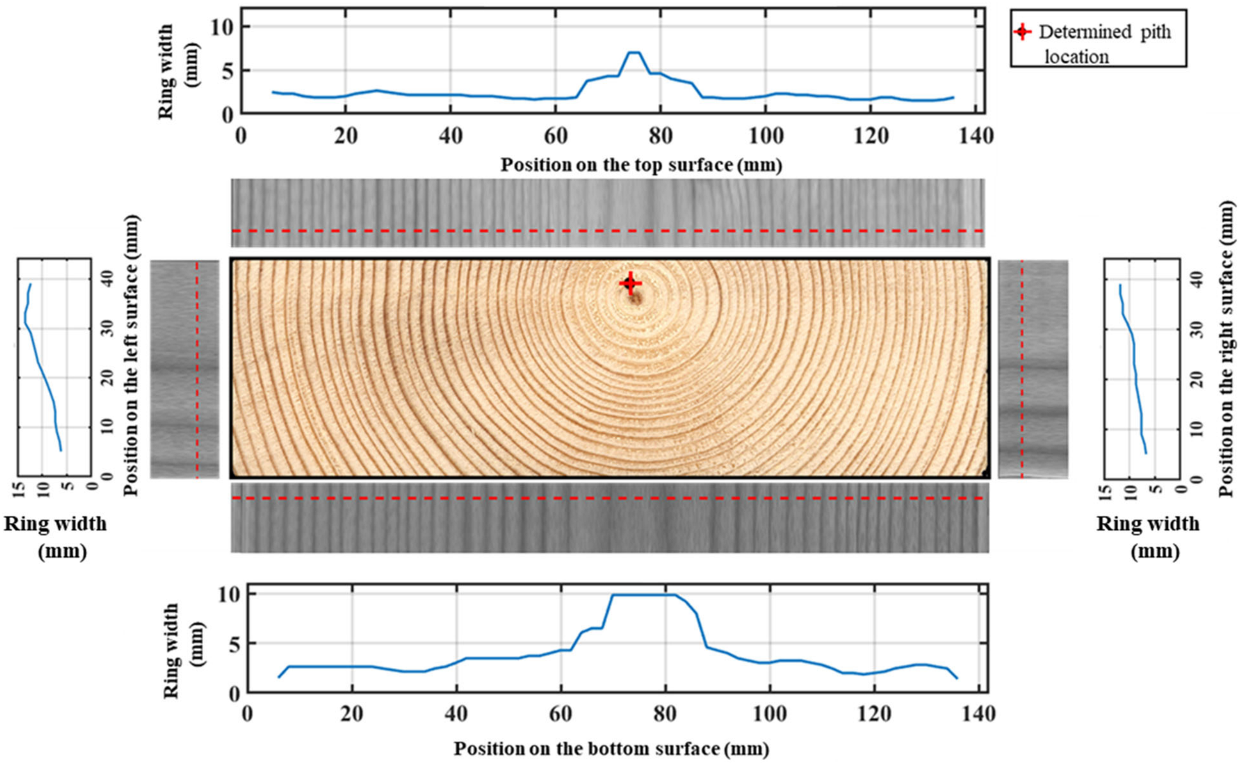

Figure 1: Determined pith location based on detected surface annual ring pattern (wavelength)

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 ISBN: 978-91-88898-64-7

Modeling fiber direction around knots in structural timber

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One of the most important properties that determine strength and stiffness of structural timber is local fiber direction. Industry scanners, which are nowadays commonly used in sawmills, are able to sample such data of timber surfaces in a resolution of a couple of millimeters. In the last years a comparably accurate grading method, which relies on knowledge of fiber direction within timber boards, was developed [1]. However, in this method interior fiber directions are determined in a rather unsophisticated way, not taking location of pith, and morphology and orientation of knots into account. Attempts to improve the modelling of interior fiber direction have been made [2, 3], but none of these utilize an optimized *combination* of fibre directions from scanning and mathematical modelling. Nor were these models based on accurate models of morphology of knots. The aim of the present research is to develop a 3D fiber angle model for timber boards, which utilizes fiber data of surface laser scanning, knowledge of pith of log and 3D reconstruction of knots within timber boards. The purpose is to determine fiber directions within timber volume more accurately, which can in turn contribute to the development of machine strength grading methods more accurate than those available today.

For the modeling of fiber direction, a 3D fiber model valid for a single knot [4] is employed and extended to be applicable for full-size timber boards including multiple knots that are interactive on determination of fiber directions. For information of the pith location, morphology and orientation of knots, a timber board of Norway spruce in dimension 64 × 190 × 3400 mm, was thoroughly investigated in laboratory by repeatedly alternate from laser scanning using a WoodEye 5 scanner and planing by a thickness planer. In such a way, in-plane fiber direction of every 2 mm and surface images of every 1 mm in the board thickness direction were obtained. This data was also used for calibration and verification of the proposed fiber angle model.

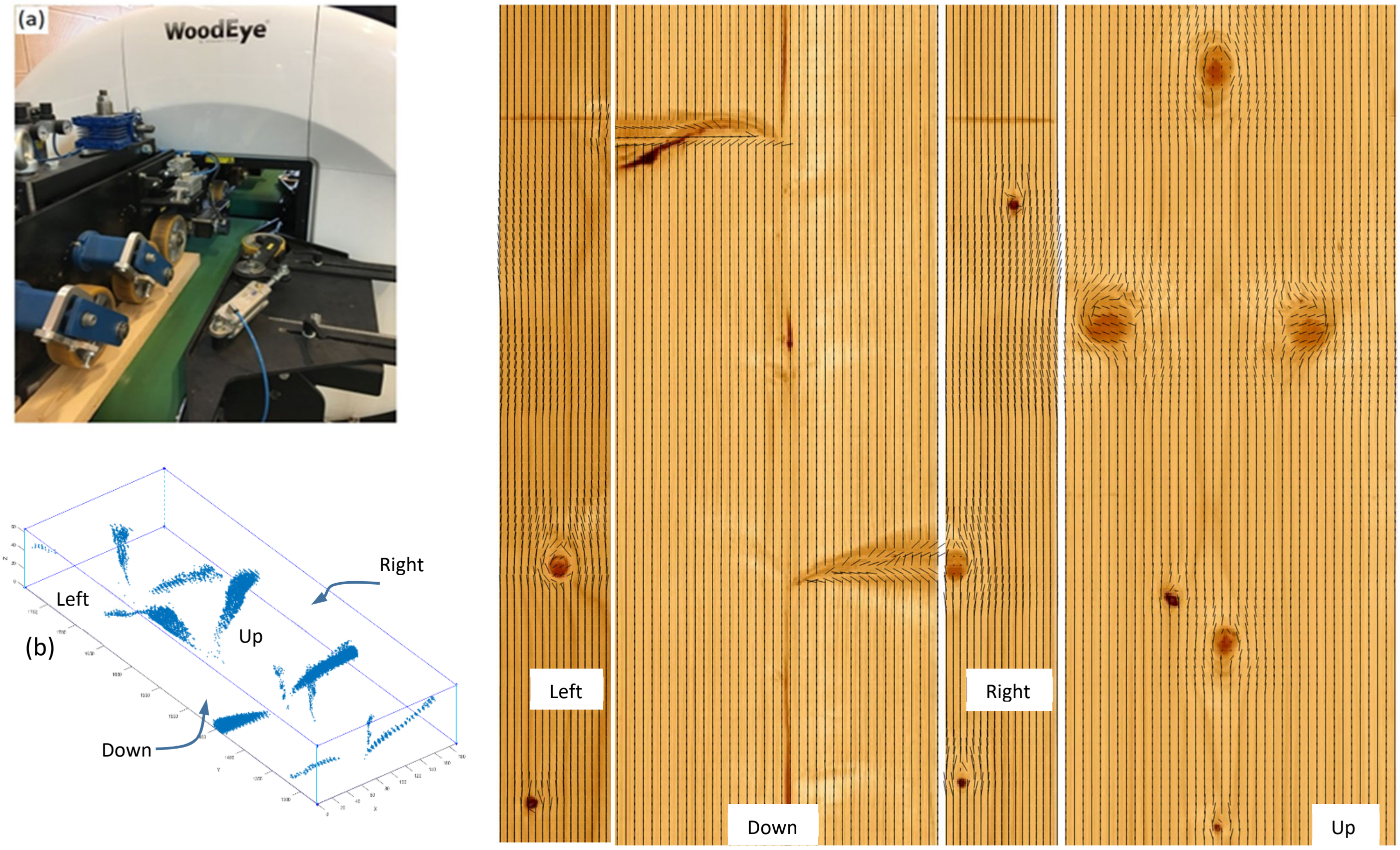

Figure 1: (a) A WoodEye 5 scanner. (b) 3D reconstruction of knots and (c) Modelling results of fibre direction on timber surfaces.

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Investigation of density variations in molded wood tubes using gamma-ray CT and correlation with load-bearing behavior

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It is well known that mechanical properties of wood correlate with the density [1]. Since wood is a naturally grown material, variations in the density distribution still exist in timber elements leading to a non-uniform distribution of mechanical properties. To investigate the density distribution in timber elements at the meter scale non-destructively, the gamma-ray computed tomography (CT) scanner, firstly introduced in 2007 by Hampel et al [2], has been applied. The CT scanner offers a spatial resolution of about 1-2 mm. Nevertheless, small single structures like cracks or branches can be revealed up to a size of several micrometers.

As object of interest, a molded wooden tube (MWT) [2] with a length of 3 m and a diameter of 0.3 m made of beech (*Fagus sylvatica*) is used. The MWT is produced in a thermo-hydro-mechanical process incorporating densification and recovery of wood transverse to the grain [3]. Thus, besides naturally grown density variations also variations due to the production process of the MWT occur. Figure 1 (left) shows the density distribution in the MWT, measured plane-wisely along the MWT in discrete length distances of 50 mm.

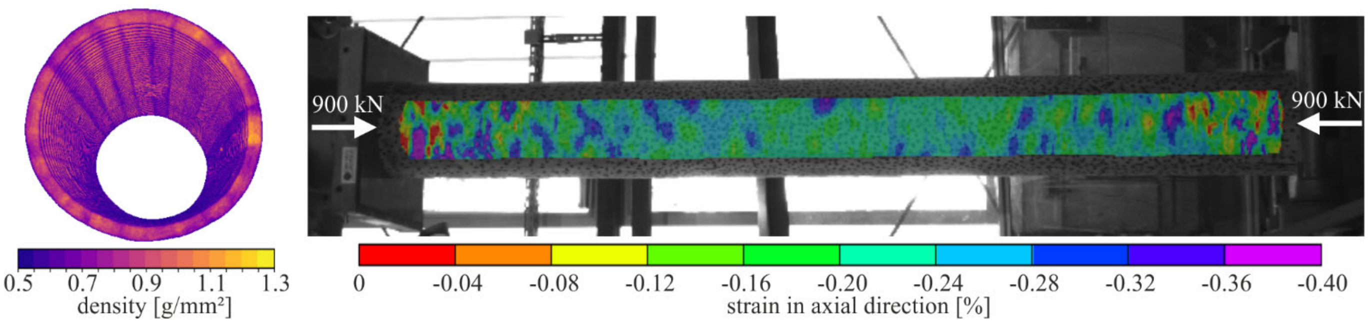

Figure 1: Experimental results. (left): density distribution determined by CT in the MWT (not to scale), (right) axial strain distribution in the MWT at an axial loading of 900 kN

In order to verify the assumption that the mechanical properties correlate with the density, an axial compression test is performed with the MWT previously scanned with CT. The spatial deformations on the surface of the MWT were measured by photogrammetry and digital image correlation (DIC) is applied to determine the strain distribution, see Figure 1 (right).

The density and geometry data gathered by CT is also used to create a finite element (FE) model. Based on the density data, the elastic properties of the respective elements are defined. The axial compression test is simulated and the results in terms of the strain distributions are compared to the experimental data determined by DIC.

The results of the investigations showed that computed tomography is highly suitable for the non-destructive determination of the density distribution in structural elements of timber. Thus, besides for research purposes CT scanning might be used also in the future for industrial grading of timber elements.

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ISBN: 978-91-88898-64-7

Computational analysis of timber structures

ISBN: 978-91-88898-64-7

A critical discussion on the application of the Finite Element Method in design and verification of timber structures

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Structural models based on Finite Element Method are widely established in engineering practice e.g. for the determination of the distribution of forces and stresses in and deformations of structures or for evaluating their dynamic response. Some of the reasons for choosing FEM instead of hand-calculation are the easy possibility to evaluate statically indeterminate structures, structures of complex geometry (e.g. Free-Form structures) and those deviating from beam theory (glulam beams of small length to height ratio), or plate and shell type elements (e.g. CLT panels with large openings, windows and doors). For the realistic representation of timber structures by means of models, the particular characteristics of wood and the material behaviour of timber pose a challenge to the engineer.

Traditionally the design of timber structures according to EN 1995-1-1 (EC5) [1] is based on an element by element approach, in which the individual elements often can be verified by hand-calculation. An example is the single supported beam loaded primarily in bending: In the verification the linear distribution of stresses according to linear-elastic beam theory is used; the characteristic values of bending strength given in the material product standards associated to EC5 are based on simple bending tests of beams with certain geometry as specified in the related test standards; the related partial safety factors for the ultimate limit state (ULS) of a beam in bending were calibrated with the two above assumptions. When deviating from these assumptions and prerequisites the design rules and recommendations given in EC5 and related standards do not apply any more. This can be illustrated with the example of a beam with shorter span and greater height: the highly non-linear distribution of shear stresses in the region of beam supports and the interaction of shear and compression perpendicular to the grain stresses are not reflected in detail in the design according to EC5.

For the estimation of the expected deformation of timber structures by means of FEM the models can be based on the mean material stiffness properties that can be found to some extent in the product standards related to EC5. The model based on these mean values of properties can be used for the validation of the structural behaviour in the serviceability limit state. However, when it comes to the evaluation of the behaviour of structurally indeterminate timber structures in ULS, the variability of the material properties has to be accounted for. Information on the variability of the material strength and stiffness properties can be found in JCSS Probabilistic Model Code [2], detailed information on the distribution characteristics of stiffness and load-carrying capacity of connections is much harder to find in literature. System effects in structurally indeterminate timber structures with parallel or serial arrangement of elements make it impossible to simply rely on conservative, characteristic values but may require a probabilistic evaluation of the structural reliability.

In the contribution the various parameters and characteristics of timber and timber structures with relevance to the modelling by FEM for design and verification are discussed, amongst others the following:

- The relatively high variability in material strength and stiffness properties of timber together with the brittleness of failure mechanism in tension and shear may requires Monte-Carlo or other probabilistic methods in order to determine the relevant design situation.
- The non-linear load-deformation behaviour in connections requires good knowledge of the adequate analytical or tabulated representation of this behaviour. Especially modern, high performance fasteners and connections are not be represented adequately by the equations and recommendations given in EC5 for traditional connections. The potential load-redistribution between different fasteners and connections due to different ductility, stiffness and yield load requires variation of these parameters in the models.
- The size dependency of the material wood requires an exact evaluation of the stresses volume. Especially when being loaded in tension perpendicular to the grain and in shear the effective strength may be impacted by local defects in the stresses volume due to the weakest link effect.
- Stress singularities, that occur in the vicinity of cracks, cuts or abrupt changes in cross-section of elements (e.g. notches or holes), are highly dependent on the choice of element type and element size in the model. Material property values in product standards are commonly determined for situations without stress singularities.

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ISBN: 978-91-88898-64-7

Industrialization of the design and production process of wooden trusses

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Currently, laminated timber is widely used beside classic sawn timber [1]. The gluing allows for higher part length and involves an advantageous behavior regarding deformations due to shrinkage and lead to better, more regular mechanical properties [2]. The drawback is a low material utilization factor. Starting from a tree trunk, only 25-30 % are part of the final product. Thus, the high-quality product has to be used as efficient as possible.

At moment mostly, plate girders made of laminated timber are used as a result of the efficient manufacturing process. If in comparison a truss system is used, a similar load bearing capacity and stiffness can be achieved with much less material effort. The aim of the presented research concept is to industrialize the design and manufacturing process of timber truss systems to be able to compete with the common plate girder systems (see Figure 1). In detail the complete process starting from the design, static optimization, static proofs according to the standards, work preparation to production process will be cumulated in a continuous digital approach.

The stiffness of a truss system is directly linked to the stiffness of the truss joints. Thus, in a first step the load bearing behavior of different wood-wood connections was tested to determine the most efficient version for the approach. Besides traditional wood connections and engineer-type wood connections (gusset plate) we also tested completely new versions, which are enabled by digital fabrication tools. Beside recording the applied loads and chosen areas by linear variable displacement transformers (LDTV's) the strains at the specimen surface were supervised by using a digital image correlation system (DIC). An exemplary result is shown in Figure 2. In a second step first prototypes of full-scale trusses with a span of 4.8m were manufactured by using an industrial robot and the load bearing behavior was tested in destructive bending tests.

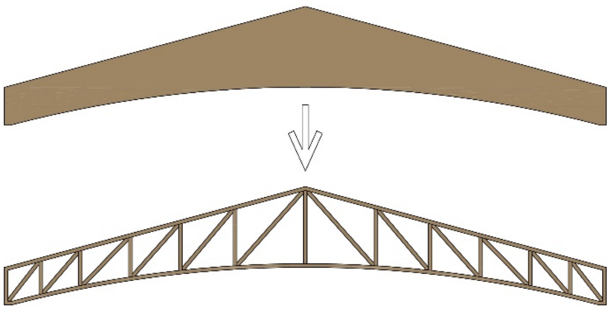

Figure 1: Concept drawing

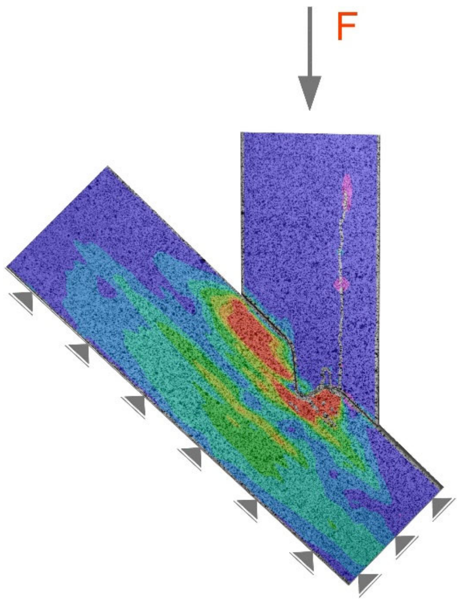

Figure 2: Exemplary results of a compressive test at a timber truss joint

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ISBN: 978-91-88898-64-7

Numerical modelling of light-frame timber walls with focus on out-of-plane deformations and elastic-plastic fastener force distribution

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Light-frame timber walls in volume modules act typically as shear walls to resist lateral wind loads. The load bearing structures of these type of walls consist of numerous slender timber elements (studs, rails, noggin pieces, door and window sills and different types of fibre- or plasterboards) that are connected together by a huge number of mechanical nail, staples or screw joints. The challenge is to model the different types of mechanical connections both properly and efficiently. The work presented here is a further development of the wall models presented in [1] and [2], where it is now focused on both elastic and plastic behaviours of the mechanical connections between the timber frame and the plasterboards. Since the plasterboards sit on the inside of the timber frame, the walls function as an unsymmetrical structure when they are loaded in an in-plane shear action.

To keep the size of the model as small as possible the FE-model is made of simple structural elements, i.e. straight beam elements for studs, rails and mechanical fasteners and planar shell elements for the plasterboards. To show better how the different elements are assembled and connected together Figure 1*(b)* shows the different structural elements used and symbols for spring connections of a typical timber frame wall.

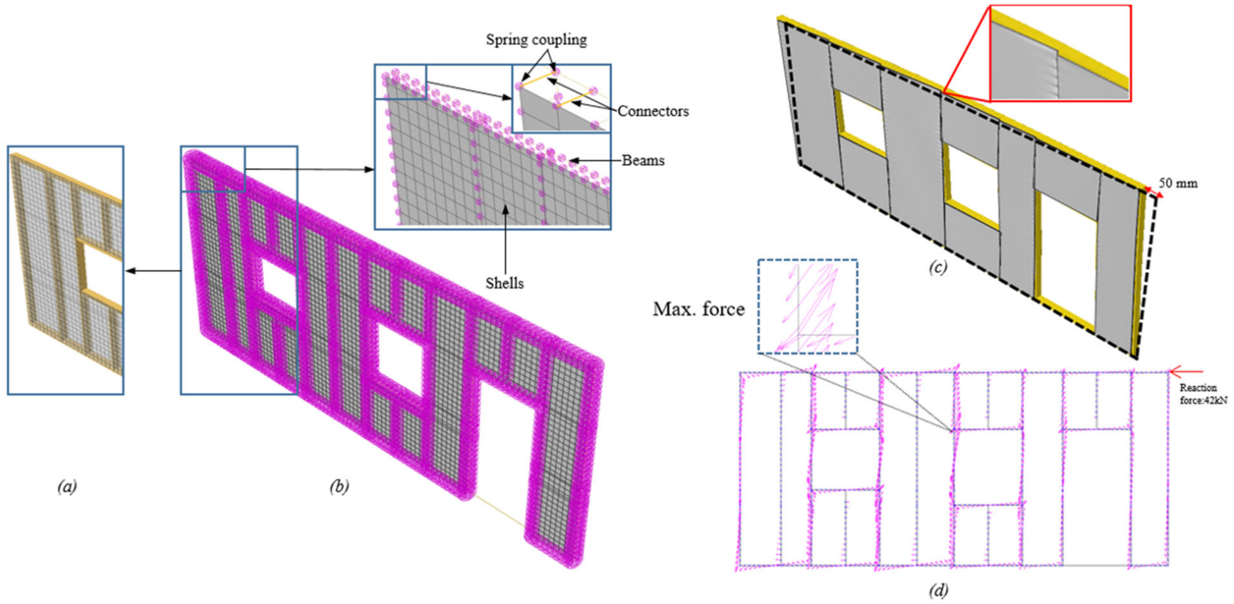

Figure1: Model illustration of a light-frame timber wall, *(a)* a 3D-view of a part of the wall, *(b)* element mesh and symbols used for spring coupling between the elements, *(c)* out-of-plane deformation and close-up on a slippage deformation between two plasterboards, *(d)* elastic fastener force distribution in the wall.

To illustrate the timber frame structure Figure 1*(a)* shows a 3D-view of a part of the wall element. Figure 1*(c)* shows clear out-of-plane bending of the wall element because of the eccentric force action in the element and a visible slippage deformation between two of the sheathing boards. Figure 1*(d)* shows the elastic fastener force distribution in the timber wall exposed to in-plane shear loading. The largest fastener forces seems to occur close to the corners of the largest plasterboards or close to window corners. These forces can be directly implemented in the design equations used for the individual shear connections. Based on reasonable input data the simulation model gives valuable information regarding both the global structural behaviour of the timber wall and the fastener forces acting in all mechanical connections. To facilitate the creation and assembly of all the timber components the model needs to be fully parameterized to increase its flexibility to analyse different types of timber walls.

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ISBN: 978-91-88898-64-7

Numerical analysis to study how out-of-plane imperfections affect the ultimate load bearing capacity of slender long span timber trusses

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Timber trusses are often used as main bearing elements in roof structures which span anywhere between 8 and 15 meters. During the last decades, there has been an increasing interest in pushing the limits for these structures and there are examples of roofs built with single spans well above 20 meters. In Sweden, the width of the cross-section used in timber trusses is typically 45 mm, implying such roofs to be very slender and an obvious risk for out-of-plane instability phenomena to occur. The desire of increased spans for the roof structure in combination with the fact that there have been several roof failures reported over the last decennium [1] calls for special attention into the topic. Several authors have made significant contributions on stability issues previously [2-4], and numerical simulations using 3D structural elements and eccentric element coupling has successively made it possible to include larger models thus avoiding difficulties associated with simplifications and limitations of the structural systems analyzed [5].

In Figure 1(a) a roof structure constructed with timber trusses is shown. Battens, top and bottom chords and tension strips are marked and stabilizing trusses installed in the plane of the top chords are numbered 1-4. The design of such stabilizing trusses may vary in several different ways and one type is shown in Figure 1(b) together with other trusses used for overall stability of the building. In the design of such a roof structure it is necessary to have a balance between the complexity of the calculation and the preciseness of the results. The desire of increasing spans implies that new models with sustained or increased preciseness as compared to those provided in EC5 today should be put forward.

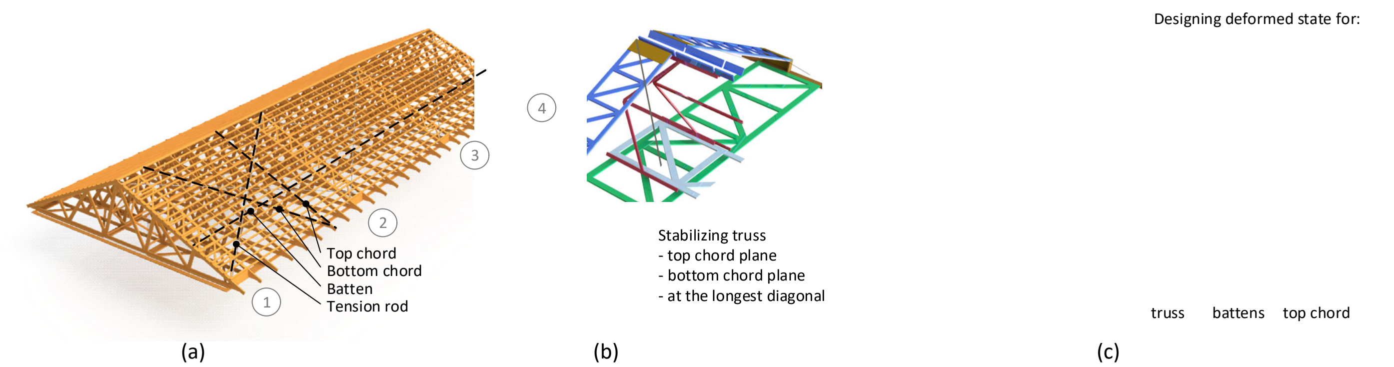

Figure 1: (a) Roof structure with 4 stabilizing trusses and steel strips in the plane of the top chords (b) four stability trusses in the roof system and (c) ultimate failure mode for the truss, the battens and the top chord respectively.

A geometrically non-linear parameterized finite element model is suggested for analysis of the critical loads for the top chords shown in Figure 1(c). The top chord is assumed to be laterally supported by discretely located elastic springs representing the stiffnesses provided by the stabilizing truss via the battens and the involved connections. Relevant imperfection modes are used to define the initial geometry of the structure. A parametric study is performed where the stiffness as well as distances between the lateral support springs are varied and effects of the load bearing capacity studied. Further on a parametric study on the influence of the magnitude of the initial deformation is performed.

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ISBN: 978-91-88898-64-7

Investigations on transversal load sharing in Timber-Concrete floors

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Despite timber-concrete slab-type structures are known and commonly used, some questions are not fully understood yet. Among them is the lateral load distribution in the composite floors if a load is applied to a part of the structure only (compare figure 1 left). By using finite element (FE) models, different parameters, like span of the slab, joist spacing, dimensions of beams, level of connection, load level as well as type and position of load, on the transversal load sharing were investigated to provide distribution factors for different load cases. The FE-model used was validated on basis of experimental investigations. They included push-out testing as well as short-time full-scale bending tests. The cluster of parameters included in the investigations was chosen in accordance to common dimensions of existing timber beam ceilings. In this paper 24 FE-models of timber-concrete composite (TCC) slabs with different geometrical dimensions are presented. The span of the beams was changed between 3 and 6 m, the timber beam distance between 0.5 and 1.0 m. The corresponding dimensions of the beams are based on the required dimensions for pure timber beam ceilings related to the presented span and interjoist of the beams. Among the described geometrical properties the height of screed (40 mm) and interlayer (20 mm) were kept constant. During the analysis applied forces as well as strain and deflections on bottom side of every beam (at midspan) were recorded.

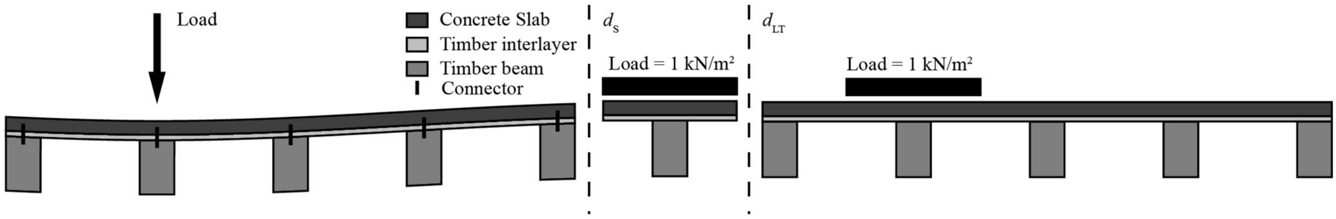

Figure 1: Principle of load sharing (left), Concentrated load applied to one beam of a slab-type system (middle), single composite beam (right)

In additional calculation only one single composite cross section with span l, beam distance a, timber beam width b and timber beam height h corresponding to the cluster of parameters, but without any bi-dimensional load transfer was considered. From that calculation the deflection without lateral load transfer (d_S – deflection single, figure 1 middle) was obtained. The deflections with lateral load transfer were received from the previously described FE-analysis of the same models (d_{LT} – deflection load transfer, figure 1 right). The relation between both deflections can be interpreted as a load reduction factor η for the loaded beam (compare equation (1)).

$$\eta = \frac{d_{LT}}{d_S} \tag{1}$$

Different geometrical properties of the timber beam ceiling have reasonable impact on the transversal load-bearing behaviour of TCC structures. Immanent impact factors are, for instance, the type of concrete topping, and the type of connection members. Loads in TCC structures are distributed more efficiently in the case of decreasing the beam spacing of the structure, and increasing of span of the system. In every considered model the deflections with consideration of load transfer were smaller than 60 % of those without consideration of load transfer. A reduction factor of 0.7 due to the deflection resulting from a concentrated loading of one composite beam is proposed for systems within the range of the parametric study.

Acknowledgement

The authors gratefully acknowledge the financial support provided by European Social Fund (ESF) and Free State of Saxony.

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ISBN: 978-91-88898-64-7

Structural behaviour of hybrid floor systems: cold-formed steel and sustainable floorboards

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Hybrid floor systems comprising steel beams and wood-based floorboards are increasingly being considered for sustainable construction [1]. To explore the capacity of such systems, a parameterised 3D finite element model is presented for analysing the performance of hybrid flooring systems comprising cold-formed steel and various engineered bio-based products under bending loads (Figure 1). Results are validated against experimental data available for flooring systems in the literature [1].

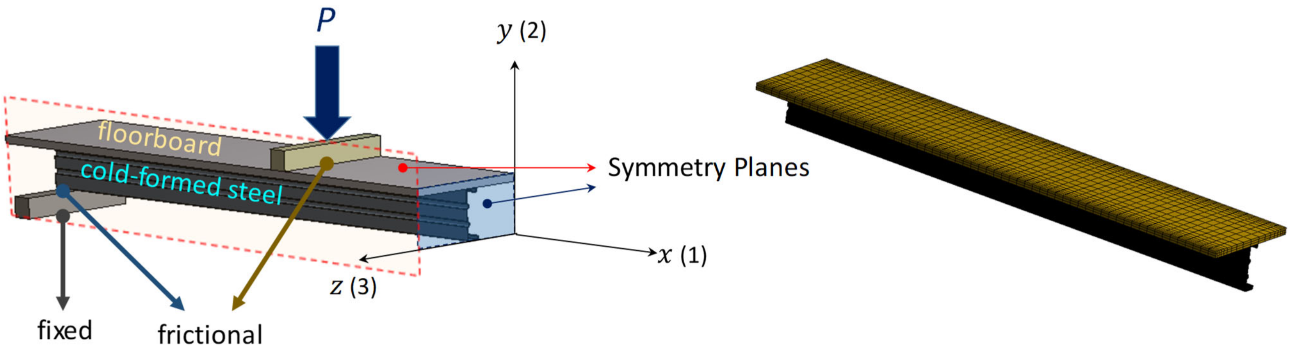

Figure 1: Composite floor system half-model under four point bending load. Symmetric boundary conditions have been applied to reduce the computational time and to increase the efficiency of the simulations.

The potential use of several engineered wood products including OSB, LSL, LVL and Laminated Bamboo in constructing hybrid floors systems is discussed in terms of their estimated ultimate moment capacity and through-thickness strain distribution once subjected to bending moments. The results provided insights into the usage of various sustainable engineered board products in novel composite floor systems and the importance of their elastic constants in addition to steel mechanical properties. The orthotropic elastic constants of wood and bamboo products are found to be important in describing the performance of such systems. Additionally, the connection between floorboards and cold-formed steel beams is shown to have significant effect on the local strain fields and the onset of failure in the studied composite floors. Comparing the predicted ultimate moment capacity (kN.m) of various composite floor systems with partial connections (practical case) highlights that by replacing PB with Laminated Bamboo, LVL, and LSL, the ultimate moment capacity of composite floor systems can be improved by 6%, 5% and 3.5%, respectively (see Table 1). No significant increase was observed for OSB due to the very low out-of-plane shear modulus (G_{23}) of OSB (170 MPa) compared to PB (958 MPa).

Table 1 : Predicted ultimate moment capacity (kN.m) of various composite floor systems. The board thickness and dimensions are the same for all systems

Connection System	Engineered Board Product				
	PB	OSB	LSL	LVL	Laminated Bamboo
No connection	40.6	40.6	41.6	41.6	41.8
Partial connection	47.8	48.0	49.5	50.3	50.5
Full connection	75	77.8	78.1	78.0	78.5

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ISBN: 978-91-88898-64-7

Modeling of wood products

ISBN: 978-91-88898-64-7

Timoshenko beam with enhanced stress recovery and constitutive relations describing the effects of variable grain direction on the behavior of a GLT beam

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Laser scanning technologies allow to collect detailed information on wood grain orientation on the surface of boards by means of the so called tracheid effect [1]. Obviously, such information could be exploited for accurate analysis of Glued Laminated Timber (GLT) beams [1,2], but it unavoidably requires models that properly account for the influence of grain direction on the strength and the stiffness of the assembled GLT beams. In greater detail, when the principal directions of an anisotropic material have arbitrary orientation, material constitutive relations are represented by a full matrix. Despite this peculiarity of anisotropic material could seem trivial, it deeply influences both stress distribution and beam stiffness, and it has been only partially addressed in literature.

This contribution aims at discussing (i) an iterative procedure for the recovery of the cross-section stress distribution, capable to effectively describe the effects of grain direction and (ii) a strategy for the evaluation of the beam constitutive relations to be used in a Timosehnko beam model. Specifically, the stress-recovery exploits the first material constitutive relation, the 2D horizontal equilibrium, and the beam equilibrium Ordinary Differential Equation (ODEs). The resulting distribution of axial stress depends not only on both axial deformation $\varepsilon(x)$ and beam curvature $\chi(x)$ via the Young's modulus, but also on the transversal internal force $V(x)$ via the non-vanishing out-of-diagonal terms of the matrix representing the material constitutive relation. The derivation of the beams constitutive relations exploits the stress recovery outputs and guarantee energy consistency between a simplified 1D model and the 2D elastic problem, leading every beam deformation to depend on all internal forces. The analytical solution of Timoshenko beam ODEs highlights that a coupling term relating the transversal internal force $V(x)$ with the curvature $\chi(x)$ can influence the transversal displacement of the structural element stronger than shear deformation.

A systematic comparison with 2D finite element solutions, obtained using very fine meshes, demonstrates the effectiveness of the proposed modelling approach. Specifically, the proposed beam model and the standard Timoshenko one, classically adopted in engineering practice, have similar costs. Nevertheless, the proposed model estimates maximal displacements and stresses with relative errors usually smaller than 10%, describing phenomena that occur in multilayer anisotropic beams with an accuracy reasonable and sufficient for most engineering applications. Conversely, coarse adaptations of beam models developed for isotropic structural elements may lead to errors greater than 30%, resulting inadequate also for basic engineering applications (see Figure 1).

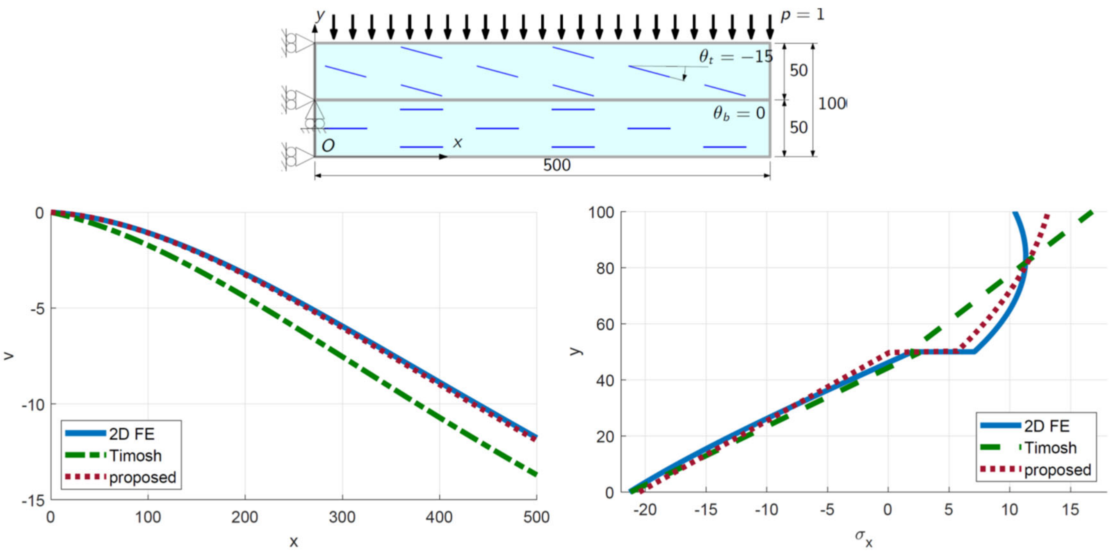

Figure 1: Bi-layer cantilever with distributed load, $E_{11}=10^4 MPa$, $E_{22}= E_{11}/20$, $G= E_{11}/10$, $\upsilon=0$. Comparison of transversal displacements and axial stress evaluated at $x=250mm$, 2D Finite Element, Timoshenko beam, and proposed beam model solutions

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On the question whether the volume of glulam bending members changes their reliability

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According to Eurocode 5 rules [1], the characteristic bending strength of glulam may be increased for member depths h less than 0.6 m to benefit from the volume effect. For depths greater than 0.6 m, the bending strength is assumed as constant value, although it seems logical that the volume effect should also be considered, at least to a certain extent. For this purpose, the depth factor k_h given in Eq. (1) was proposed [2].

$$k_{\mathrm{h}} = (0.6 / h)^{0.12} \qquad 0.3 \text{ m} \leq h \leq 3.0 \text{ m} \tag{1}$$

However, the question whether the application of this depth factor actually changes the reliability or not was still open and caused critical discussions [3]. Therefore, it is the aim of this contribution to examine the influence of the depth factor on the reliability index β. Bending strength data, obtained from computer simulations [3], for two glulam strength levels A and B (Table 1) were processed in reliability analyses using the Monte Carlo approach. Corresponding details of the numerical method are quoted in [4, 5]. In doing so, the limit state functions g_1 and g_2 given in Eq. (2) were evaluated based on millions of single realisations. In g_1, the bending strength denoted as R was divided by k_h in order to show its effect. The normally distributed bending stress denoted as E was calibrated so that the index β is 3.8 on average across the depth range of 0.3 - 3.0 m in case of g_1. That is fulfilled for glulam A with a mean stress of 15 N/mm² and for glulam B with 24.5 N/mm² and a COV of 10 % each.

Table 1: Mean (in N/mm²) and COV (in %) of the modelled bending strength values depending on depth (in m)

Depth	0.3	0.6	0.9	1.2	1.5	1.8	2.1	2.4	2.7	3.0
Glulam A	37.9/17.3	32.6/14.8	30.6/12.5	29.1/11.9	28.0/11.7	27.5/10.6	26.9/9.84	26.1/10.1	25.7/9.09	25.4/8.80
Glulam B	56.6/16.4	48.6/13.3	45.6/11.2	43.2/11.4	41.9/10.7	40.8/9.72	39.7/9.59	38.9/9.03	38.4/8.31	37.7/8.17

$$g_1 = R / k_{\mathrm{h}} - E \qquad g_2 = R - E \tag{2}$$

Fig. 1 shows the impact of the depth factor on β. In case of g_1, the course of β oscillates around 3.8, and in case of g_2, β oscillates too, but decreases with increasing depth. The oscillation is due to limited amount of simulated data points per depth. The proposed depth factor (Eq. (1)) is suitable to modify the glulam bending strength of members with depths greater than 0.6 m in order to reflect a more balanced reliability within the Eurocode 5 format. However, simplifications regarding the modelled bending stress in the reliability analyses, i.e. omitting variable loads, limit impact of the results. Hence, this contribution aims at triggering and facilitating discussions on further steps such as size depending load or stress models and the inclusion of appropriate model uncertainties.

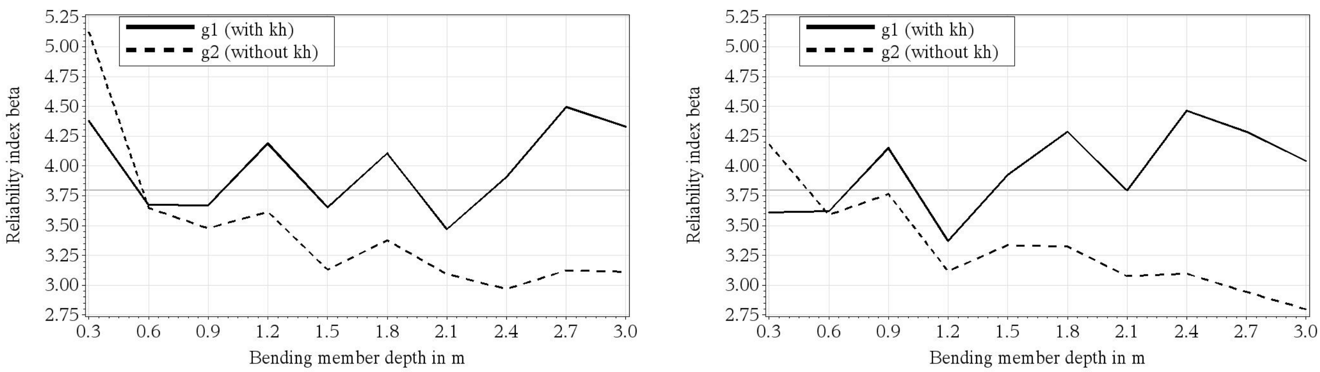

Figure 1: β depending on member depth and application of depth factor k_h, glulam A (left) and B (right)

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ISBN: 978-91-88898-64-7

HYBRID GLT-LVL GLULAM – MODELLING AND EXPERIMENTS

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It is reported on the ultimate limit states calculation and respective experimental full scale verification of hybrid glued laminated timber (GLT) members subjected to bending. The beams are built up asymmetrically with unjointed and jointed laminations made of laminated veneer lumber (LVL) placed at the bending tension edge, whereas the majority of the cross-sectional depth consists of finger jointed spruce-fir laminations as in case of usual GLT. The number of LVL laminations, i.e. the ratio of LVL depth vs. total depth, may vary from very few percent to about 25 %, depending on the targeted reinforcement degree and capacity gain. The LVL material can consist of spruce/fir, or from hardwoods such as beech and birch, showing rather different stiffness and strength properties. It is evident that the LVL reinforcement leads to a bending capacity increase depending on the reinforcement ratio. It is further sensible to expect a decreasing reinforcing effect with higher LVL ratios. The composite action is strongly influenced by the non-linear yielding and compressive damage effects of the solid wood laminations in the compression zone of the cross-section. Further, the maximum reinforcement gain may be influenced by a normal-shear stress interaction, too.

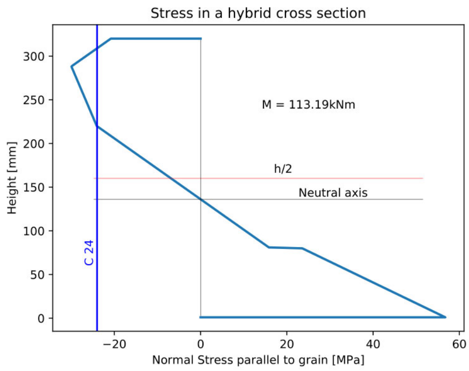

Figure 1: GLT beam with 25 % Beech LVL Reinforcement

The paper firstly describes some aspects of an iterative moment-rotation model based on Bernoulli's theorem of preservation of plane sections, hereby accounting for the compressive non-linearity (yielding and softening of the compression zone). Some quantitatively significant differences to usual unsymmetrical GLT beam buildups are highlighted. Secondly, the quasi analytical model is compared to results from finite element simulations, taking into account the localisation of compression damage and its impact on the spacious load redistribution.

Finally, the calculation results are compared to the findings of an extensive full-scale experimental campaign on hybrid GLT-LVL beam buildups. It is revealed that the employed iterative semi-analytical calculation approach enables a good and safe assessment of the regarded new high performance GLT build-up configurations.

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ISBN: 978-91-88898-64-7

Influence of the material thickness and microstructure on the mechanical properties and the pressure distribution in timber constructions

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Wood is widely used as construction material due to its mechanical efficiency and damage tolerance [1]. Typically, construction sizes are large, i.e. buildings or bridges, and to realize glued joints between components, vacuum or hydraulic presses are used, to get a sufficient pressure in the joining area. Screw-press-bondings (SPB) give the opportunity to apply pressure, where an external pressure application cannot be realized [2]. However, the resulting pressure distribution is significantly lower and strongly dependent on parameters like the used base material, planking, adhesive and type of screw [2], so optimizations are necessary to reach conditions set by the adhesive industry. To create safe and resistant constructions with SPB in future, it would be desirable, if the resulting pressure distribution in the glue line can be predicted and estimated over numerical methods.

In this study, the mechanical properties, i.e. strength and stiffness, of spruce and beech were determined over quasi-static tensile, compression and bending tests to describe the material behavior and implement it in a finite element model (FEM). To assess the effect of external influences, like component size and microstructure, specimen geometries and orientations were varied. To calculate input parameters for FEM, like elastic modulus E, shear modulus G and Poisson ratios ν, total strain was measured by using digital image correlation (DIC) as a contact free method, excluding the influence of extensometer edges on the surface [3]. In addition to the DIC, to characterize damage visible on the surface, acoustic emission (AE) measurement was used to get information about internal damage.

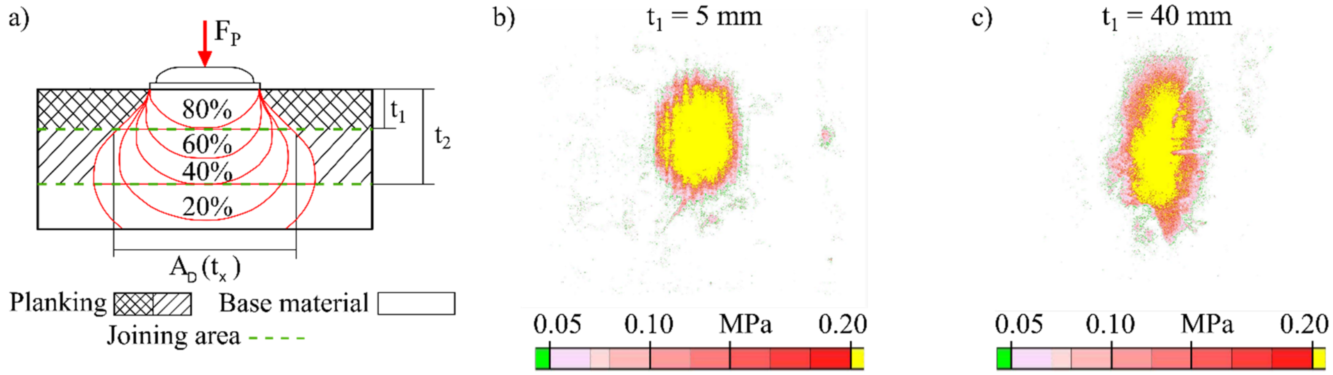

Figure 1: a) Scheme of pressure distribution, and experimental results for spruce with b) $t_1 = 5$ mm and c) $t_2 = 40$ mm

To enable a qualitative characterization of pressure distributions in glue lines, resulting from specific combinations of the above mentioned parameters, special pressure measurement films (PRESCALE, Fujifilm Europe) were used by placing them between the wooden components during mechanical loading. Figure 1 a) shows a scheme of pressure distribution to explain the material behavior under pressure, where the pressure area increases with material thickness, whereas the joining zone pressure decreases. Figures 1 b) and 1 c) show the corresponding results of the measurement films for different material thicknesses t_x determined in quasi-static compression tests.

In further steps, material behavior will be implemented in FEM to realize a validation between numerical and experimental results. Further, screw forces and their effects on the resulting glue lines will be included. The aim of the project is the direct numerical calculation of efficient screw arrangements dependent on the existing screw force and needed joining zone pressure according to the adhesive.

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 ISBN: 978-91-88898-64-7

2D computational modeling of the influence of transverse reinforcement on perpendicular to grain stress in double tapered glulam beams

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The present paper focuses on numerical simulation of the influence of reinforcement on the distribution of perpendicular to grain stress in a double tapered glulam beam. Analysis of certain distributions without the reinforcement was shown among others in [1-3], where geometrical parameters and various orthotropic properties were taken into account. The numerical model was created using the finite element method in Abaqus environment. The double tapered beam (Fig. 1) is a simply supported structure with a span of L=36 m, height over the supports 1 m, apex height 3.4 m and width 0.3 m. The beam is loaded with concentrated forces P=125 kN and is made of an orthotropic material corresponding to the glulam GL28h. It was modeled as a 2D plane stress problem within the linear theory of elasticity. The apex zone was reinforced with 6 bars of length 2 m and a diameter varying from 9 mm to 13 mm, which were modeled as bar elements. In the beam 10206 finite elements type CPS8R (eight-node plane stress elements with reduced integration) were defined. Each reinforcing bar was divided into 15 finite elements type B22 (three-node plane beam elements). The translational degrees of freedom of bar elements are consistent with the 2D model of the beam.

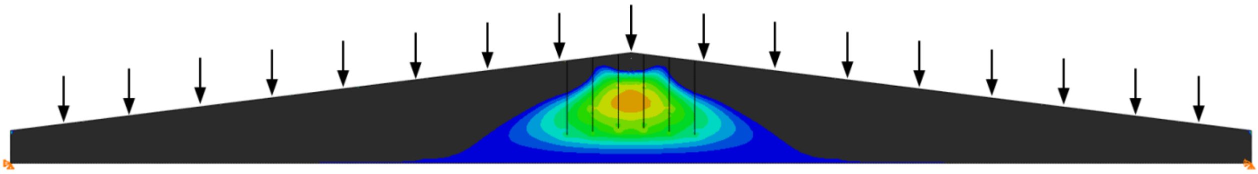


Figure 1: Extensional normal stress distribution in double tapered glulam beam with reinforcement.

Examples of the distributions of positive normal stress perpendicular to the grain are presented in Fig. 2.

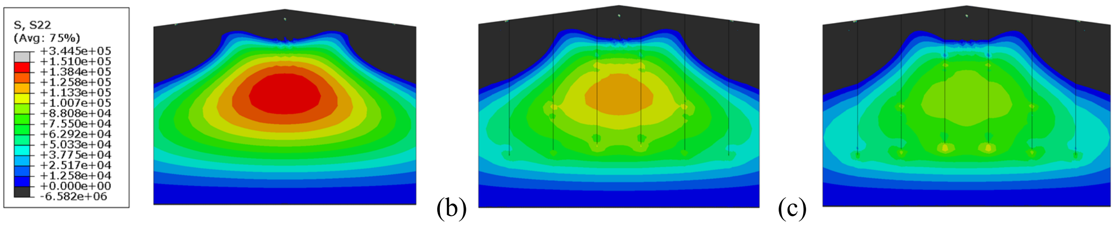
(a) 

Figure 2: The influence of the reinforcement: no reinforcement (a), 6xϕ9 reinf. (b), 6xϕ13 reinf. (c).

An analysis of perpendicular to grain stress depending on the level of reinforcement (6 bars with diameters 9, 10, 11, 12 and 13 mm) is shown in Fig. 3. Distribution of stress in the apex cross-section (Fig. 3a) and influence of the reinforcement on the maximum value of stress (Fig. 3b) are given. The numerical simulation indicates that it is possible to reduce perpendicular to grain stress by 30%. A wider numerical simulation will be presented at the conference.

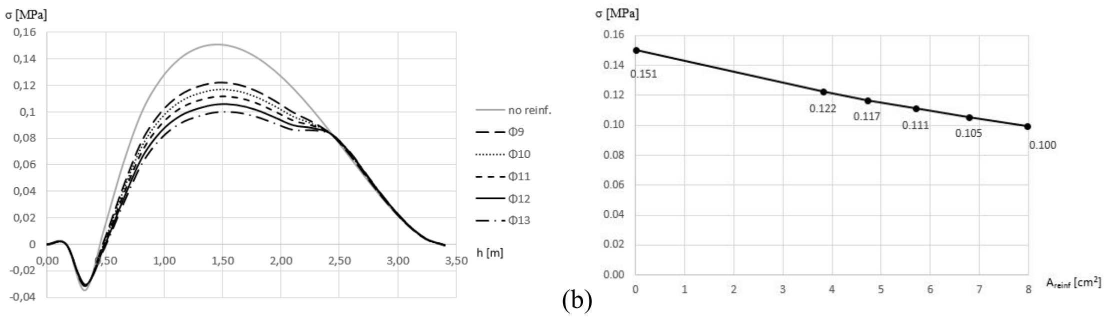


Figure 3: Normal stress analysis: distribution in the apex (a), maximum values against the reinforcement (b).

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Numerical modelling of beam-beam connection systems using compressed wood plates and dowels

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The compressed wood could be an eco-friendly alternative for metallic fasteners and adhesives in timber construction. Compression of wood leads to improved material characteristics, increase density and dimensional stability [1]. In this paper, the results of a finite element analysis on beam-beam spliced connection with compressed wood (CW) plates and dowels were presented. The main aim of this numerical modelling was to perform a parametric study to optimize the number and configuration of CW dowels and plates in the connection. There were three FE models developed for three different design configurations of beam-beam connection systems fabricated using CW dowels of 10 mm diameter and CW plate of 10 mm thickness. Figure 1 shows the geometry and stress distribution of a typical beam-beam connection using full CW plate. In the numerical models, the timber and CW were modelled as orthotropic materials with elasto-plastic behaviour. The interaction between the beam and plate is modelled by defining both tangential and normal contact behaviour. Tie constraints were used between the CW dowels and timber, and CW plates and dowels to form a tied connection during the simulation. Hexahedral 8-node reduced integration elements were used. Due to symmetry, only a half of the geometry was modelled in the finite element analysis. A comparison of testing [2] and numerical prediction of load-deflections and moment-rotations responses are presented so as to validate the developed 3D finite element models. The validated FE model is used to develop the influence of connection geometry on the responses. Based on the obtained results from parametric study, recommendations were provided on dowel patterns and geometrical conditions of the connection.

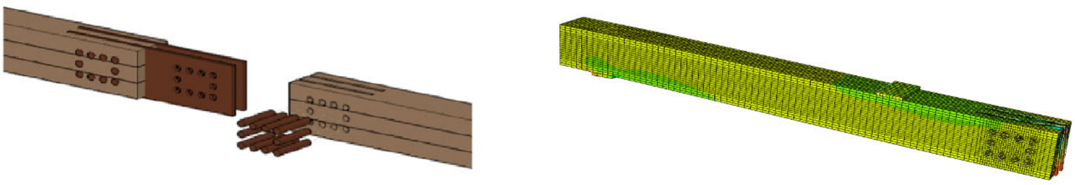

Figure 1. Full-plate connection (a) geometry; (b) stress distribution

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ISBN: 978-91-88898-64-7

Simulation of materials and structures

ISBN: 978-91-88898-64-7

Modelling wood anisotropy by the mean of the Discrete Element Method for cutting process simulation

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When modelling wood at mesoscopic scale in order to simulate processing operations such as wood cutting, one may face strong barriers using the straightforward methodology. Indeed, traditional cutting simulations use Finite Element Model (FEM) to represent the material (often metal) and the cutting tool. When the material machined is wood, and the attention is focused on the chip as well as the final product (for material value enhancement) the simulation must be able to robustly consider multiple cracks, fragmentations, and contacts. In this case, the method starts to lose in efficiency to the benefice of the Discrete Element Method (DEM) which display much better performances in modelling robustly these phenomena.

Formerly designed to model granular media [1], the Discrete Element Method is now also used for continuous material modeling. The material is discretized into small discrete elements (DE), usually spheres. The DEs are linked one to another thanks to bonds. Bonds behavior are set to reproduce the media behavior. DEM has shown today its ability to model isotropic materials faithfully, however it is not yet the case for anisotropic media such as wood at mesoscopic scale. This study, based on a very recently published paper [2] highlights the obstacles encountered when modeling wood-like orthotropic media.

Two different modeling approaches are considered: cubic regular arrangement, where discrete elements are placed on a regular Cartesian lattice (an approach already implemented once in a wood cutting simulation [3]), and random sphere-packed arrangement, where elements are randomly packed. As the second approach is initially favoring isotropy of the domain, a new method to affect orientation-dependent Young's modulus of bonds is proposed to create orthotropy. Domains created by both approaches are loaded in compression in-axis (along the material orthotropic directions) and off-axis to determine their effective Young's modulus according to the loading direction.

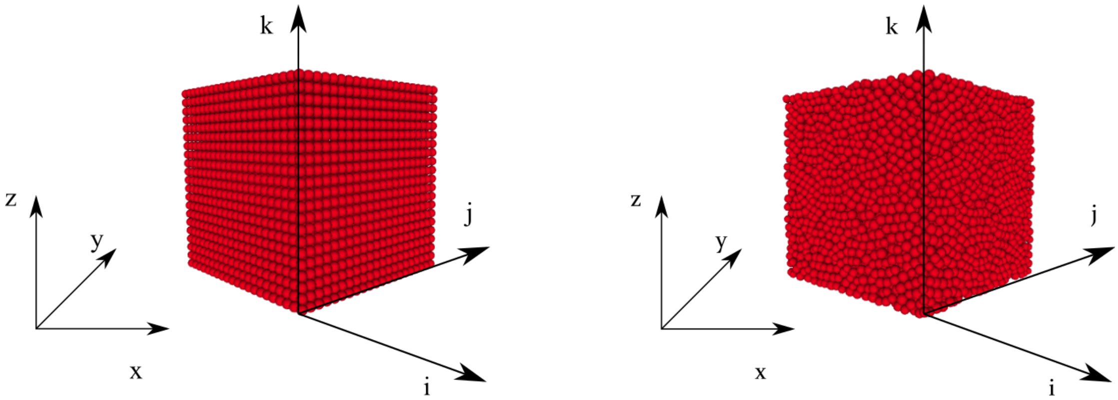

Figure 1: Discrete models based on cubic regular arrangement (left) and on packed arrangement (right) [2]

Results are compared to an estimation given by the Hankinson formula which is often used to represent high orthotropic behavior such as encountered in wood or synthetic fiber materials. For this class of materials, it is shown that, contrary to cubic regular arrangement, the random sphere-packed arrangement exhibits difficulties to reach highly orthotropic behavior. This prevent from the modelling of wood from most of the tree species. Conversely, this latter arrangement displays results closer to continuous orthotropic material during off-axis tests. Therefore, in the case of modelling of only radial-tangential planar phenomena or woods with low anisotropy, the random sphere-packed arrangement is to prefer, while cubic regular arrangement should be used for modelling of longitudinal-transversal phenomena, but with peculiar care.

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 ISBN: 978-91-88898-64-7

Modeling inhomogeneities of veneers with a grayscale mapping approach

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The formability of a veneer sheet on a given geometry depends on its individual macroscopic structure especially at critical areas, where large deformations occur. Hence, this local material behavior has to be taken into account for numerical predictions of the forming process of wood surfaces for automotive interior trim parts. A method is presented where grayscale values from images are mapped to material IDs on finite element meshes. This method provides an automatic discretization of visible macroscopic structures.

Images of sliced and burled ash wood veneers were chosen for the tests (Figure 1). The images were exported into the grayscale file format pgm (portable graymap) containing a grayscale value within the range of 0 to 255 for each pixel in ASCII format. Mapping has been performed with a recent development version of the mapping tool envyo® [1]. Based on the image size, a point cloud was created which contained all pixels and their assigned grayscale value. After alignment with the target finite element mesh, a nearest neighbor search was performed. Depending on the found grayscale value, material properties of early or late wood were assigned to shell elements in order to consider the different areas. The grayscale mapping method was applied to simulations of a Nakajima forming test (Figure 2). An orthotropic elastic material model was implemented in LS Dyna. Engineering constants of early and late wood were derived from tensile tests. Comparison of the simulation results to experimental tests of [2] showed a very good agreement in the major strain distribution (Figure 2b).The meshes generated by grayscale mapping showed the general behavior of the ash wood veneer sheets under 3D stress, induced by the sinking punch. Strain concentrated on the early wood zones due to the lower Young´s modulus in the direction transverse to the fiber axis (Figure 3).

With the presented method early and late wood areas as well as knots and other visible defects can be captured in numerical models always supposing that local testing data can be provided. A standard procedure for the configuration of grayscale ranges and element size must be further developed, to capture macroscopic structures properly according to the anatomical structure of wood. Additionally, the mapping of local fiber directions would improve forming predictions, especially for burled veneers.

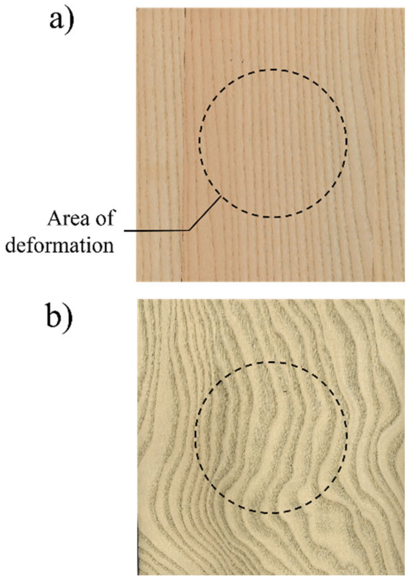

Figure 1: Images taken for mapping of a) sliced ash wood veneer sample and b) burled ash wood veneer sample

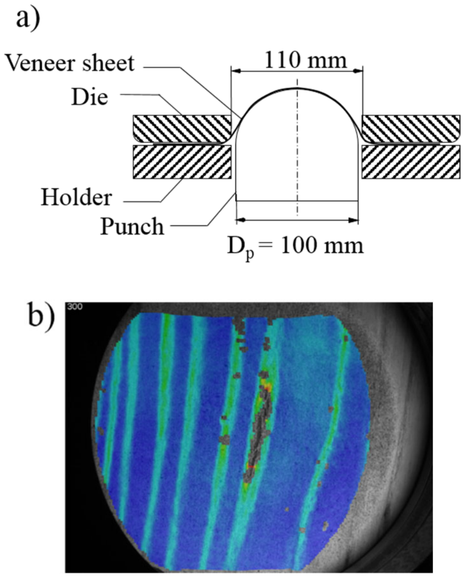

Figure 2: a) Nakajima test setup, b) Experimentally determined major strain distribution with crack of ash wood veneer

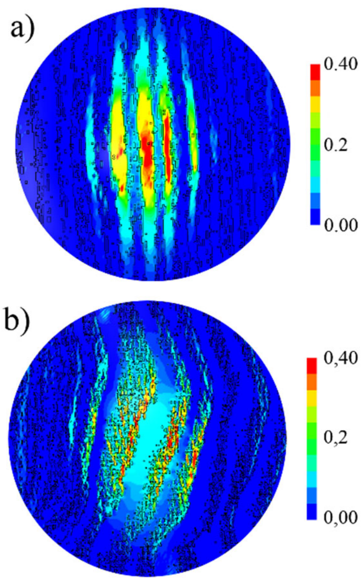

Figure 3: Simulation results (major strain) for meshes created from grayscale mapping for a) sliced and b) burled ash wood veneer

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ISBN: 978-91-88898-64-7

Notches in wood at arbitrary beam location – numerical modelling and challenges

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In both historical and modern structures, the phenomena of notching are omnipresent and evident. Notching is either characterized by abrupt changes of cross section with respect to height or width, holes in beams or even loading – external or by internal forces or moments – not distributed over the whole cross section, but acting on subsections. At the location of the (therefore) created edge, the phenomenon of peak stress affects all involved stress components. The present design recommendations in EC5 [1] only handle the onset of crack formation induced by a coupled set of shear force and moment, as it is the case of e.g. single span beams without further cantilevers. In the meantime, strategies for the approval of reinforcements have been developed and implemented in national appendices of EC5, still referencing the initial ultimate load carrying capacities at onset of crack formation at the tensile bending side of the notched beam. Nevertheless, further research revealed, that the increased level of load carrying capacity for the case of reinforced notches is also limited by a factor of approximate two times the load carrying capacity at onset of crack formation. Because of this scientific finding, all timber structures designed in the past according to the traditional design concept for reinforced notches should be classified as insufficient with respect to the expected safety level and therefore eventually be upgraded. Besides this economic disaster, the most important consequence would be the loss of engineers trust in the reliability of traditional and future design concepts related to this topic.

The here presented research activities aim at a gentle solution of the above-mentioned problem by scanning all related background documents, reproduction of the referenced derivation of design equations from Gustafsson [2] and validation against a more flexible calculation made using numerical 2D formulations. First findings have revealed that the referenced **beam model is better adjusted to moments than to shear forces**. With the shift of the notching to the compressive bending side of a beam, a global **increase factor of about two due to the change of the corresponding fracture mode** (from I to II) and related smaller change of the compliance could be applied with reference to the basic design equations from EC5. When considering the original concept of Gustafsson [2], the validity of the design of the original formula has been checked especially in the context of **varying height of beams, also to account for high beams made of LVL or glulam**. A study **on possible decomposition of internal forces** was performed and results were compared to the original formula. The **fracture mode** (I or II), suitable to be applied to some specific problem, could **be chosen according to change in deflection respective slope** of the (newly created) crack. The early-stage research showed good applicability of simple numerical models to describe phenomena according to Fig. 1.

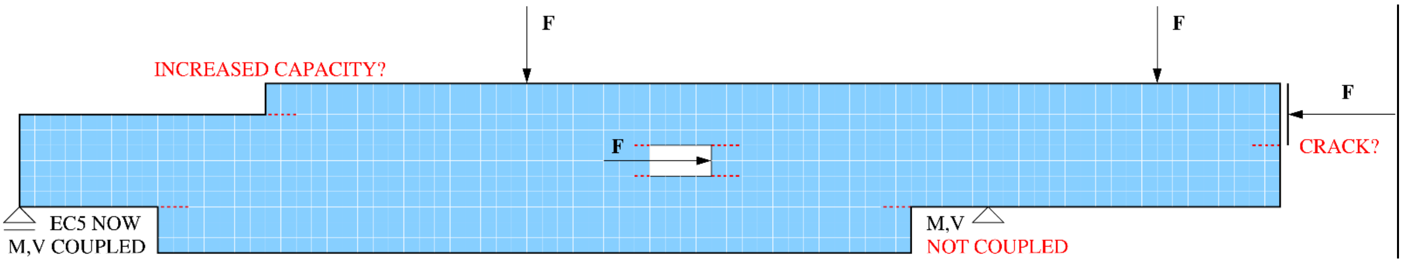

Figure 1: Problem sketch of some of the problems found in practice

Concluding, the issue of notching should principally be applicable for an arbitrary set of internal forces consisting of moments, shear forces and normal forces from global structural assessment. Since analytical formulae, only to be derived from simplified beam models, will never do this job, **reliable and more flexible numerical models (and submodels)** in the background of structural engineering software [4] could offer a reasonable solution for this urgent problem and frequent design situation in timber engineering. However, this topic requires high amount of experimental data not only from the past, but also new ones, since verification of the design procedure should be made using extensive additional experimental work. A project relating this topic should be considered in order to fulfil the ambitious goals.

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ISBN: 978-91-88898-64-7

In-plane buckling analysis of transversely loaded timber beams

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The loadbearing capacity of a transversely loaded timber beam is today in structural design determined by use of a strength criterion $\sigma_m \leq f_m$. The bending strength f_m is considered as a material parameter representative for the wooden material without considering any influence of second order effects.

It is well known that buckling should be considered in case of compressive axial loading but that this also holds for transversely loaded beams is not yet recognised as for the cantilever beam shown below.

A new model is presented in this paper, dealing with timber beams subjected to transversal loads. For simplicity only in-plane displacements will be considered. Two different kinds of buckling modes (orthogonal eigenmodes) of great importance can appear and both should be checked in order to determine the loadbearing capacity.

Greenhill was first to solve the problem of finding the buckling load for a column subjected to a uniformly distributed dead load. He used Bessel functions, see [1] and [2]. Nowadays this type of problem can be solved by finite element analysis considering it as an eigenvalue problem. The most important material parameter will be the bending stiffness as in case of ordinary Euler buckling. The second kind of buckling modes refers to cases where the loading mainly causes bending deformations. Buckling will occur for a much lower load value than expected with respect to the measured value of the bending stiffness. The existence of this second kind of buckling seems to be unknown in literature.

To explain the buckling phenomenon of the second kind the beam is thought to be lengthwise subdivided into two halves, one mainly in compression and the other one in tension. The fictitious cut dividing the beam into two parts is along the neutral axis. For statically determined beams we can directly find the shear stresses acting on the cutting planes of the two beam parts. The shear stresses can be considered as external axial loads giving rise to an antisymmetric buckling mode. The critical load value when buckling occurs is obtained by an eigenvalue analysis. The lowest eigenvalue will be the same for the compressed and tensioned parts except for the sign.

The example below shows the main features in the modelling for solving beam buckling problems of the second kind. The specimen studied is given dimensions according to EN 408 and is of special interest as it is used in classifying timber into different strength classes. The validity of the proposed theory is supported by results from test series with more than 500 boards of different dimensions [3].

When the E-modulus is constant over the cross section the total loadbearing capacity is three times that of one individual beam part. This assumption of a constant stiffness over the cross section will often result in misleading results. A much better strength prediction is normally obtained if the radial variation of the longitudinal E-modulus from pith to bark is considered.

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ISBN: 978-91-88898-64-7

Timber composite and wood material characterization

ISBN: 978-91-88898-64-7

Development of an Innovative Multifunctional Roof and Ceiling Design in Timber-Concrete Composite Construction

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Office buildings are usually build using a flat ceiling construction to gain the maximal space utilization. Therefore sandwich constructions are typically used because of their simple installation. The transfer of this construction to residential and multi-storey buildings is inefficient due to the high dead weight of this construction. A lighter construction could satisfy these requirements. In a cooperative research project at the University of Kaiserslautern such a light system in timber-concrete composite construction is going to be developed. From a static point of view the timber-concrete cross section presents the optimal load-bearing system with a high bearing capacity in combination with minimized dead weight and the possibility to implement wide spans for transverse roof surfaces. Furthermore, it should be possible to use the timber-concrete composite construction as a roof as well as a ceiling. The concrete slab should be orientated to the rooms, which are going to be conditioned thermally by the heating and cooling pipes integrated in the concrete slab. In case of a roof, the wooden beam is on the top and in case of a ceiling, the wooden beam is under the concrete slab.

The new construction consists of two elements which should be connected easily at the construction side. The first element of the multifunctional construction is a wooden beam with a formed steel sheet fastened in two slots in the beam. The transmission of the shear forces to the wooden beam will be realized with self-screwing steel studs. The second element is a precast concrete element with a similar formed steel sheet integrated in the concrete. To finally get a load-carrying connection the cavity will be filled with a cementitious suspension. During the manufacturing processes of the steel sheets, the sheets will be shaped with cams and holes to transmit the longitudinal shear forces.

Figure 1: Schematic Sketch of the New Timber-Concrete Composite Construction

To determine the load bearing capacity of the new construction for the combination of tension and longitudinal shear, pull-out and push-out tests will be carried out. The results of the push-out and pull-out tests, performed at the laboratory, will be presented in comparison with numerical simulations. The aim of these investigations is to define the separate carrying components of the different materials and elements. As a result of these conclusions a new design model considering all failure mechanisms will be developed. In addition to that, the long-term behavior, especially the temporally different creep behavior of wood and concrete, will be another important point to include in the design method.

Determination of Moduli of Elasticity of Latewood and Transition Latewood of Japanese Cedar by Using Digital Image Analysis

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Wood is a kind of composite material found in nature. The latewood (LW), transition latewood (TL), and earlywood (EW) as layers of composites are observed in a tree ring and are varied from species to species and from site to site. To understand the stiffness of wood structures, it is necessary to accurately determine the modulus of elasticity (MOE) of the timber. However, as a typical example, the MOE of Japanese cedar (*Cryptomeria japonica*) ranges from 2 to 12 GPa [1-3]. Therefore, it is difficult to select an appropriate value of MOE to be used in computational mechanics for studying wood and wood-based products. In this paper, X-ray densitometry (XrD) technique, tracheidogram method, and digital image analysis (DIA) were employed to determine the boundary of LW, TL and EW proportion of Japanese Cedar popularly found in both Taiwan and Japan. The determination of MOEs of LW, TL, and EW were based on the correlation between MOEs and density.

In this paper, Japanese cedar was used as the specimen material. The wood log was cut across over the pith into wood lumber pieces and then air dried for more than one year before the experiments were performed. XrD specimens with a thickness of 2 mm were cut from the top of the lumber. All samples were maintained at 25°C and controlled at 60% relative humidity for more than a week. A commercially available X-ray densitometer, QMS QTRS-01X Tree Ring Scanner [4] was used to measure the density profiles of the XrD specimens.

The tracheidogram method uses curves of cell size variations in radial files of xylem cells [5]. To observe variations of tracheid dimensions along a tree ring, the digital image of cell walls was measured by using an optical microscope at 100X magnification. Because of the limitation of the field of view, an image stitching technique must be used on the obtained full image. A commercially available software, MATLAB [6], was used in this paper to measure the lumen radial diameter and the radial wall thickness. According to the Mork's definition (MD) [7], the boundary of LW, TL, and EW were defined as follows: MD $\leq$ 0.5, EW; 0.5 $<$ MC $<$ 1, TL; MC $\geq$ 1, LW.

The DIA technique proposed in [8] is based on the optical properties reflected by the light on the specimen surface. The gray level was determined from the light reflected by the radial cross-sectional surface. The dark and light fringes represent a lower and higher gray level, respectively. It should be noted that the dark and light fringes correspond to the higher and lower density of the density data obtained from XrD technique, respectively.

Based on the resemblance between the density profile, MD curves, and reverse gray-level profile, the use of DIA was proposed in this paper to determine the boundary of LW, TL, and EW portions. Finally, the values of MOE of LW, TL, and EW were determined by using the boundary of LW, TL, and EW based on the correlation between MOE and density.

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ISBN: 978-91-88898-64-7

Hybrid glulam beam made of beech and spruce laminations – experimental and numerical investigation

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A standard four point bending test was performed on a hybrid glulam beam made of beech and spruce laminations. The beam consisted of 8 spruce laminations with the thickness of 18 mm in the core of the glulam cross section and of 6 beech laminations with the thickness of 13 mm on both outer sides. The total dimensions of the cross section were b/h = 140/300 mm, where the length of the beam between the supports equaled 470 cm. The length of finger joints equaled 19 mm for all laminations. The melamine urea formaldehyde (MUF) adhesive was used for the finger jointing and for gluing the laminations. The described glulam beam failed in a brittle manner (Figure 1). The outer beech laminations predominantly remained undamaged, whereas the critical areas were the longitudinal adhesive layers and the finger joints. In the core of the cross section, which consisted of spruce laminations, a diagonal crack occurred across the laminations.

On the basis of the experiment, a numerical model was defined in Abaqus software. Model consisted of parts with linear elastic and orthotropic behavior for beech laminations, whereas for the spruce laminations orthotropic elastic and perfectly plastic behavior in tension were assumed. The adhesive layers between the laminations and the finger joints were modelled by cohesive surfaces, using a nonlinear cohesive zone model (CZM). The input elastic parameters for the Slovenian beech and spruce were based on previous studies [1]. On the other hand the nonlinear fracture parameters for the CZM were selected according to the literature dealing with adhesively bonded beech timber [2].


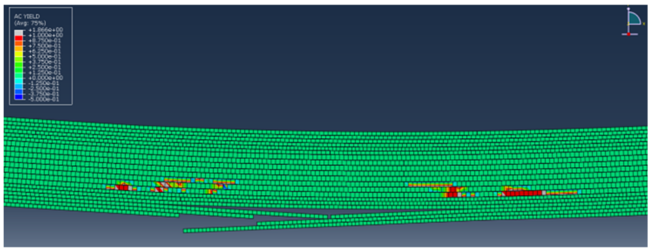

Figure 1: The failure of the hybrid glulam beam: experimental (left) and numerical (right)

The comparison between the numerical model and the experiment showed good agreement for global output parameters (Table 1). The model fits best in the elastic part of bending as well as in estimating the force at the initiation of failure, whereas for estimating the post-failure response the model needs to be improved.

In the conference contribution the influence of input parameter variation on the response of the numerical model will be presented. It was found, that the initiation of failure of the hybrid glulam beam model was highly dependent on the nonlinear input parameters for the CZM of the finger joints and the longitudinal adhesive layers.

Table 1: Comparison of experimental results and calibrated numerical model

	Experimental	*Numerical*
Local modulus of elasticity in bending $E_{m,l}$ [MPa]	1563	1378
Beam deflection at failure w [mm]	64,6	68,9
Force at failure F [kN]	133,4	138,6

The research was a part of a Slovenian national project (TIGR4smart), where the use of beech timber in structural applications was encouraged. The results of experiments and numerical modelling indicate that the capacity of the finger joints in beech and/or the adhesive layer between the beech laminations should be improved.

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ISBN: 978-91-88898-64-7

Mechanical Properties of Oil Palm Wood (*Elaeis guineensis* JACQ.)

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Oil palms (*Elaeis guineensis* JACQ.) are mainly cultivated in large plantations for palm oil production to be used for food, chemicals, pharmaceuticals and energy material. Worldwide, oil palms cover an area of nearly 25 million ha of which 75 % are located in Asia. After 25 years of age, the palms are felled and replaced due to declining oil production. Like all other biomass, the trunks remain on the plantation site for nutrient recycling. This leads to increased insect and fungi populations causing problems for the new palm generation. Many regions where oil palms grow currently suffer from a decline in timber harvested from their tropical forests. The average annual total volume of trunks from plantation clearings amounts to more than 100 million m^3. Recent research has explored the commercial uses of oil palm wood. In many cases, the wood can substitute for tropical hardwoods, e. g. as panels (block-boards, flash doors, multi-layer solid wood panels) and construction timber. Appropriate use of the wood requires defined elasto-mechanical properties and therefore grading of the lumber.

Being monocotyledons, palms show distinct differences in the anatomical structure compared to common wood species. Only lateral and no radial growth of the stem means no growth rings, no wood rays, no knots. The wood consists of lengthwise oriented vascular bundles (VB) embedded in parenchymatous ground tissue. The vascular bundles are composed of vessels for water transport and sclerenchymatous fiber cells (fiber caps) with thick walls formed to fiber bundles for structural stability; the density of the VB is high between 0.8 to 1.4 g/cm^3. The parenchyma cells are thin walled and contain lots of water and sugars. Under load they easily buckle. The density of the dry parenchyma is low from 0.15 to 0.4 g/cm^3. Thus, from the structural mechanics point of view, if vascular bundles are considered as reinforcements (fibers) and ground tissue as matrix, oil palm wood can be seen as unidirectional long-fiber-reinforced bio-composite. The structure of parenchyma and vascular bundles defines the physical and elasto-mechanical properties. In the context of two research projects, elasto-mechanical properties of oil palm wood were tested on small-size test specimens: modulus of elasticity (MOE) and modulus of rupture (MOR) in bending, Young´s modulus and strength in tension and compression (parallel and perpendicular to the vascular bundles), torsional strength, shear strength, torsional modulus and G-modulus (in three main directions), embedding strength and screw withdrawal strength (parallel and perpendicular). All elasto-mechanical properties of oil palm wood correlate with density and volume fraction of vascular bundles respectively their fiber caps. Bulk density of palm wood depends primarily on the age of the palm tree and the size, number, and anatomical structure of its vascular bundles. Thus, palm trunks show a significant density gradient over both trunk height and cross section [1]. The number of vascular bundles decreases logarithmically from the cortex to the center of the trunk [2] and therefore density and elasto-mechanical properties decrease accordingly. The number per area of VB increases along with stem height [2], but because the anatomical structure of the VB varies as the stem height increases (cells in the upper trunk are younger and missing intensive secondary cell wall thickening), the bulk density and elasto-mechanical properties decrease accordingly. Therefore, the size and number of vascular bundles per area is not a sufficient visual grading criteria for oil palm lumber, neither for density nor strength and stiffness.

Different from common wood species, $f_{c,0} \approx f_m \approx f_{t,0}$ for low densities, with increasing density $f_{c,0} > f_m > f_{t,0}$; whereas $E_{t,0} > E_m >> E_{c,0}$. All property values are increasing with the density (and fiber volume fraction) as power law relationships with much higher exponents compared to common wood species and the rule-of-mixture cannot be confirmed for $f_{t,0}$, $E_{t,0}$, $E_{c,0}$ and $E_{c,90}$ because the concentration of VB, as well as the share of fibers within the bundles, is greater in the periphery of the stem than in the central tissue. Furthermore, the cell wall properties themselves are not constant, “cell wall thickening” is more pronounced in the peripheral tissue than in the central tissue and more in the bottom of the trunk than at the top [2]. The “fibers” of the composite material are not homogeneous nor regularly spaced, which leads to exponents > 1 of the power law relationship. Very much influenced by the properties of the “matrix”, shear properties and nail holding are lower compared to common timbers. In contrast, screw holding is high because of the anchorage of the screw between the vascular bundles.

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ISBN: 978-91-88898-64-7

Cross laminated timber structures

ISBN: 978-91-88898-64-7

Global Vibration Modes of a Four-Story Wood Building

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A four-story wood office building was equipped with a purpose-built measurement system for measuring temperature, moisture, and vibration data. Five bi-directional geophone sensors were included for long term monitoring. Vibration data from days with high wind were used to estimate the first three modes (eigenfrequencies, damping factors and mode shapes) of the building by use of operational modal analysis (OMA). Although the vibration levels of the building were very low, the first three modes were possible to extract with reasonable confidence. The eigenfrequencies were found to be approximately 3.3, 3.6, and 4.0 Hz, with relative (viscous) damping factors in the range of 2 to 3%. The mode shapes were shown to agree with those of a finite element model. An attempt to scale the modal model was made by installing a small electrodynamic shaker with a 1 kg moving mass, and excite the building with harmonic vibration close to the eigenfrequencies, and using the OMAH method. Although the results were inconclusive, the small shaker was shown to be sufficient to detect harmonic response around the building. More work is needed to increase the confidence in the scaling results.

ISBN: 978-91-88898-64-7

Numerical Optimization of Novel Connections for Cross-laminated Timber Buildings

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This paper presents Finite Element Analyses (FEA) on the performance of two types of novel hold-down connections, suitable for shearwalls of mass-timber buildings. The first hold-down consisted of hollow circular steel tubes, inserted in use in Cross-laminated Timber (CLT). The FEA models, which accounted for nonlinear material properties of both timber and steel components, were validated against experimental results [1]. The primary objective was to optimize the connector geometry and material properties to achieve a target yield capacity. Considering a capacity-based design approach, where all the wood components remain elastic and only the steel tube deformed, a sensitivity analysis was performed to quantify the input-performance correlations. It was shown that the diameter (d) and thickness (t) of the tube, as well as the coupler diameter (c) were the main parameters influencing the yield capacity and overall performance of the connection. The subsequent optimization resulted in an optimum geometry and provided specific material properties requirements, a load-carrying capacity of 100kN, an elastic stiffness of 30kN/mm, and a ductility of 9 was deemed obtained. Thereafter, a robustness analysis quantified the impacts of uncertainties in the material properties and geometry of the connection; it was shown that the optimized detailing exhibited a robust performance in the presence of uncertainties in the timber and steel material properties, as well as geometry [2]. The second hold-down consisted of a similar concept but replaced the circular steel tube with a rubber based material which provided the required strength and stiffness, but did not exhibit any plastic deformation.

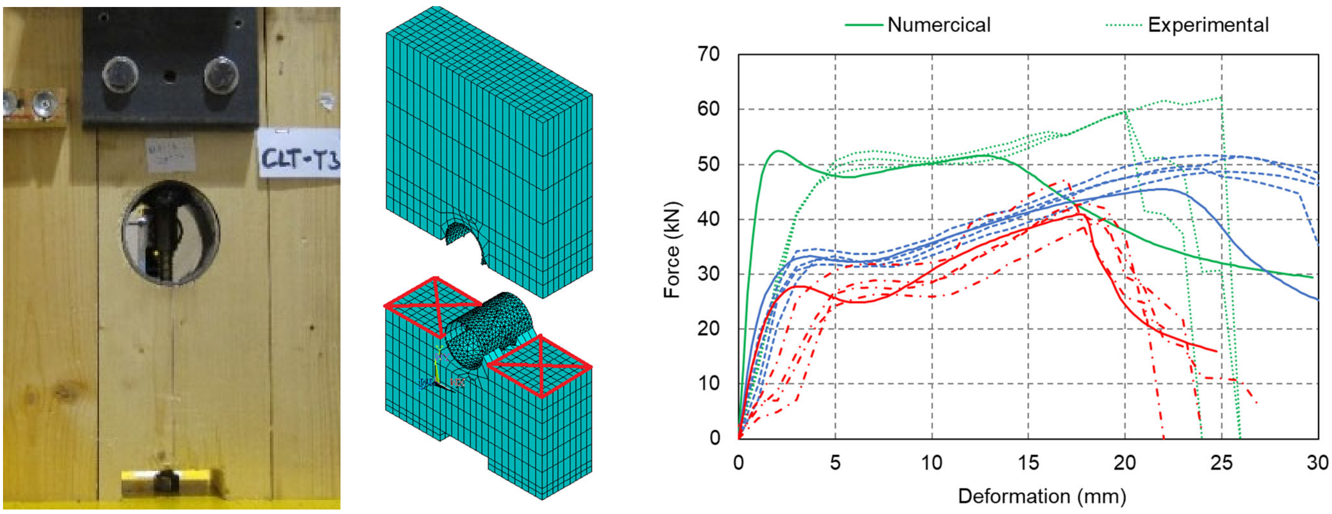

Figure 1: Tube test specimen (left); FEA model (center); and load-deformation curves (right)

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Nonlinear computational modelling of cross-laminated timber buildings

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Cross-laminated timber (CLT) is a relatively new construction building system based on structural panels made of several layers of boards stacked crosswise and glued together on their faces [1]. As CLT panels are light-weight structural elements with high stiffness and strength to bending, compression and shear, they are an economically competitive building system when compared to traditional options and therefore, are a suitable candidate for some applications which currently use concrete, masonry and steel. Given the numerous advantages of building with timber, an increasing number of multi-story CLT buildings is sprouting around the world.

In this work we investigate the nonlinear computational modelling and collapse of CLT buildings by means of a multi-scale modelling strategy. In order to determine the mechanical properties of CLT, a computational homogenisation scheme based on the volume averaging of the stress and strain fields over a representative volume element (RVE) of material is adopted. CLT floors and walls are modelled with mechanical properties obtained by the present multi-scale approach. Metallic connectors are modelled with their hysteretic non-linear behaviour, including damage. The behaviour of each connector is defined along three orthogonal axes. The building chosen for this investigation includes angle brackets, hold-downs, shear connectors and panel-to-panel screws. Some of our numerical predictions are compared with experimental results and are validated successfully. This is part of an ongoing research.

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ISBN: 978-91-88898-64-7

Simulating the mechanical behavior of connections

Strength and stiffness of hardwood joints experimental and numerical investigations

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In this study, a comparison between experimental and numerical results of hardwood dowel-type joints is presented. A phenomenological approach has been used to describe the contact between wood and steel dowel. This contact usually called embedment behaviour was idealised by a beam on nonlinear foundation where nonlinear springs composed the foundation and the dowel was modelled by 1-dimensional beam elements with elastic perfectly plastic behaviours (nonlinear Moment-curvature relationship). The previous described approach was investigated for the first time for steel-to-timber dowel-type connections by [1]. This idealised scheme was repeated for each dowel and linked each other by beams with elastic behaviours thus including the connection elements deformations.

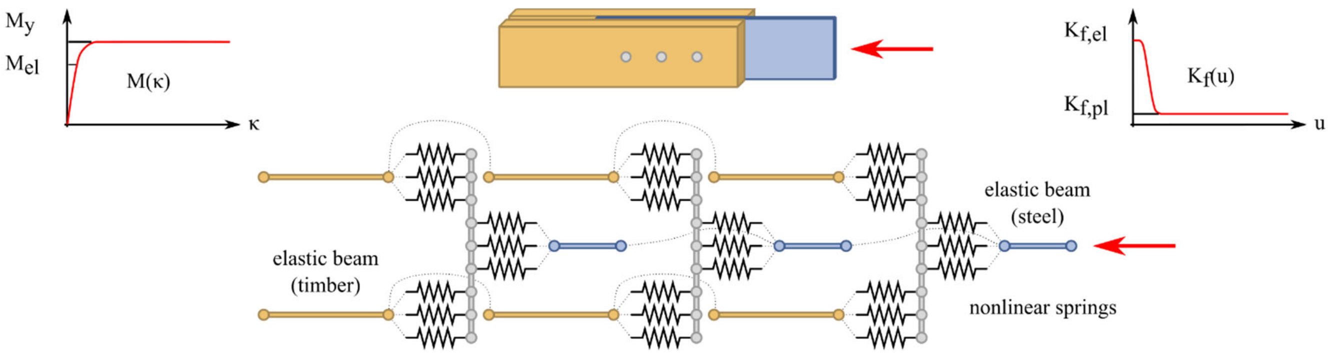

Figure 1: Description of the beam-on-foundation modelling for the numerical results of hardwood dowel-type joints (mesh example of a steel-to-timber connection with two shear planes)

In order to define the nonlinear springs behaviour of the foundation, this study was completed by embedment tests. Three different species were used: oak, beech and poplar. For each hardwood species, two dowel diameters were tested: 12 mm and 16 mm. Forty tests were carried out per subseries for the purpose to have a significant database.

The dowel steel grade being a significant parameter to describe the mechanical behaviour of timber, this study was also completed by tension tests in order to quantify the tensile yield stress of used dowels.

The comparison between experimental and numerical results was based on two types of joints with several subseries as defined in the table 1 below. The timber members thicknesses were chosen to encompass all failure modes defined by Eurocode 5.

Table 1: experimental programme of hardwood joints tests.

Type of joints	Species	n[1]	n_c[2]	d[3] [mm]	t_{out}-t_{in}[4] [mm]
Steel-to-timber	Oak	3	3	12, 16	15-12[5], 30-12[5], 85-12[5]
Timber-to-timber	Oak, beech, poplar	5	3	12, 16	15-40, 30-40, 97-75

[1] number of specimens per subseries
[2] number of dowels in a row
[3] dowel diameter
[4] respectively thickness of outer and inner timber members
[5] steel plate thickness

Beam-on-foundation model calculations and their comparison to the experimental results highlighted the validity of the numerical approach described above. In addition, effects of the dowel slenderness and diameter sensitivity were showed. These effects have already been demonstrated for timber-to-timber connections with softwoods in [2] (experimental way) and for single-fastener joints in [3] (modelling).

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 ISBN: 978-91-88898-64-7

Simplified calculation model for interconnected timber elements using wood-wood connections

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Metal fasteners and adhesive bonding are the main assembly methods in modern timber constructions. However, traditional wood-wood connections can be used effectively as well, thanks to increasing automation of the construction industry (CNC, robotics, etc.). The feasibility of this type of construction technique has been proven for buildings with complex geometry such as the wooden pavilion of the Vidy theatre [1]. As a result, this research is focused on the development of a simplified calculation model for standard construction elements using wood-wood connections called Through Tenon (TT). The goal is to obtain a dimensioning tool which is easy to use in practice for roof, slab and wall elements.

The newly proposed calculation model is inspired by previous research done on interconnected timber elements using both metal fasteners and wood-wood connections [2][3]. As shown in Fig. 1a, the numerical model is composed of beam elements (lines) with eccentricities represented by rigid fictive beams (dotted lines). Associated geometrical and material properties are assigned to each beam element. For TT joints, the tenon part of the connection is also represented by beams, while the contact zone between the tenon and the mortise is defined by springs to simulate the contact stiffness of the joint (Fig. 1c).

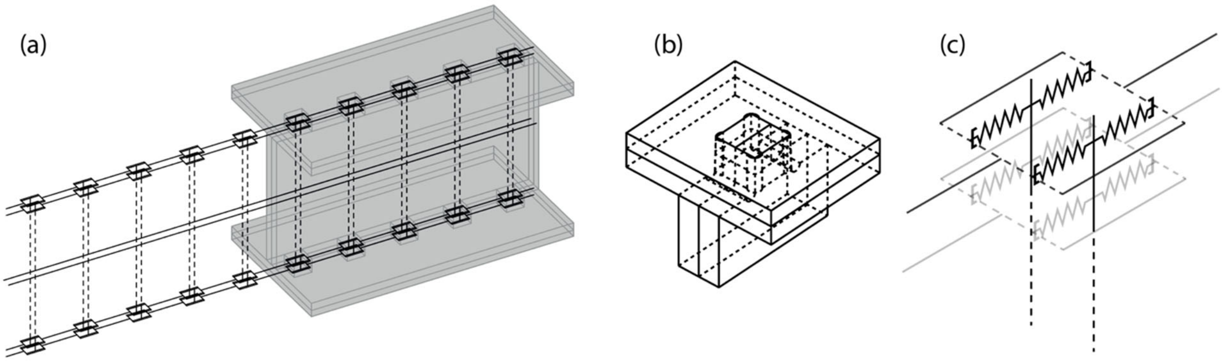

Figure 1: (a) representation of an I-Beam using wood-wood connections with the simplified calculation model (b) Axonometry of a TT joint (c) modelling of the TT joint with beam elements and springs.

The effective bending stiffness (EI_{ef}) of this type of structural element was analyzed to understand the corresponding mechanical behaviour and to assess the calculation model. Therefore, experiments were carried out on TT joints to determine the contact stiffness of the joint, and four points bending tests were performed on large specimens of eight meters made out of oriented strand board and laminated lumber veneer panels.

The EI_{ef} of the model was 12% lower compared to the test specimens, while a fully rigid model was 42% higher. The results show the importance of the semi-rigidity of TT joints. However, friction is not considered within the model which might explain the difference between model and tests. Concerning the tensile failure mode in the bottom flange, a shear lag coefficient should be applied due to the non-uniform stress distribution.

Acknowledgement

This research was supported by the NCCR Digital Fabrication, funded by the Swiss National Science Foundation (NCCR Digital Fabrication Agreement \#51NF40-141853).

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Mechanics of timber – to - timber shear connections with metal fasteners considering perfect plasticity and large deformations: The rope effect

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The ultimate load bearing capacity of timber - to - timber shear connections with metal fasteners depends on two quantities, firstly, the fully plastic embedment capacity of timber, parallel to the fibre (plastic limit line load, force per length), and secondly, the fully plastic bending moment capacity of the metal fastener (plastic limit bending moment, force times length). The underlying theoretical mechanical model represents perfect rigid plasticity from the material point of view combined with the small deformation setting from the kinematics point of view. This requires perfect ductility of the timber embedment mechanism as well as perfect ductility of the metal fastener bending mechanism, over relevant ranges of deformation. Based on these assumptions the spectrum of practically relevant shear connection configurations can be analyzed and the related perfectly plastic shear connection capacities can be analytically determined. The latter quantities have a typical formal appearance since they result as solutions of quadratic equations [1].

There are several striking model features which come into play as consequence of the chosen perfectly plastic model framework, as follows: 1. The metal fasteners act as statically determinate initially straight beams, 2. these beams are continuously loaded in transverse direction by field - wisely constant, oppositely directed embedment limit line loads, 3. these beams have no explicit fixed support conditions and 4. they are governed by equilibrium conditions only, 5. the bending moments are piecewisely quadratic, according to the loading, with smooth transitions, since there are no concentrated transverse loads in the interior of the beams, 6. the bending moments are limited by the plastic limit condition, which demands vanishing shear force at these distinct locations, i.e. the extremal property of the limit bending moment. These ideas are well known today and go back to the pioneering works of Johansen and others [1 to 4].

Increased load - carrying capacity can be achieved when the fasteners (bolts, self - tapping screws) are built - in at initial angles $\alpha_0 > 0$ against the normal to the timber - to - timber shear plane [6]. In this case an additional load - carrying mechanism is activated. Relative tangential displacements occur between the inclined fastener axis and the surrounding timber, based on elementary geometrical considerations. This leads to the build - up of a fastener - normal - force - based force - transmission - system between the timber components, where the effectivity of force transmission is clearly controlled by the actual force transmission capacity between fastener and surrounding timber (bolt friction, bolt head pullthrough, screw pullout etc). The Johansen load - carrying mechanism which is exclusively based on fastener - transverse - forces remains unaffected by this additional normal - force - based mechanism. The inclination of the fastener axis is accounted for by introducing the geometrical and material quantities related to this inclined fastener axis into the otherwise unaltered Johansen expressions. Finally, both of these statically determinate load -carrying mechanisms join together additively in global force equilibrium of the shear - connected timber components.

At this very point, now the large deformation effect is brought into play. In the present context geometric nonlinearity manifests itself in a uniform finite rotation ϕ of the central segment of the continuous fastener, i.e. which crosses the shear plane and extends between neighbouring plastic hinges [2, 3, 5]. Thereby the shape of the fastener axis changes from an initially straight line (with $\alpha_0 = 0$ or $\alpha_0 > 0$) to a zag- zig- zag line with two finite kinks at the locations of the plastic hinges. The consequences of occurrence of this finite deformation mechanism are elaborated upon in detail in this paper, on the general background of perfect rigid plasticity, leading to the genuine "rope effect" as will be shown. The formal framework, established for the geometrically linear situation before, remains completely unchanged. The angle α_0 has simply to be replaced by $\alpha = \alpha_0 + \phi$ for the central segment of the fastener, other things staying equal. The result presented in [7] is incorrect; it contradicts the requirements of the basic perfect plasticity model assumption.

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ISBN: 978-91-88898-64-7

3D Finite Element Model for Shear Stiffness of Wood-Wood Connections for Engineered Timber Panels

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With the development of automated design and fabrication tools, researchers have shown a growing interest for traditional timber joining techniques. Innovative wood-wood connections, referred to as integral mechanical attachments, have been developed and successfully applied to full-scale building structures. However, it has been shown that these connections highly influence the mechanical behavior of structures. The mechanical behavior of the connections themselves therefore needs to be investigated. Numerical modelling is largely used for complex structural analysis problems when analytical solutions are either cumbersome or non-existent. Furthermore, it has the potential to replace expensive and time-consuming experimental tests, for which a limited number of parameter combinations can be tested. For the numerical modelling of timber connections in particular, various studies have been carried out because of the difficulty to achieve analytical models. However, numerical modelling of timber remains arduous due to the anisotropy and inhomogeneity of timber, with largely different mechanical properties parallel and perpendicular-to-grain. In this paper, a 3D finite element model predicting the shear stiffness of wood-wood connections is presented. It is based on models developed by Sandhaas for timber joints [1] and Roche et al. for the study of the rotational stiffness of multiple tab-and-slot joints [2]. In this paper, instead of modelling each layer of engineered timber panels with its orientation (longitudinal layers at 0° and crosswise layers at 90°), a single material with the thickness of the timber panel was used, allowing to extend the model for a larger range of engineered timber panels. The model was compared to experimental tests performed on a shear testing setup. Promising results were obtained when comparing experimental tests to the numerical model. However, the model showed limitations for some configurations of joints.

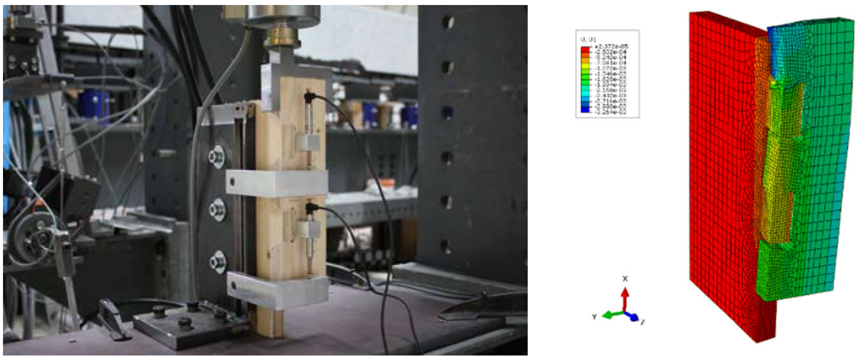

Figure 1: Shear testing setup [3] *(left)* and 3D finite element model of the tested configuration *(right)*

Acknowledgement

This research was supported by the NCCR Digital Fabrication, funded by the Swiss National Science Foundation (NCCR Digital Fabrication Agreement \#51NF40-141853).

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Connection stiffness and vibration transmission in timber frame structures

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Timber-frame structures are widely used in Sweden for the construction of single-family houses but also for apartment buildings of up to four floors. The production of timber buildings is using a high degree of prefabrication, for both office and residential purposes. Floor and wall panels are manufactured in workshops under controlled conditions. The jointing of these elements needs to fulfill functions such as load transfer, tightness and sound insulation.

Two joint projects investigated the connections between floor and wall elements in such timber frame structures in an experimental study of the static and dynamic performance under serviceability loading [1]. Manufacturers delivered multiple different variants of their systems (four in total) of two different types: with the walls standing on top of the floor as well as with the floor attached sideways to the wall. The elements used original dimensions, choice of materials and connectors whilst the overall dimensions were reduced in order to fit the test machine (see Figure 1, left, for an overview of one of the set-ups). Modifications to the base-variants led to in total 13 different set-ups. Thereby, parameters such as the number and spacing of the connector screws, additional vertical dead weight or the influence of a silicone bond on the connection area were varied.

In the dynamic tests, the floor elements were excited in a frequency sweep between 10-150 Hz. In total 21 accelerometers on the wall and floor recorded the response of the structure. In the static tests, the wall elements was loaded inwards with local measurement devices positioned in the joint region as well as a contact-free displacement measurement system at the backside.

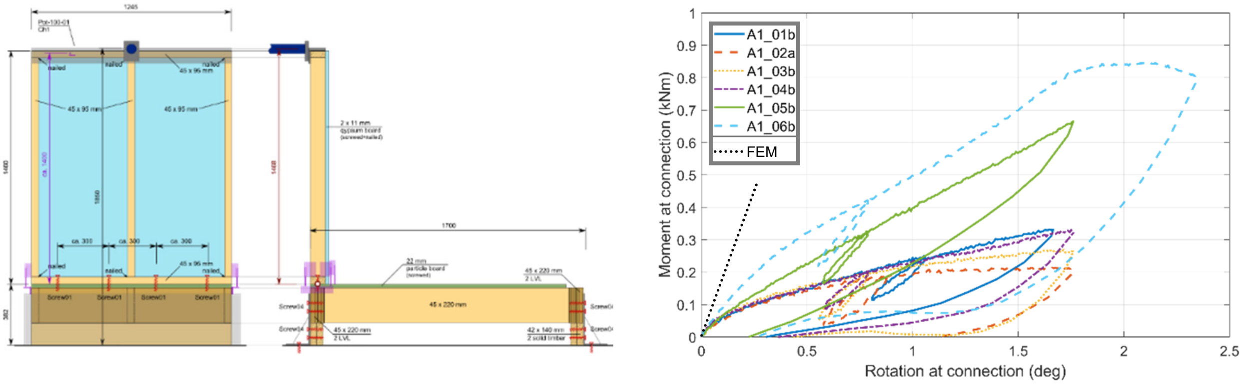

Figure 1: Geometry of one of the set-ups (left) and the moment-rotation curves for the variants of the same setup (right).

The results of the static loading were the moment-rotation curves from which the rotational stiffness in the connection was determined (see Figure 1, right, for one of the set-ups); the results from the dynamic loading were eigenmodes and the insertion loss. The positioning and the number of screws in the variants did not have a significant influence on neither the static nor the dynamic performance for the set-ups with the wall on top of the floor elements. The weak point in the set-ups was the nailed connection between the bottom rail and the vertical studs. Replacing these with screws increased stiffness significantly. A general conclusion is that a stiffer connection is less optimal for transmission of vibrations (and vice-versa). Still, this may not be generalized to other situations and dimensions of such structures.

Finite Element simulations were performed additionally using Abaqus. The model recreates the static experiments by using solids for the timber parts as well as the wall sheeting and the flooring. Contact behavior with friction is introduced between all parts of the model. Connectors such as screws and nails are modelled using beam elements using embedded region constraints for connecting them to the individual parts. The initial stiffness of the connection in the very beginning of the loading can be reproduced with the numerical model and the applied simulation techniques. Nevertheless, it becomes obvious that the actual behavior is non-linear from a very early stage on, most likely due to the behavior of the nails/screws in the connection between the studs and the bottom rail.

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 ISBN: 978-91-88898-64-7

Historic wood applications

ISBN: 978-91-88898-64-7

Studies for the Mona Lisa conservation: the implementation of its panel's Digital-Twin

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Since 2004, the wooden panel of the "Mona Lisa" painting by Leonardo da Vinci has been studied by an international research group of scientists [1,2] and several experimental campaigns have been carried out to understand its characteristics and provide information for the Artwork's conservation [3]. Among these, the implementation of its "digital twin" is here presented as a fundamental step for its conservation. Indeed, this digital twin is providing a deeper understanding of the mechanical characteristics of the panel, and after it has been accurately calibrated, it will provide the means for evaluating the stress states which the Artwork undergoes when the surrounding climatic conditions vary. Moreover, it will allow to evaluate in a non-invasive way the effects produced on the Artwork by any changes of its framing conditions, including the internal stresses or the external forces that it can bear without damage.

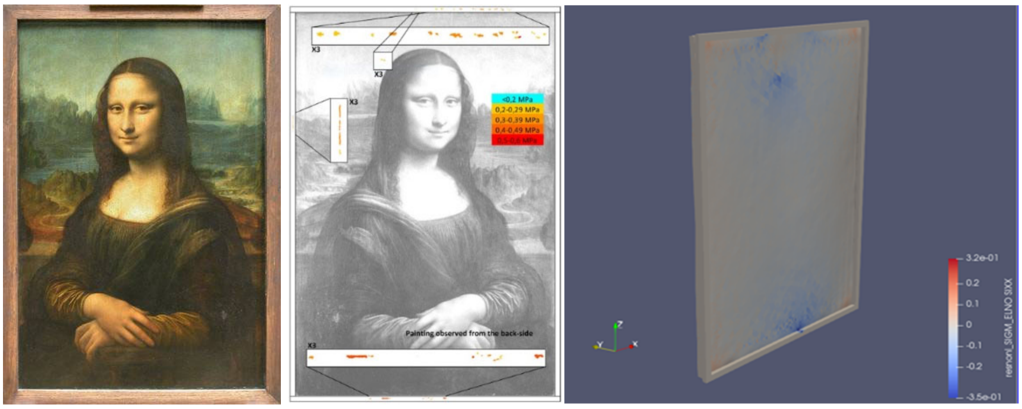

Figure 1: (left) the Mona Lisa panel inserted inside its frame; (centre) the contact areas between the panel and its frame, detected with pressure sensitive film; (right) the panel's numerical simulation to assess the stresses (σ_{xx} in MPa) induced in it by the contact forces.

The implementation of the digital twin, through a Nelder-Mead optimization scheme, starts from the definition of the shape of the Artwork, through optical methods [4], and the identification of the boundary conditions, through an experimental campaign based on the use of a film sensitive to pressure [5]. During the whole year, while exhibited in the conditioned display case, the panel continuously tends to deform, due to the slight variations in the surrounding environment. Its mechanical behaviour is automatically monitored and recorded every 30 minutes by an ad hoc equipment placed close to its back face: four miniature load cells (located at the four corners) measure the forces pressing it against the frame, and three displacement transducers measure its deflection at mid height. Finally, a method is here described to compare two different framing conditions, through the numerical computation of point-by-point stress and deformation differences, in order to provide information for optimizing settings and constraints to Conservators.

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ISBN: 978-91-88898-64-7

A preliminary numerical analysis study on the oriental historic timber-frame buildings

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The oriental historic timber buildings in Taiwan are mainly composed of timber frames and load-bearing brick walls. As brick walls are generally perceived to be stronger and stiffer than its timber frame counterparts, hence brick walls are often regarded as the primary structural components responsible for countering lateral force during seismic analysis. Due to the complexity of oriental timber frame construction and the lack of proper seismic evaluation methods for this type of traditional structures, the conventional practice is either to neglect or to set all timber connections as hinge, by assuming these connections bear no moment resistance. However, from past experimental results [1-5], it is noted that oriental timber joint connections generally performed more like semi-rigid joints than hinges. The joint stiffness and strength of these traditional joints are not only found to be closely related with joint designs and vertical loads, these timber joints also tend to slip when subjected to lateral force. Thus, if the above observations are not considered during seismic analysis, misunderstanding of the actual deformation pattern might subsequently lead to underestimation of the overall performance of the oriental timber structures.

Base on our team's past experimental works on the traditional Dieh-Dou type timber structures [2-5], a general understanding on the structural behaviour and fracture patterns of the global and critical joint systems have been established. Although mechanical models derived from the above test observations are generally in good agreement with the static test results, more verifications have to be carried out to find out if the assumptions made are well validated (Figure 1). Hence in this study, the above mechanical models were subjected to two types of 'real-life' test trials. In the first instance, the models' assumptions and calculated parameters were cross-referenced with shaking table tests of a partial Dieh-Dou type timber frame structure to check for its validity. Next, with the use of structural analysis software, these calculated assumptions were further applied onto two existing Dieh-Dou type timber frame monument buildings to generate prediction models (e.g. push-over and time-history analyses) and anticipated weak points of the monument timber buildings. These predicted outcomes are then compared with existing test results and post-reconnaissance earthquake reports for cross validation. Preliminary results from the above test trial revealed that the assumptions made for the numerical analyses are generally in good agreement with the experimental results, and the predicted weak points of the timber structural frame mostly concentrated around the Dou and its adjoining members.

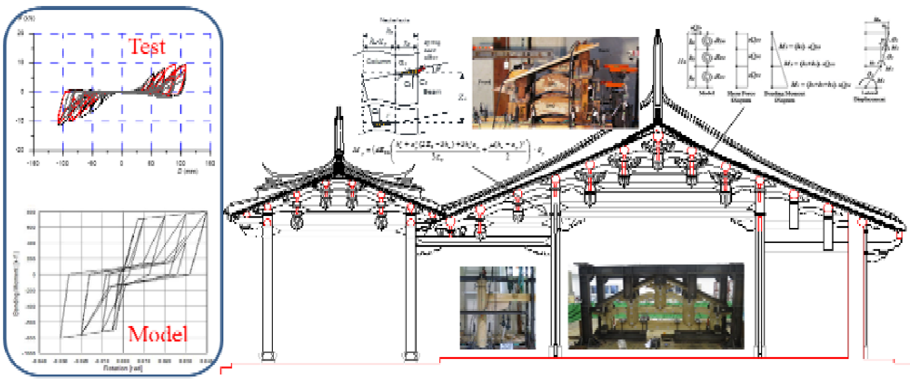

Figure 1: Brief overview of the structural tests conducted and mechanical models proposed.

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ISBN: 978-91-88898-64-7

Blockhaus buckling analyses: Numerical and analytical models to evaluate the critical load

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The problem of instability has been dealt with at the mathematical as well as at engineering level since ancient times; a slender columns stressed by an axial compression force, undergoes to an instability phenomenon called instability for peak loads.[MF1]

Over time, this type of instability has also been found in other slender structural elements, equally stressed, behaving as slabs and plates such as profiles and panels; these elements have been the subject of study by various Authors.

The wooden structural elements, due to the anisotropy of the material, the presence of defects and deviations of the grain, are particularly sensitive to this problem. This work investigates the elastic instability of Blockhaus walls, which are prone to the phenomenon of instability for peak loads because of the mechanical factors described above but also for the lack of metallic connections that solidarize the constructive elements [1]. Two different mechanical models (fully discrete and longitudinal continuous-transversal discrete) have been analytically formulated in order to estimate the critical load of walls without openings; two different load cases (concentrated and distributed loads), top restraint conditions (restraint and unrestraint) and lateral boundary restraint (simply supported and clamped) have been considered. The results obtained analytically have been compared with those deriving from numerical analyses on 3D elements implemented in ABAQUS software package and with previously conducted experimental tests [2]. Afterwards, to increase the accuracy of proposed analytical models, the effect of small notches and grooves has been added. Geometrical factors causing a decrease in critical load such as load eccentricity and geometric imperfections have been considered.

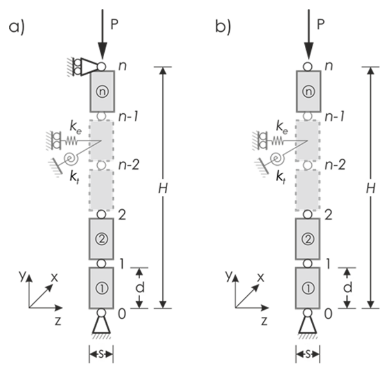

Figure 1: Fully discrete mechanical model of the wall with a) Top restraint condition and b) Top unrestrained condition

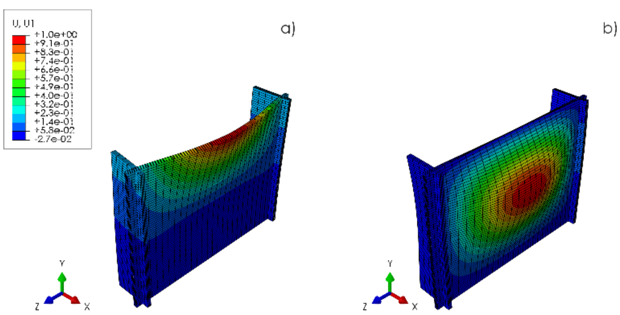

Figure 2: Linear Buckle Analysis first mode shape under distributed load for a) Top unrestrained condition and b) Top restraint condition obtained by ABAQUS Software Package

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 ISBN: 978-91-88898-64-7

Design of the Double Step Joint to account the Shear Crack with Cohesive Surfaces

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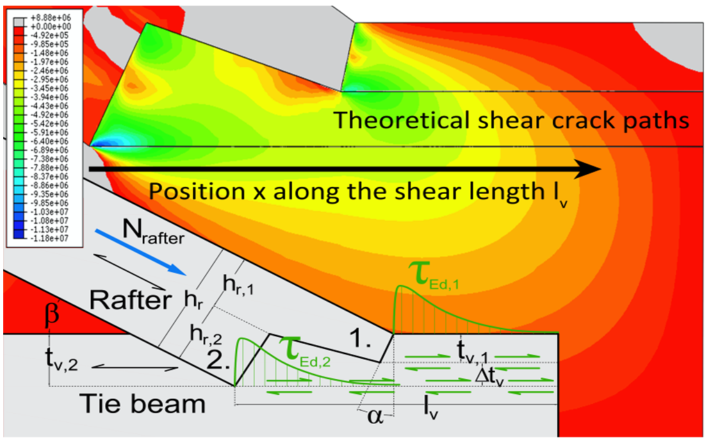
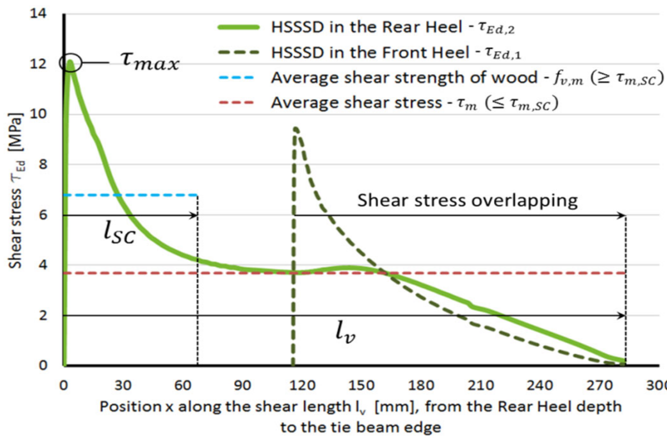
With low rafter skew angles β, the Double Step Joint (DSJ) can be subject to high horizontal thrust, alike the Single Step Joint (SSJ), resulting in the shear crack at the heel depth t_v over the shear length l_v in the tie beam. In order to prevent this brittle failure mode, design equations (1)-(2) from the literature review [1] must be checked in both DSJ heels (i.e. *i*=1 for the Front Heel; *i*=2 for the Rear Heel).

$$N_{rafter,i} \leq k_{v,red,i} \cdot f_{v,m} \cdot \frac{b \,.\, \min(l_{v,i},\ 8.t_{v,i})}{\cos\beta} \qquad (1)$$

$$N_{rafter,1} \leq N_{rafter,tot} \leq N_{rafter,1} + N_{rafter,2} \qquad (2)$$

Being conditioned by the geometry of both DSJ heels, the total rafter load-bearing capacity ($N_{rafter,tot}$) of the connection should vary between the rafter load-bearing capacity in the Front Heel ($N_{rafter,1}$) and the sum of rafter load-bearing capacities related to both heels ($N_{rafter,1}$+$N_{rafter,2}$). The present numerical research aims at investigating which DSJ geometrical parameters may trigger the shear crack in the tie beam (Figure 1), over the shear lengths along the grain $l_{v,i}$ at the Front and/or Rear Heel depths $t_{v,i}$. Alike previous numerical studies [2, 3], the Cohesive Surfaces method has been chosen to simulate the emergence of shear crack in both DSJ heels through modifying their geometry. Furthermore, their two respective non-uniform shear stress distributions, called Hammock Shape Shear Stress Distributions (HSSSD), have been assessed through several parameters (Figure 2).





Figures 1 and 2: Shear stress along the grain in the tie beam (left). Pattern and parameters of the HSSSD in the Rear Heel (right).

As outcomes from this numerical research, different scenarios of shear crack emergence in both DSJ heels as well as their design equation variants related to (2) have been determined, through modifying two geometrical parameters: (i) Difference of heel depths Δt_v=$t_{v,2}$-$t_{v,1}$; (ii) Ratio between the Rear Heel and rafter heights $h_{r,2}/h_r$. Based on empirical relationships already established for the SSJ design [3], the reducer coefficients $k_{v,red,i}$, which take into account the presence of HSSSD in the Front and Rear Heels, have been checked and adjusted to enhance the reliability of the DSJ design equations (1) against the shear crack in both heels.

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ISBN: 978-91-88898-64-7

Fracture analysis of single-shear joint equipped with oak dowel loaded perpendicular to grain with eccentricity

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A common damage of tie beams and rafters is found in historical structures: decayed beam ends due to water leakage or insect-related issue. Lap joints for replacement of such parts could be a suitable solution. Since the combined loading is often found in the structural members, **the design of the lap joint is not an easy task**. The design of lap scarf joints for tensile and compressive loading in the direction along the grain is possible, because the existing codes partly cover the topic [1]. Unfortunately, combined loading induces a complex stress state on the coupling elements, which is not fully covered in standards [2], especially when concerning surrounding matrix failure. In the case of momentum (see Fig. 1) or combined loading, the task is more complicated. The requirement of cultural heritage institutes is often to preserve the maximum amount of the original material and aims at the use of historical traditional carpentry approaches which often deal with the all-wooden joints, meaning the ones equipped with coupling elements made of wood. English oak, as a widespread hardwood specie in Europe, is usually selected for the dowels. The bearing capacity of the oak dowels in the direction parallel-to-grain is significantly lower when compared to the steel coupling elements. However, oak dowels do not suffer the issues related to incompatibility of the materials, the fitting could be adjusted using pre-dried dowels compensating the wood shrinkage, and, finally, provide nearly invisible look suitable especially for exposed parts of the structure. Therefore, the task covered in the paper is clear: **assess the bearing capacity of oak dowel** connecting two softwood elements (Norway spruce) **loaded eccentrically perpendicular-to-grain** using LEFM approach and numerical modelling. The results are compared to the experimental outputs.

Numerical model is developed using ANSYS Mechanical APDL with geometry given in Fig. 1. A linear material model is considered (limited plasticity models available) with orthotropic material model for the dowel and surrounding matrix (English oak, Norway spruce, respectively) according to [4, 5]. The contact between the dowel and the matrix is defined and the friction coefficient of μ=0.4 is used. The same strain energy release rate as in [5] is adopted, i.e. $G_c = 300\ \mathrm{Nm}^{-1}$ (opening mode I present). The mesh is made: 1) uniform, 2) densified around notch tip. Both mesh types are evaluated in the results. Unit load (F) is applied and following load steps were considered: a) loaded system, b) the same system with the first nodes at the crack propagation zone sequentially released using coupling equations from the inside to the outer edge on both members simulating the crack growth Δx. The change of system compliance is computed from the load steps. Based on the fracture mechanics relation between energy needed to create a new surface of length Δx and change of system compliance the critical force $P_{90,Rk} = \sqrt{2G_c b\Delta x/(\Delta u(\Delta x)/F)}$ of the onset of crack is computed. The force is compared to the results of parallel-to-grain values [1] and to the results of steel dowels [5]. The detailed results will be presented at the conference.

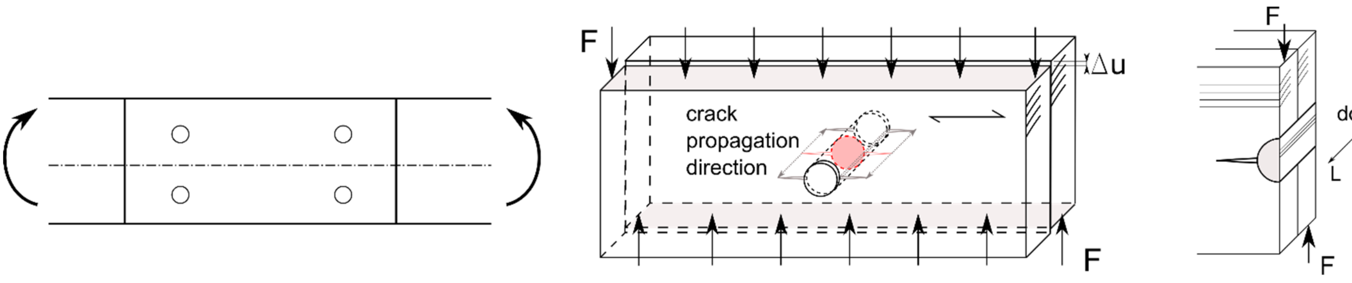

Figure 1: The configuration of the stress state (left), problem definition (rest)

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 ISBN: 978-91-88898-64-7

Simulation and testing

ISBN: 978-91-88898-64-7

Hygro-mechanical modelling of glutin-based bond lines in wooden cultural heritage

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A comprehensive modelling of the transient hygro-mechanical behaviour of complex wooden structures by the finite element method is targeted. New methods and material models for glutin-based bond lines are developed, since bond lines proved to have significant influence on moisture transport and fracture behaviour. The modelling of wood is based on previous research on three-dimensional orthotropic elastic, plastic and fracture formulation on the mechanical side, and on multi-Fick'ian moisture transport formulation on the hygric side. The coupled monolithic description of transient hygro-mechanical behaviour of wood is based on previous research (see [1]).

By the contribution, a new multi-Fick'ian hygro-mechanical model for glutin-based bond lines [2] is presented. A cohesive element model is modified by adding features to simulate the mechanical behaviour of and the moisture transport in the joint. The joint, which includes one layer of adhesive and two layers of interphase zones, is simplified by using a single layer of cohesive elements. The material properties of the adhesive, the interphase zones and both wooden adherents are assigned to the cohesive element. Thus, the bond line will fail, for instance, by exceeding the minimum strength with respect to material, direction and moisture content. This method decreases computational effort with a reduced number of material and element layers, while still maintaining the reliability of the model. The investigated glues are gelatine-based adhesives, which are often used in wooden cultural heritage objects found in museums and collections. Further references on the basing experimental investigations of the hygro-mechanical properties of gelatine-based adhesives and the element and material formulations are described comprehensively in [2].

The new bond line model is validated by numerical investigations of two wood species, based on own and further experimental studies available in the literature. The simulated specimens contain a single bond line of animal adhesive. Hygro-mechanical shear tests (Figure 1) and diffusion experiments are simulated and compared to the experiments. The validations show that under mechanical loading, the numerical simulation of fracture under tension and shear is in good agreement with the experimental results. However, the numerical results of transient moisture transport are less precise. The reason is primarily due to the incomplete experimental data, which serve as input parameters into the simulation.

Finally, the methods are applied to the hygro-mechanical structural analysis of wooden cultural heritage objects at changing ambient relative humidity. The barrier effect of the bond lines leads to large moisture gradients and internal hygro-expansional constraints at the adhesive layers. The results show the high potential of cracks around the bond line.

The new bond line model enables to consider the influence of adhesive layers on fracture behaviour and resistance on moisture transport within structural analysis of wooden cultural heritage objects. Further experimental research on hygro-mechanical fracture behaviour and moisture transport in all members of the investigated structures are required to enhance the accuracy of the simulation results. Keeping in mind that every model is limited, simulation results can help conservators to evaluate constructions, detect overloaded structural members in a non-destructive way and with that develop conservation measures and define climate conditions.

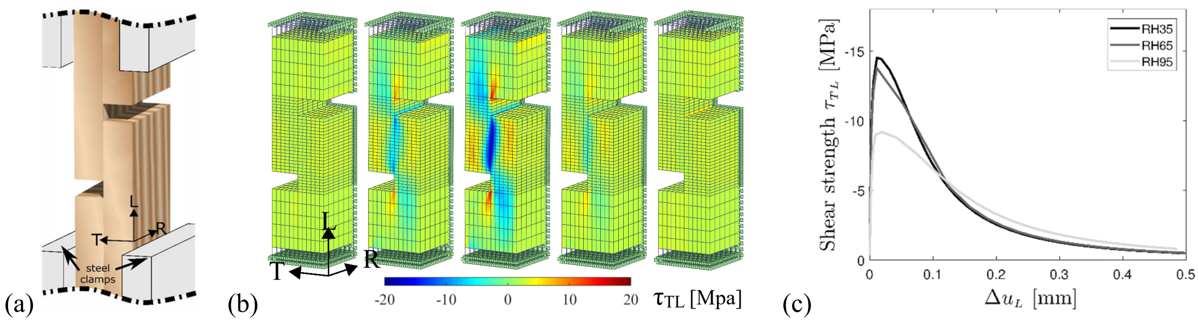

Figure 1: Shear test of adhesive-wood joints (glutin-based glue and European beech): (a) experimental setup; (b) shear stress (τ_{TL}) propagation (from left to right): initial (u_L=0 mm), elastic, peak before failure (u_L=0.196 mm), directly after failure, final condition (u_L=0.5 mm); (c) dependency of the shear stress on moisture [MC_{beech}(RH): 6.8%(35%), 12.1%(65%), 23.8%(95%)] [2]

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 ISBN: 978-91-88898-64-7

On the need for reliable rolling shear characteristics in CLT lamellas and for efficient related test methods

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Effective modeling of structural behavior of cross-laminated timber (CLT) elements requires reliable input on the mechanical properties of its laminations. The cross-lamination of layers provides for dimensional stability of CLT elements. In this arrangement, however, all laminations in shear walls and the layers of floor elements oriented perpendicular to the major strength axis transfer shear stress in the radial–tangential plane, often referred to as rolling shear. It is among the least documented characteristics of wood, since it had been of marginal interest for structural lumber and engineered wood composites until the emergence of CLT.

While the numerical models may easily account for the contribution of rolling shear in the immediate and long-term deformations of laminated panels, simulations are charged with wide margins of uncertainty because of shortage of reliable experimental data. Rolling shear is not the easiest property to measure, and it received only limited coverage in the literature [1-7]. What has been documented was that the rolling shear strength and stiffness in the cross-layers in CLT floor panels is related to the species, density, growth ring orientation, and manufacturing parameters, but there is no evidence for a meaningful correlation with the grade of lumber, whether established by visual or machine grading.

In the presentation, we will discuss the pressing need for reliable data on rolling shear characteristics in clear wood and in structural lumber, their statistical distributions in species important for CLT industry, as well as for efficient test methods to allow generation of relevant data in timely manner. Prototype methods and preliminary data will be presented.

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ISBN: 978-91-88898-64-7

Applying the XFEM method to the simulation of tensile failure in timber boards and finger-joints in a glulam strength model

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Numerical strength models for glulam have played an important role in the certification and standardization of glulam of different species [1]–[3]. A key aspect of such models—although normally not referred to as such—is the chosen failure criterion which determines the bending strength of the simulated beams. In the past, different simplified (brittle) failure criteria, based i.a. on the Weibull weakest-link theory, have been used to model both, failure of the wood and finger-joints (e.g. in [1], [2], [4]). Fracture mechanics, due to its inherent rather high computational demand and associated complexities, has been left out of such models until recently. Investigations on crack propagation on finger-joints in glulam beams using fracture mechanics was studied i.a. by 5, but it were 6 who applied this concept to a glulam stochastic strength model. For this, a finite element (FE) model with quasi-brittle failure behavior was developed, using a softening curve for the stress-displacement relationship and fracture energy. The results of the model indicated a much better representation of the size effect observed in experimental tests as compared to simply applying a brittle criterion, especially for smaller cross-sectional depths between 100 mm and 600 mm. This model applies the fracture criteria at the material level of each finite element, which, contrary to surface approaches (e.g. cohesive zone), makes the parameters—in this case the fracture energy, G_f—more sensible to the mesh size.

The relatively recent development of the extended finite element method (XFEM) and its implementation in commonly used FE software, has added one more option to be considered for the modelling of fracture mechanics problems (linear and nonlinear). The main advantages of this method, among others, consist on a mesh independence regarding the location of the crack, while allowing for surface-to-surface damage criteria (similar to a cohesive zone). These aspects, added to a simple implementation from the user's perspective, makes this method worthy of consideration for glulam strength models.

As a part of a research project dealing with the development of a glulam strength model for hardwoods, the applicability of XFEM to the simulation of tensile failure in board and finger-joint segments was performed. It was found that the method produces good results, comparable to experimental tests in glulam beams, where the size effect is well represented. A parametric analysis was performed to observe the effects of mesh size and G_f on the computed bending strengths. It was found that a mesh size of 1/3 of the lamination thickness, t (for an investigated t = 20 mm) was a good compromise between accuracy and solving time. An increase in the fracture energy of the finger-joints from 6 J/mm^2 to 20 J/mm^2 produces a somewhat linear increase in the characteristic bending strength, $f_{b,k}$, of (from 35.7 N/mm^2 to 41.2 N/mm^2) for a small beam depth of 100 mm.

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ISBN: 978-91-88898-64-7