COST Action FP1004

Innovative Timber Composites - Improving wood with other materials

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TTR Centre/Frederick University,
Nicosia, Cyprus

Edited by
Kay-Uwe Schober

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Enhance mechanical properties of timber,
engineered wood products and timber structures

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Preface

Timber and wood-based engineered products are important as structural materials, especially in the drive towards sustainable technologies and construction. It is very important to improve the properties of these products, making them more competitive and reliable. This applies particularly to larger, more complicated structures where timber is becoming a realistic alternative to other construction materials.

The orthotropic properties of timber are well known - strong and stable along the grain, weak and susceptible to movement across the grain. Combining wood with other materials, to achieve composite action, enables the strength of wood to be realized and its shortcomings to be strengthened or eliminated.

COST Action FP1004, “Enhance mechanical properties of timber, engineered wood products and timber structures” provides a network for learning and development in a range of connected topics. This conference will present and record various methods for improving wood with other materials. It brings researchers and practitioners together to learn about various applications, and enables them to take part in discussions about future research and development.

The purpose of the COST program is to strengthen Europe and other countries in scientific and technological research, for peaceful purposes, through the support of cooperation and interaction between European researchers. This conference adheres to these principles. It will be an extraordinary opportunity to hear presentations from highly specialist, invited speakers and to participate in debate, in the island of Cyprus, which is located at the crossroads of three continents Europe, Asia and Africa in the Northern-east part of the Mediterranean Sea.

Richard Harris

Chairman of COST FP1004

Michalis Socratous

Host and Director of TTR Centre

About COST Action FP1004

Timber and wood-based engineered products are becoming very important as structural materials, especially in the drive towards sustainable technologies and construction. For structural wooden products, it is very important to improve their properties to be more competitive and reliable as a sustainable low-carbon material and a major contributor to affordable buildings. This applies particularly to larger, more complicated structures where timber is becoming a realistic alternative.

This Action aims to boost the performance of structural timber products and construction, thereby improving use of timber in construction in existing and new applications. This includes the enhanced predictability and reliability of timber structures. Improving the mechanical performance of connections and reinforcing timber in weak zones are large-scale research domains in Europe, which will require coordination and scientific/engineering approaches. This COST Action will deliver increased knowledge of improving strengthening, stiffening and toughening techniques, modeling enhanced performance and experience in real projects to create new opportunities for timber construction. Exchanging information will highlight gaps in knowledge and inform future work and potential collaboration between research groups, supporting timber construction and its wider uptake in the European construction industry. This Action may also create opportunities for patenting possible new technologies and products for reinforcing timber mechanical properties.

The scientific program is divided into three main scientific areas expressed as Work Groups (WG) with the same aims but different perspectives:

WG 1: Enhance performance of connections and structural timber in weak zones

WG 2: Enhance the mechanical properties of heavy timber structures with particular emphasis to timber bridges

WG 3: Modeling the mechanical performance of enhanced wood-based systems

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Short Rotation Forestry in Cyprus

Michalis Socratous[1]

Summary

The island of Cyprus is strategically located at the crossroads of three continents, Europe, Asia and Africa in the Northern east part of the Mediterranean Sea. It has a population of about eight hundred thousand and an area of nine thousand two hundred and fifty one square kilometers (9,251).

As described by the ancient authors in classical times, Cyprus was rich in forests and they called the island «δασόεσσα» (dasoessa), that it means full covered by forests. Nowadays, due to the deforestation through the centuries the forests cover only 18.6% of the total area (172 000 Ha). Cyprus is a very small producer of forest products and for this reason is net import country for timber and wood products. The round wood production is limited due to the recent national law concerning the deforestation. The allowed cutting volume is not exceeding the 8,000 m^3 per year.

The Furniture and Woodworking industry are an important manufacturing sector of the Cyprus economy. Among other factors and due to the fact that Cyprus do not produce the needed quantities of wood raw material makes the industry not competitive with the result to face many problems.

Therefore, the general aim of this paper is dealing with these problems. From the scope for the survival of the Furniture and Woodworking industry that it means facing them and offering possible solutions and in parallel to secure the implementation of National and European Union targets in the fields of wood supply and protection of the environment.

1. Introduction

The demand for wood is growing worldwide but lack of wood is common in many countries and especially in dry land is more critical than in humid countries. In countries such as Cyprus where resources of wood are very poor due to climatic conditions and other factors, fast-growing tree species with less demand for water consumption can play a vital role in providing good supply of wood raw material.

The most effective way to meet growing demand for wood raw material or wood biomass is the establishment of fast growing tree plantations. This resource will act positively as the balance factor between supply and demand and will offer relief for the natural forests acting as a stabilizer. Short Rotation Forestry has been investigated since 1960s and is gaining more and more consideration and establishment both at world and European levels.

[1] Director of TTR Centre (Timber Training and Research Centre), Cyprus

Short Rotation Forestry (SRF) is defined as the silvicultural practice in which high density, sustainable plantations of fast growing tree species either produce wood biomass on agricultural fertile lands, wastelands or degraded lands generally outside the traditional forests. The trees are grown with single stems or as coppice systems, with rotation period of less than 20 years and with an annual wood production of at least 10 tones DM/ha. The biomass produced may replace the wood from traditional forest areas and is used for energy, paper and pulp, fodder, construction, bio-fuel and to produce electricity. SRF has also been found useful in amelioration of degraded sites; establishing vegetation filters to treat polluted waste water and sewage sludge, reforestation of clear felled old virgin forests, carbon sequestration, etc (IUFRO)

2. Research Project

The purpose of this project is to establish the feasibility and sustainability of using the SRF practice as a source for raw material for the woodworking and furniture industry. In addition, this study aims to establish output and information for operational scale establishment of SRF in Cyprus. There are plenty of possibilities to tie local development on the private forestry sector.

Species were selected by reviewing the literature and by evaluating species performance in older local trials. The results of the investigation had shown two species, which are the most, appropriate and have the biggest potential according to the project criteria. The two species was Melia Azedarach and Ailanthus Altissima but because of some other reasons, we took only the first one. The selected site was in Xyliatos, a village outside Nicosia, and the trees are planted last December. The trees are irrigated by one simple irrigation system and the water comes from the Xyliatos dam.

Fig. 1 The experimental plantation after eight months from planting

No significant problems were encountered. All the trees are growing well, no one dies and more of the trees (85%) are growing rapidly as expected. The plantation shows from the early beginning great success according to the project objectives.

The outputs of this project deriving from the research results will be the follow:

- The preparation of Best Practice Guidance for operational scale establishment of SRF in Cyprus.

- The preparation of cost and benefit analysis
- The preparation of the commercial viability assessment
- The preparation of the Environmental Impact assessment

3. Further development

Cyprus considered as a semi-arid country. Water is a valuable resource, which has to be used effectively and economically without wastes. Having in mind the above the next phase in our program is the establishment of another one pilot scale plantation. This plantation will irrigate by wastewater and sewage water. It is well known that domestic wastewater or sewage water contains nutrients in the form of phosphate and nitrate. By this way, we can replace the using of commercial fertilizers to provide the necessary nutrients to the trees. This is a new area of research in Cyprus and needs lot to be done.

4. Conclusions

This paper provides an overview of wood sector in Cyprus and sets out the starting point for Short Rotation Forestry. SRF could play a vital role in socio-economic development as employment generator and the driver by securing wood raw material for the woodworking industry. We should secure wood raw material for the woodworking industry but in parallel, it is our obligation to protect natural forests and the environment with the only way available, SRF practice.

By the research results, it will be managed to persuade the policy makers to understand the benefits and see the whole situation from another point of view. The governments support policies are needed since due to economic crisis there is an additional reason to drive the Cyprus economy to a new more dynamic model.

The Short Rotation Forestry was started and we move forward to develop a more sustainable future depending on the productive sectors of the economy, respecting the Mother Nature and the environment.

5. Acknowledgement

The author of this paper would like to express his sincere thanks to the Cypriot people for their support by adopting and by helping to plant the trees of the first SRF experimental plantation in Cyprus.

Summary of shear connector methods for timber-concrete composites

Alfredo Dias[1]

Summary

Composite timber-concrete structures are an increasingly popular solution for the rehabilitation/strengthen of timber floors, as well as, for new floors. The composite behavior leads to a solution with a higher bending stiffness and consequently lower deformations, on which tension stresses are mostly carried by timber while compression stresses are mostly carried by concrete. In this type of structures, the connection plays a critical role, once the level of composite action that is possible to achieve relies mostly on its mechanical performance. For these reasons, the search for new connections is one of the fields that have attracted more attention in terms of research. Different types of connection have been proposed for different conditions and applications (Fig. 1).

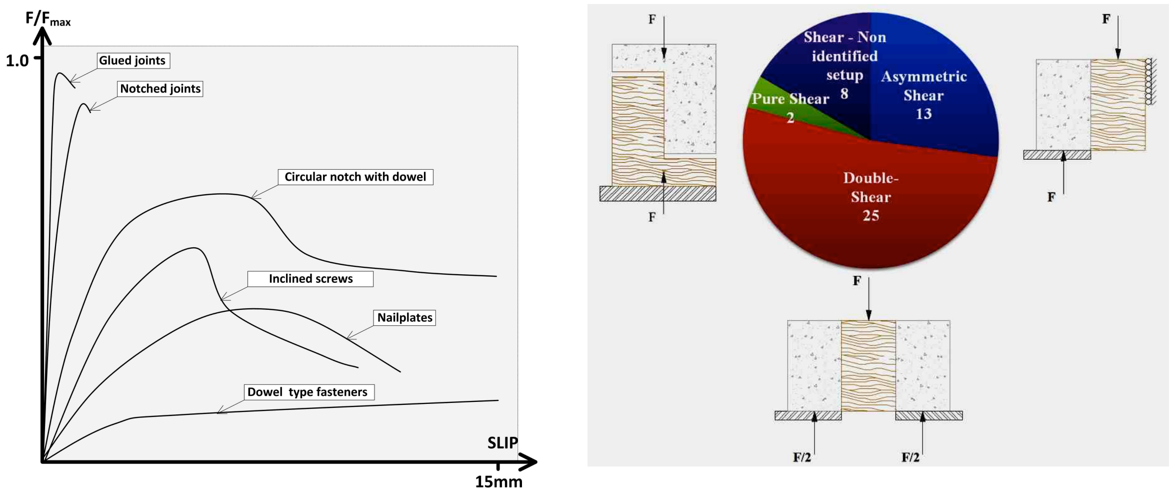


Fig. 1 Load-slip curves for the various connection types (left); most widely used test set up configurations(right)

In this presentation, the various types of connections that can be found to fasten timber and concrete will be presented and described. This information was obtained on a database which includes data from more than 1000 experimental tests [1], the number of tests, for the types of connection most often used, are listed in Tab. 1. The test results available were obtained from experimental work undertaken in many different laboratories located all around the world.

Due to the inexistence of a commonly accepted testing procedure, to test timber-concrete connections, different test set ups and loading/measuring protocols were

Tab. 1: Sample considered in the analysis

Connection	No. of tests
Dowel-type fasteners	263
Axially loaded fasteners	438
Notches	204
Notches combined with steel fasteners	51

[1] Civil Engineering Department, University of Coimbra Portugal

applied. Once, these conditions might have a non-negligible influ-

ence in the mechanical properties obtained, namely load carrying capacity and slip modulus, a detailed analysis was undertaken to identify it. This analysis was based on numerical results obtained with Finite-Element Models, specifically developed for this purpose, and on the experimental data available from the tests.

The properties for this type of connections are usually available in disperse studies whose main purpose is the assessment of a specific connection type for a particular application. Up to now, no comprehensive and global analysis has been made regarding the mechanical properties of timber-concrete connections. Furthermore, to date there are no statistical and reliability analysis data for timber-concrete connections. In this presentation, results of a statistical analysis aiming to assess the statistical distributions of the most relevant mechanical properties of the timber-concrete connections, slip modulus and load carrying capacity, will be discussed. With this type of information it is possible to estimate the characteristic values for design of timber-concrete composite structures according to EC5-1-1 Annex B [2], (see Fig. 2).

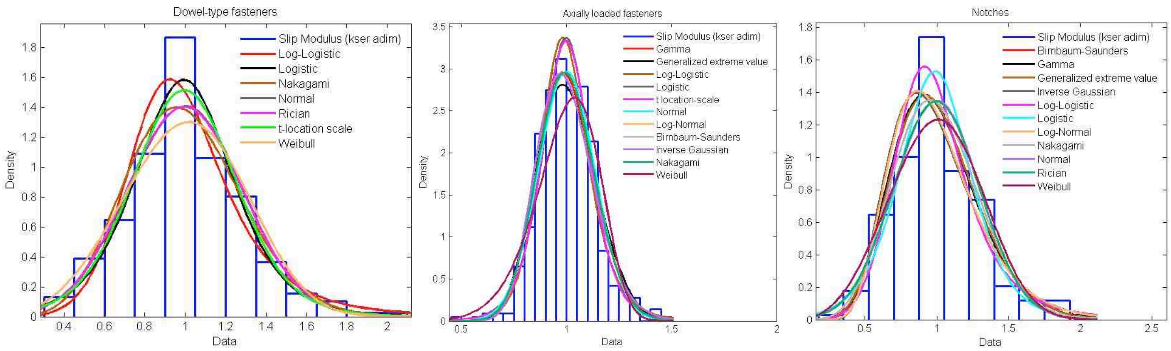
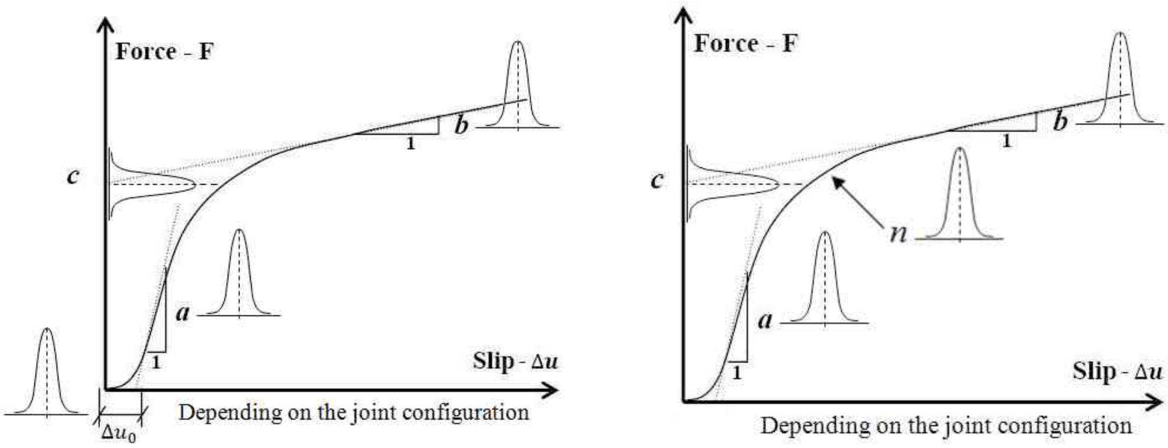


Fig. 2: Example of histograms for the connection stiffness of various connection types

Additionally, mathematical models are proposed to describe the load-slip behavior of connections, aiming advanced material non-linear analysis and reliability analysis of timber-concrete composite structures. The experimental load-slip results were fitted to five load-slip mathematical models, available in the Bibliography (see Fig. 3).



(A) (B)

Fig. 3: Examples of two descriptive models used and the corresponding parameters: (A) model with 3 parameters and the initial displacement (Foschi's model); (B) model with 4 parameters

The parameters obtained were subjected to a statistical analysis in order to assess the most adequate statistical distributions in each case. The results obtained clearly show that is possible to obtain good descriptions, with most of the proposed models. Furthermore, for most of the connection types, available in the bibliography, good descriptions could be achieved with an exponential 3 parameter model. Additionally, statistical analysis showed that most of the parameter distributions are either normal or lognormal. These results are presented and discussed.

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[1] Monteiro, S.R.S., Dias, A.M.P.G. & Negrão J.H.J.O. (2013). Assessment of Timber-Concrete Connections Made with Glued Notches: Test Set-up and Numerical Modeling. *Experimental Techniques*, 37(2).

[2] CEN, Eurocode 2 - Design of timber structures - Part 1-1: General - Common rules and rules for buildings. 2004, CEN: Brussels.

Concrete-based adhesives used in connections

Kay-Uwe Schober[1], Michael Drass[2]

Summary

A new-type jointing system for truss structures have been developed based on the experiences made with glued-in rods over the past years. The connecting technology uses concrete-based adhesives (CTA) and larger drill-holes or slots to overcome the uncertainness of the bondline quality and the structural performance of missglued connections. The jointing system is suitable for pre-fab truss structures as well in structural rehabilitation of traditional flooring systems and slabs. Due to preparation on site, the CTA adapts exactly the joint surface, whatever its look like.

1. Introduction

In timber engineering the joint is generally the critical factor in design. Here, the strength of the structure will be determined by the strength of the connection. The stiffness greatly influences the displacement behavior. Member sizes are often determined by number and physical characteristics of the connectors used, rather than by the strength requirements of the material.

Different types of mechanical fasteners for truss structures, like glued-in rods, have been developed in the past years. In most cases, the full structural capacity is not used. Only two third of the applied loads are transmitted through the joint into enclosed members due to a low embedding strength and reduced load-carrying capacity when loaded under an angle to the grain.

Glued-in rods became very famous in timber engineering due to the high stiffness and fire-resistance, the resistance to corrosive atmosphere and the good aesthetic appearance. Under the improvement of glued-in rods by counteracting the negative aspects like the stringent quality control, the high assembling effort and of course the geometrical restrictions [1], this mechanical fastener got a large potential in timber engineering with relation to economic, scientific and ecologic aspects.

When it comes to quality control, an uncertainness of the bondline quality and the structural performance of missglued connections still exist. Therefore, a further step to develop new-type connections using concrete-based adhesives (CTA) and larger drill-holes or slots have been made.

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[2] Research Associate, Mainz University of Applied Sciences, iS-mainz - Institute of Innovative Structures, Mainz, Germany

2. Applications

Glued-in rods are often used for overhauling damaged structures; a typical field of operation is the strengthening of new structures (Fig. 1). Most design codes and approaches based on research results deal with EP or PU adhesives and a thin glueline to ensure a proper mechanical bond [2].

Fig. 1: Examples for unconventional glued applications with concrete-type adhesives: truss structure (left) and tree structure (right)

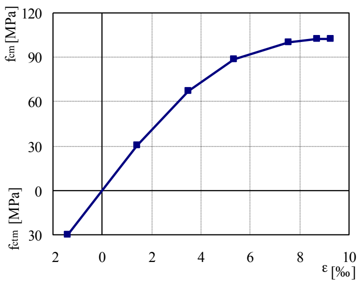
The new-type jointing technology use modified adhesives with dried fillers to create a concrete-type connection with high stiffness that guarantees stiff bonding to enclosed timber members. This is suitable for applications shown above as well as for structural upgrading and rehabilitation of traditional flooring systems.

3. Material characterization

The proposed CTA is a polymer-bound concrete formed of double component type liquid epoxy resin with a mineral aggregate. The resin for the production of the CTA was free of dissolvent, stable crystallizing low-molecular epoxy resin based on BPA. The hardener adapted for the product is a fluid, colorless to light yellow at room temperature polyamine adduct, which holds an average reactivity for the interlacing of the fluid resin. The mineral additive is composed of well-graded gravel with a grain size of max. 6 mm. All components were mixed together in a special ratio by weight. A comparison with the usually on building site used concrete is shown in the following table.

Tab. 1: Comparison of used CTA with r/c following DIN EN 206-1.

Material property	CTA		RC C25/30		Ratio
Density	2.0	g/cm^3	2.4	g/cm^3	0.83
Tensile MOE	19,600	MPa	30,000	MPa	0.64
Bending strength	30	MPa	5.5	MPa	5.45
Compressive strength	110	MPa	30	MPa	3.37



The high compressive strength of CTA results from the bonding behavior of the polymer binder material and especially the mineral fillers, which leads into a high packing density. With regard to the stress-strain behavior of the composite material, an ideal-elastic behavior with a semi-ductile hardening rule is observed. The failure behavior in tension or shear loading performs in a more brittle way compared to pressure loading.

Due to the larger amount of gravel in this composite material it is possible to assimilate a large amount of concrete, e.g. for much bigger drill holes compared to conventional drill hole diameters when dealing with glued-in rods, without getting in trouble with exothermic chemical reactions like in higher-content resin and curing agent adhesives compositions.

4. Experimental and numerical investigations

The modification of conventional glued-in rods was done first by a geometrical adjustment addressing much larger drill holes (Fig. 2) and second by adhesive modification with dried fillers to reduce the stringent quality control and assembling effort on site.

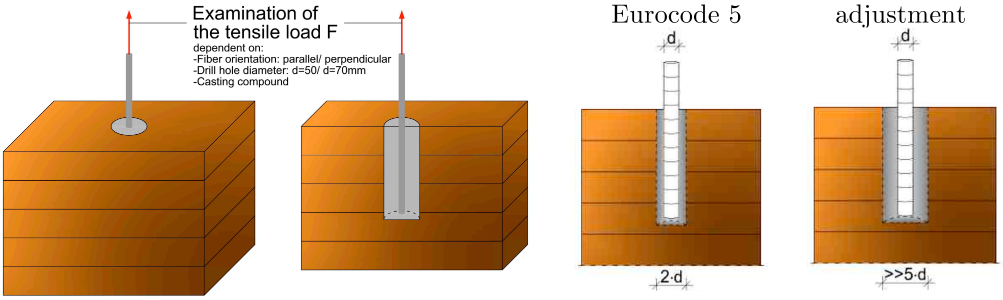


Fig. 2: Comparison of drill holes [3]

Based on the consideration of the specialties in near surface mounting on wood adhesives a full-size numerical FE-model was created and calibrated with push-out lab tests addressing the structural and material nonlinearities of wood and CTA as well as damage and debonding behavior in the joint (Fig. 3).

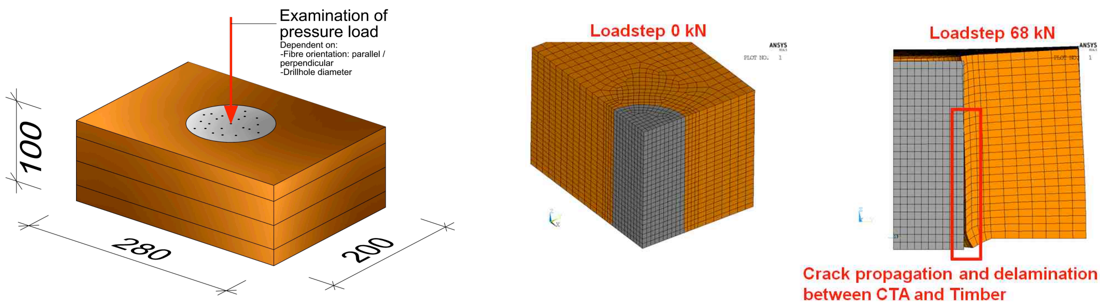


Fig. 3: Push-out test specimen for calibrating the full size numerical model

The proposed numerical model show a good agreement with results obtained from the lab tests (Fig. 4). Further investigations are ongoing to describe the proposed building system in a more analytical way and will be reported in the future.

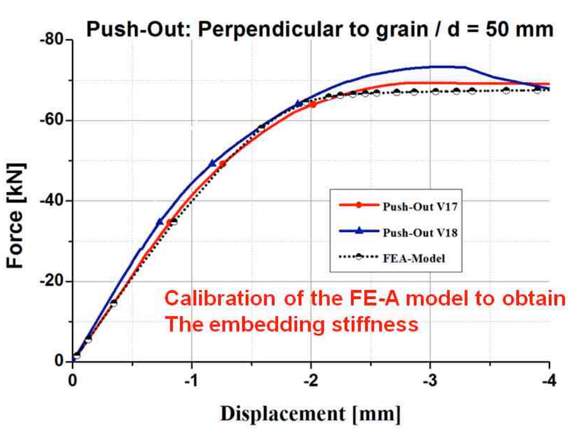


Fig. 4: Load-displacement behavior, comparison of lab test and numerical model

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Enhanced timber properties with FRP composites

Robert I. Kliger[1], Alann André[2]

Summary

This paper presents the results from several projects related to timber and glulam members enhanced by FRP composites. The studies were conducted at Chalmers University of Technology, Division of Structural Engineering. The aims of these studies were to investigate the opportunity to strengthen glulam beams with carbon-fibre-reinforced polymers (CFRP) to increase both the stiffness and strength of the beams both in the short term and in service (long-term performance). The first studies included experimental tests which resulted from short-term tests to failure of nine glulam beams strengthened with bonded steel plates or CFRP laminates and mechano-sorptive creep tests of strengthened glulam lamellas. The next study included improving the performance of beams loaded in bending and strengthening on both the tension and compression sides. The compression failure mechanism in the strengthened compression side of the timber beam was of special interest. The last study included a new and innovative technique to pre-stress beams using FRP laminates in order to utilise the FRP more effectively, to reduce the cross-section of glulam beams and improve the overall performance.

1. Introduction

The limiting factor when it comes to design timber structures is often the stiffness properties of timber products. The stiffness requirements in serviceability limit state, both short-term and final deformation especially in horizontal members, is a factor that often makes it necessary to increase the dimensions of the member. This cause increased material use and, as a result, higher production costs. Timber is a material that is relatively brittle in tension, but it may plasticize when loaded in compression. The tensile strength is often the same or smaller than the plasticizing point in compression, leading beams loaded in bending to fail in brittle way on the tension side. Reinforcement of timber structures have been made for several decades. Early attempts were made over six decades ago by for example [1] using steel rods placed in grooves in the top and bottom surface of timber beams. Several more studies have been made since then. One problem with using reinforcement is the incompatibility between the wood and the reinforcement, most notably the differences in hygro-expansion and creep that resulted in failure in the bond line between timber and steel leading to delamination. Modern adhesives has minimised this problem and reinforcing timber beams are now a real opportunity to improve the performance of timber structures. Reinforcement on the tension side of beams leads to higher short-term strength and a more ductile behaviour.

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[2] Swerea SICOMP AB, Mölndal, Sweden

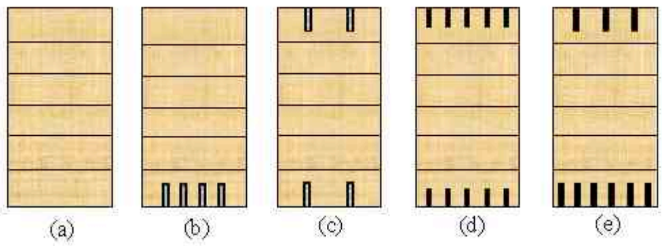
The increased stiffness will also reduce the short-term deflection but the reinforcement will even further reduce the long-term deflection due to the very low creep effect of the reinforcement material.

2. Strengthening glulam with CFRP or steel plates

2.1 Experimental short-term tests to failure

The experimental investigations comprise both short-term tests to failure of nine glulam beams strengthened with bonded steel plates or CFRP-laminates [2] and mechano-sorptive creep tests of strengthen glulam lamellas loaded in bending at approximately 8 MPa [3]. The effect of various geometrical and material properties of laminates was investigated, as well as strengthening with different amounts of laminates on the tension and compression side. A simple analytical model for wood with plasticity of the compression was developed.

The five configurations tested are shown in Fig 1. The beams were of the dimension 115 x 200 x 4000 mm. Two un-reinforced beams were tested to obtain a reference value for the strength and stiffness. The reinforcement was placed along the total length of the beams, i.e. continuing over the support. Two configurations with steel as strengthening material were tested, both with a reinforcement percentage of 2% of the cross-section area. Two beams were tested with all the steel reinforcement on the tension side and two beams with 50% of the reinforcement on the tension side and 50% on the compressive side of the beam. Two different types of CFRP were tested, one with high strength and one with high stiffness. Two beams were tested with 50% of the reinforcement in tension and 50% of the reinforcement in compression. The last beam was tested with 33% of the reinforcement on the compression side and 67% of the reinforcement on the tension side.



Beam Type	Reinforcements				Adhesive System
	Type	Size	E-Modulus	% in c/s	
a	Blank	-	-	-	
b	Steel	4x(4mmx30mm)	210GPa	2%	SikaDur®-30
c	Steel	4x(4mmx30mm)	210GPa	2%	SikaDur®-330
d	Sika® CarboDur® H	10x(1.4mmx25mm)	300GPa	1.5%	SikaDur®-330
e	Sika® CarboDur® S	9x(1.4mmx30mm)	165GPa	2.5%	SikaDur®-330

Fig 1: Reinforcement configurations [2]

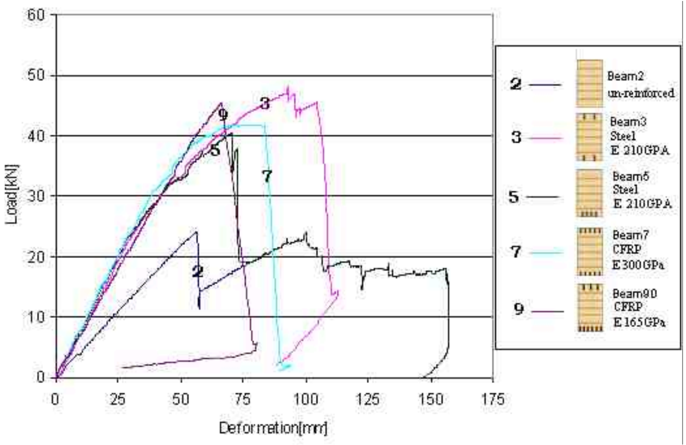


Fig 2b: Fiber buckling of both, timber and CFRP laminates, in the compressive zone.

Fig 2a: Load-deflection curves for one beam from each configuration [2].

For the global stiffness, the main observation is that the stiffness increase was according the amount as well as the stiffness of the reinforcement introduced. The highest stiffness was achieved for the beam reinforced with the stiffest material (CFRP-stiffness 300 GPa, beam 7), see Fig. 2. The two beams reinforced with CFRP (configuration d) show excellent similarity in their properties. The stiffness varies with less than 1% while the ultimate loading capacity varies with 8%. All the reinforcing schemes had a positive effect on the overall ductility of the beams.

2.2 Simple non-linear model

A non-linear model was used to study the behavior of the reinforced beams under loading. The aim with the model was to study the effect of different amounts of reinforcement as well as the distribution of the reinforcement between the tension and compression sides of the glulam beams with the cross-section of 115 x 200 mm^2. Another important aspect was to control the ductility of the beams. To study the influence of division of reinforcement between the tension and compressive side of the beam the ultimate moment capacities and elastic stiffness of different configurations with different reinforcement ratios in the tension and compression zones were plotted. The reinforcements were arranged vertically in the cross-section at the tension and/or compression side of the beam. Five different failure modes were necessary to take into account in the modeling. The first and second was tension failure of the glulam when the beam was in a linear-elastic state and while the timber has started to yield in the compression zone. The third mode was compression failure (the compressive strain is too large) before tension failure, this occurs when the beam is heavily reinforced on the tension side. The fourth mode was yielding of the steel reinforcement. The fifth mode was rupture of the reinforcement. There were some other possible failure modes that can happen but that were not included in the models; these were failure of the adhesive and compression failure of the reinforcement as a result of buckling of the reinforcement fibers in the compressive zone. Another possible failure mode was shear failure of the beam. The results contained curves for total reinforcement percentages ranging from 1% to 5%, which was per-cent of the cross-section area. The results for stiffness showed as expected that the higher the percentage of reinforcement the higher the stiffness of the beam. For stiffness, the highest increase can be gained when placing half of the reinforcement on the compressive side and half of it on the tension side. The results for ultimate bending strength showed the same results when increasing the amount of reinforcement, the more reinforcement the higher the ultimate bending strength, at least up to a level of 5% of reinforcement. However, the results showed that for the ultimate bending strength it is better to have only between 25-30% of the reinforcement on the compressive side of the beam.

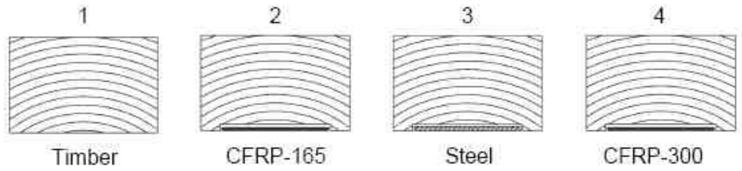
2.3 Compression failure of strengthen glulam beams

Compression failure in the wood and in the CFRP reinforcement were observed, initiating across the width at the top of the beam and propagating downward along the height of the beam. The understanding of this compression failure mechanism is a key parameter in the design of reinforced timber beams. The aim of this study was to investigate the opportunity of predicting analytically and numerically the load bearing capacity of glulam beams reinforced with CFRP based on an empirical tri-linear material model for the beam material in the compression side of the beam. Additional two beams, cf. Figs. 1 and 2, with one new configuration was tested in order to study the compression failure in more detail [5], [6]. Although the statistical significance could not be proven based on the low number of tests performed, it was experimentally shown that strengthening of glulam beams by means of FRP laminates is successful and can be optimized in order to trigger specific failure modes. The ductility is caused by two factors in these beams. 1) Reinforcing on the tension sides will lead to yielding of the glulam on the compressive side [7]. By controlling the reinforcement percentage on the tension side of the beam it is possible to induce a desired amount of yielding on the compressive side of the beam. 2) Buckling of the CFRP laminates on the compressive side of the beams. At high load levels, it is possible to induce fiber buckling in the CFRP laminates. In this study, it was not useful to apply higher reinforcement percentages than approximately two per cent in volume. Higher reinforcement percentage led to shear failure, which is a brittle failure mode of wood.

3. Long-term deformations

In this pilot study [3], three different strengthening schemes, see Fig. 3, as well as a reference beam were creep tested in bending. In each group 6 beams were used. 24 specimens were used with the timber dimensions 45 x 70 x 1100 mm^3. The 24 specimens were taken from six trees from two different stands. Three of the trees were from a fast growing stand and the other three are from a slow growing stand. The specimens were sawn from the second log in the tree, taken 3 meter from the butt. The log was sawn 3x log with the centre part being 70 x 300 x 3000 mm^3, this piece was further resawn to six 45 x 70 x 3000 mm^3 studs. In this study, the two centre studs were discarded due to their content of juvenile wood and all the material used came from the outer four studs. The creep test specimens were taken from a part of the studs with small knots. The origin of the timber is well documented in [4].

The creep tests were conducted in four point bending. The stress level was set to 8 MPa for the combined sections. The reinforcement was placed on the tension side of the beam. Directly after loading of the specimens the climate was set to 90% RH, thereafter the climate was changed between 30% RH and 90% RH in two-week cycles.



Beam type	Type	Size	E-modulus	Tensile strength
1	-	-	-	-
2	CFK 150/2000	70 mm²	165 GPa	2310 MPa
3	CarboDur H514	70 mm²	300 GPa	1350 MPa
4	Steel	100 mm²	210 GPa	235 MPa

Fig 3: Creep test specimens - different strengthening schemes and reference beam sealed with epoxy.

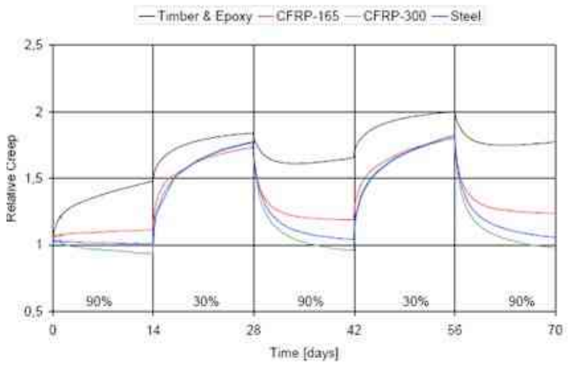


Fig 4a: Mean relative creep for the four different reinforcement schemes

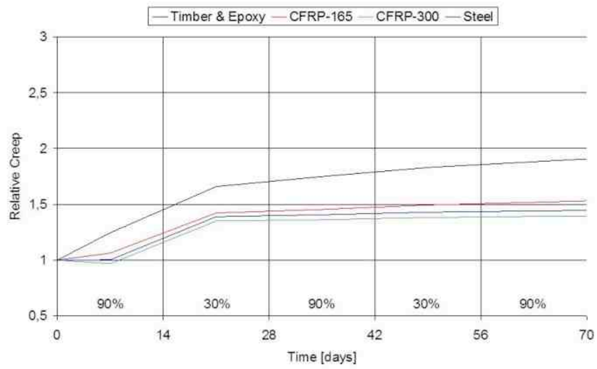


Fig 4b: Mean relative creep - trend lines for the four different reinforcement schemes

The results showed that the reinforced specimens had a lower initial deflection than the un-reinforced specimens. The creep results are presented as relative creep, i.e. the deflection divided with initial deflection (after 60 s). Mean values of the relative creep from all series are shown in Fig 4a. The variation in relative creep between the wet and dry periods was very high, higher for the reinforced specimens than for the unreinforced specimens. For the reinforced specimens the variation between 90% and 30% RH was of the same magnitude as the initial deflection. This was caused by the longitudinal shrinkage of the timber material while the reinforcement material does not shrink. This effect was larger the more one-sided reinforcement is used. This problem could be improved by distributing the reinforcement material on both the tension and compressive side of the beam.

The trend lines, Fig 4b, were calculated by taking the sum of the initial value and the final value for each 2 week period. The trend showed clearly that the reinforcement prevents the mechano-sorptive creep rather well. After the first 4 weeks, the increase of the relative creep is for the steel and CFRP-300 reinforced specimens only a fourth of the increase for timber with epoxy.

4. Strengthening glulam beams with pre-stressed FRP

Pre-stressing FRP laminates for strengthening and repair of structures presents several challenges, due primarily to difficulties, which can occur due to the bonding of pre-stressed FRPs to glulam beams. One of the greatest challenges encountered is development of high shear stresses at the ends of the pre-stressed FRP, which can easily exceed the strength of the adhesive. Shear strength both in conventional adhesives used in composite structures and in timber is well below

25 MPa, whereas the shear stress at the ends can typically reach values of around 40-50 MPa. These high shear stresses in the ends of the pre-stressed FRP may result in delamination or debonding of the FRP laminate from the structural member. Another challenge more specific to pre-stressing glulam beams is the low rolling shear capacity in timber. If the shear stress in bonding exceeds this limit, failure may occur due to very small pre stressing loads in a timber structure. A further challenge is the potential risk of delamination. The most appropriate solution to overcome these previously outlined challenges is through implementing step-wise pre-stressing of FRP laminate which is very innovative method used in this study, see Fig 5.

The purpose of step-wise pre-stressing is to delay the axial force development, thereby transferring the axial force over an extended amount of time instead of suddenly as would be expected without step-wise implementation. The slower axial force transference in return transforms the shear stress development curve from one with a sudden initial peak, to several smaller "steps," corresponding directly to the axial step size. Consequently, accumulation of shear stress is prevented, and by controlling the length and magnitude of the axial "steps," the shear stress development can be limited to a set value.

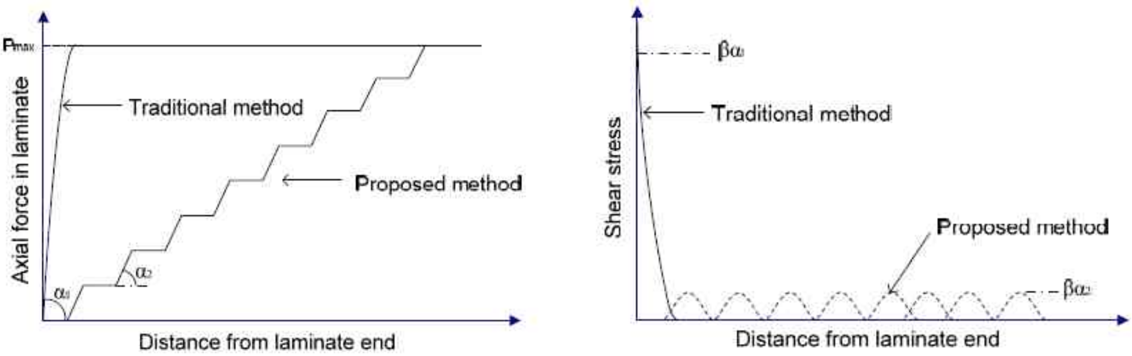


Fig 5: Axial force and shear stress distribution in traditional and the proposed method with a step-wise pre-stressing.

Using a step-wise pre-stressing method, the optimization procedure was developed. Many building projects require the lowest possible building height. The Moelven Töreboda multi-storey column-and-beam system building "Trä8" could be beneficial, with smaller glulam sizes with a span of 7.8 m. By analyzing an existing case, a more realistic vision of the potential advantages of introducing pre-stressed BFRPs can be obtained [8]. The size of the original glulam beam was 165 x 630 mm and the beam was exposed to a total load of 11.5 kN/m (SLS) and 20.5 kN/m (ULS) respectively. The main principle, which controls the design of the beam, is the deflection in SLS and ULS, as is typically the case in glulam design. So, by pre-stressing the glulam, the deflection can be reduced and a smaller section could resist the same load combination.

An example of the optimization procedure; the input data, which include geometry, loads, deflection criteria requirements, strength, and stiffness data for glulam and FRP and the obtained results are shown in Fig. 6 [8]. The results show savings in terms of the number of saved lamellas, utilization ratios for moment and shear, instantaneous and final deflection, maximum pre-stress force possible and type of analysis considered, i.e. elastic or plastic.

5. Conclusions

In order to improve the design tools for timber structures and particularly for timber beams reinforced with FRP, a full understanding of the materials used in different loading conditions is a prerequisite. The strengthening of glulam beams was successful in both short- and long-term tests. Main conclusions:

- All interventions resulted in higher stiffness and ultimate moment capacity.
- The amount of reinforcement is decisive for the failure modes. Increasing the reinforcement may not necessarily increase the capacity, as the shear failure mode can be decisive.
- The arrangement of reinforcement is used to control the strength, stiffness and failure modes.
- When it comes to the mechano-sorptive creep, it is beneficial to strengthen both the tension and compression sides of the beam to avoid large moisture-induced movements.
- It is very advantageous to strengthen beams with pre-stress FRP.

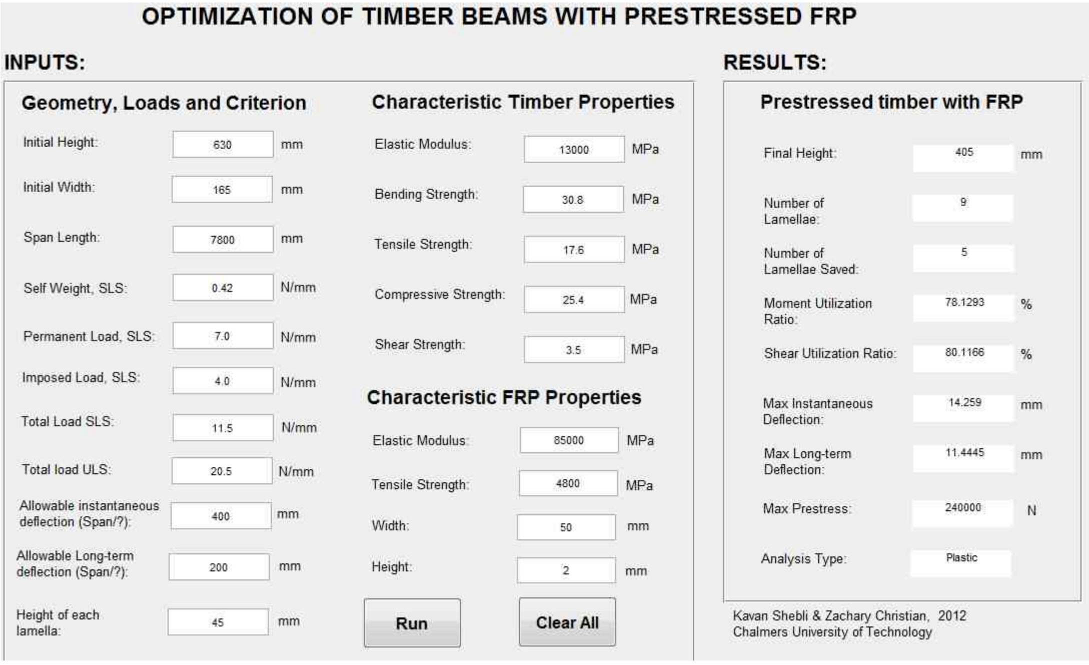


Fig 6: Results of optimized glulam beam [8].

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Investigation of the potential of hot-pressed wood in all-timber connections

Wen-Shao Chang[1], Nicholas Nearchou[2]

Summary

Glued-in rods (GIRs) are becoming an increasingly popular method in the construction industry for connecting timber, favored for their versatility. The most commonly used system is glued-in steel rods, which are typically prone to brittle failures due to material disharmony with the parent timber. The adhesives that are typically used have poor environmental credentials and reduce the recyclability of the parent timber at the end of life cycle of a building. This study aims to investigate a potential alternative, hot-pressed wood, to conventional bonded-in rod systems, which allows material harmonization and excludes the use of adhesives.

1. Introduction

Connections within the framing are usually the critical points of the structure. The choice of connection type and layout is crucial since it will directly affect the global performance of the structure. The construction industry currently uses conventional bonded-in rods, which often are responsible for unfavorable brittle failures and have poor environmental credentials. Some systems have tried to address this issue including: glass fiber reinforced polymer (GFRP) glued-in rods and glued-in hot-pressed timber dowels. These two systems allow better material harmonization so reduce brittle behavior. However, GFRP rods are not easy to cut or machine onsite, which makes them extremely awkward to recycle when using wood recycling techniques.

There is a gap in the industry for an eco-friendly, recyclable and non-brittle alternative to existing GIR systems leading to the development of an all-timber binder-less connection. This study proposes a new system, binderless hot-pressed dowel, to tackle the issued addressed above and explore the potential of this new system used in a timber-framed structure.

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2. Proposed connection

The proposed connection works by exploiting the swelling properties of hot-pressed timber dowels. It relies on friction along the dowel-hole interface caused by the expansion of the soaked dowel within the clearance hole. As the dowel expands it will exert a confining force onto the parent timber hole wall (N_{swell}), which will then mobilizes a frictional resistance force (F_{mob}) when the dowel is subject to pull-out (P), shown in Fig. 1.

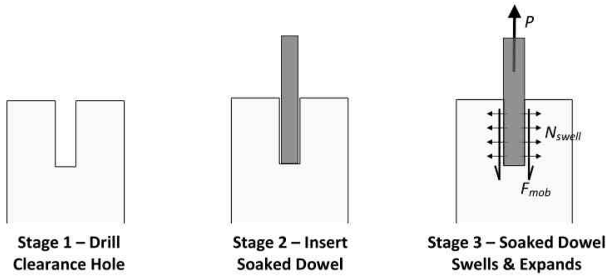

Fig. 1: Proposed connection

3. Analytical approach

The analytical approach considers: (1) Dowel as a beam in pure flexure; and (2) Friction between dowel and parent material, the moment capacity of beam can be expressed in the form of Eq. (1):

$$M_{Rd} = M_d D \left(\frac{1}{E_d I_d} + \mu \frac{3}{2l} \right) + \frac{1}{2} \mu f_{swell} \, l D \pi d \tag{1}$$

where M_{Rd} is the ultimate moment in the beam, μ is the kinetic coefficient of friction, M_d is the dowel bending capacity, D is the lever arm (see Fig. 2 and 3), f_{swell} is the swelling pressure of dowel and E_d is the modulus of elasticity of dowel.

4. Experiments

4.1 Dowel bending tests

Four point bending tests were conducted to establish the mechanical properties of the hot-pressed dowels. Red Western cedar (Thuja plicata) was used to fabricate specimens in bending and punching shear tests. 10 hot-pressed specimens were then cut into dimensions of 12x12x240mm for bending tests. All testing procedures and determination methods were conducted in compliance with BS EN 408:2003.

4.2 Punching Shear Tests

Punching shear tests were conducted in order to determine the performance of the bond between parent timber and expanded dowel. Two series of specimens were prepared for punching tests, series A and B. The specimens of the series A were left for 4 days to allow for spring back then shaved into 12 mm diameter circular dowels. The specimens in the series B were shaved into 12 mm diameter round dowels right after the completion of hot-press process. The completed samples were then sliced into punching shear specimens. Thicknesses tested include 10, 20, 30, 50, 80, 100 and 50 mm. (Fig. 2). A total of five tests were conducted for each group with different thickness, which makes up 70 specimens for punching tests.

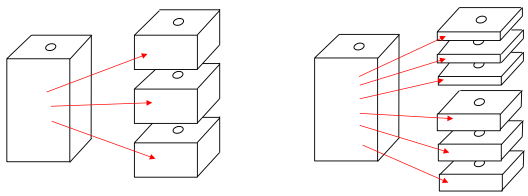

Fig. 2: Punching shear specimen slicing preparation for 10, 20 and 30mm thickness specimens

4.3 Beam tests

A hot pressed dowel was used to connect two pieces of timber with dimensions of 75 x 150 x 600 mm to build a beam with length of 1.2 m. A total of 24 specimens were fabricated for bending tests. Grade C16 European whitewood (*Picea abies*) was used as the parent beam material.

Tab.1: Selection of approaches for the design of notched beams

Specimens	type of dowel[1]	edge distance	No. of specimens
A-2d	1	2d	3
A-3d	1	3d	3
A-4d	1	4d	3
A-5d	1	5d	3
B-2d	2	2d	3
B-3d	2	3d	3
B-4d	2	4d	3
B-5d	2	5d	3

[1] "1" represents the same process of dowels in group A in punching shear tests; whereas "2" means the group B.

The embedment length was kept constant at 100 mm each end. Tab. 1 gives the name and details of each specimen. The factors investigated in this study include edge distance and types of dowels. The edge distances tested include: $2d$ (24 mm), $3d$ (36 mm), $4d$ (48 mm) and $5d$ (60 mm).

5. Results and Discussions

5.1 Dowel bending tests results

The results of bending tests have shown an average Modulus of Elasticity of 13.2 kN/mm^2 with a standard deviation of 0.64 kN/mm^2, which is comparable to timber with strength class of C35 ($E_{0,mean} = 13$ kN/mm^2). Based on BS EN 14358:2006, the 5% MOE ($E_{0.05}$) is 11.91 kN/mm^2, which is comparable to timber with strength class of C50 (11.05 kN/mm^2). This shows the MOE has increased by two times compared with strength class C16 after the hot-pressed process and demonstrates that hot-pressed process can be a useful method to enhance the mechanical properties of low-grade timber.

5.2 Dowel bending tests results

The results have shown that the hot-press process will significantly reduce the variation between individual timbers. The averaged Modulus of Rupture was 87.2 kN/mm^2 with standard deviation of 7.1 kN/mm^2. Fig. 3 plots the maximum shear stress against specimen thickness. It can be found that the specimens in series B show much higher peak values, and in consequence lead to higher averaged maximum shear stresses. It is because the series B hot-pressed dowels did not spring back before they were inserted into the holes in beam members.

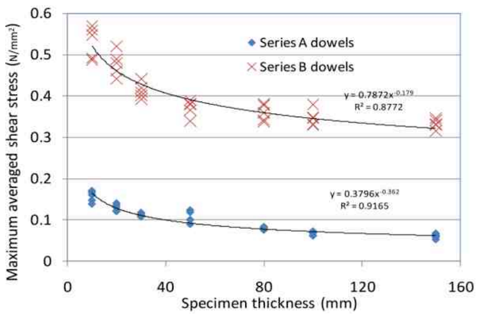

Fig. 3: Average Shear Stress at Peak Load vs. Specimen Thickness

5.3 Beam bending tests results

It was observed that beam connected with hot-pressed dowels normally fail in ductile mode. It was found that those specimens connected by dowels with density higher than 700 kg/m^3 exhibit failure in brittle manner. For structural application, lower density dowels (below 700 kg/m^3) will help to ensure a ductile failure.

The ductility appears to be provided by this frictional resistance; a large portion due to the swelling of the dowel and this is particularly obvious when the beam specimens are connected by series B dowels. This also indicates that more swelling provides higher post peak residual strength.

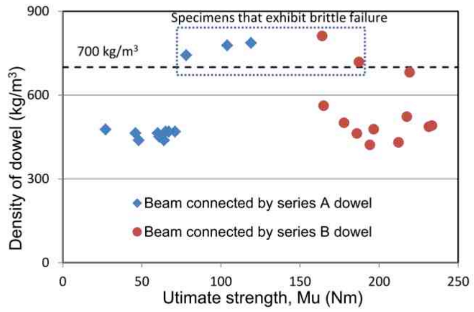

Fig. 4: Density of hot-pressed dowel vs. ultimate bending capacity of beam

5.4 Comparison with analytical model

The Eq. (1) shows the moment capacity of beam connected by hot-pressed dowel by using energy method. Fig. 5 shows good agreement in comparison between the estimated moment capacity of beam obtained from Eq. (1) and those results obtained from experiments.

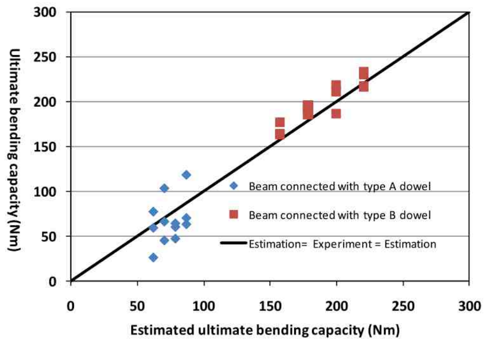

Fig. 5: Comparison of results from estimation and experiments

6. Conclusions

The proposed connection system was used in a beam splice connection; bending tests were carried out to assess its performance in a practical application. Fairly modest moment capacities in the region of 0.055-0.085 kNm were observed for specimens with type A dowel and 0.16-0.24 kNm were observed in the specimens with type B dowel. All the specimens showed very high rotations at failure around 0.30-0.35 rad (17-20°) which is favorable for structural applications to provide early pre-collapse warning.

Comparison of different techniques for the strengthening of glulam members

Robert Widmann[1]

Summary

There are several reasons for strengthening timber members made out of glulam. Besides ageing and delamination, issues that are linked to a certain stage of reduced strength and/or stiffness due to existing failures, there are also needs for strengthening intact glulam members, e.g. in the course of a change of use with planned higher structural loadings.

The work presented in this paper is part of an ongoing Swiss federal research project that deals with the assessment and strengthening of glulam members. The paper focuses on the strengthening part.

In this paper, the performance of two strengthening methods, self-tapping screws and CFRP sheets is being discussed on base of results of preliminary tests.

1. Materials and Methods

1.1 Glulam beams

The material consisted of timber beams made out of glulam from Swiss grown Norway spruce. The cross section was width $b = 140$ mm, depth $h = 600$ mm. Following a grading including the determination of density and dynamic MOE as well as knots, the lamellas were sorted in order to build up beams with homogenized material properties. The quality of the lamellas fulfilled the requirements for GL24h. Two different measures were taken to reduce the strength of the beams. For one series of 8 beams, the middle lamella was glued only on one third of its width, reducing the shear strength respectively. The length of the missglued beams was 2.50 m. In order to prevent early bending failures the outer lamellas of all beams where made of high strength larch timber. For the introduction of the high loads, the support and loading point areas of the beams without notches were reinforced with self-tapping screws with dimensions of 8.2 mm x 140 mm and 13 mm x 250 mm respectively.

1.2 Reinforcements

Two different reinforcement methods were used so far: self-tapping screws 13 mm x 800 mm and CFRP tissues. The screws were provided by Swiss company SFS unimarket AG and the CFRP tissues by Sika AG, also a company based in Switzerland. Both, the CFRP sheets and the screws were applied under a 45° angle

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to the grain (Fig. 1 and Fig. 2). The reinforcements were applied by technicians of the two companies and/or under their supervision in order to guarantee optimal quality. For the application, it was assumed the beams are installed in an existing structure and that their top edge is inaccessible. In consequence the reinforcement was applied from underneath.

The number of screws applied per side was varied from one to two to four. Two beams each were reinforced with these configurations and the remaining two were reinforced using the unidirectional CFRP sheets.

Fig. 1: Beam reinforced with SFS screws 13 mm x 800 mm (principal)

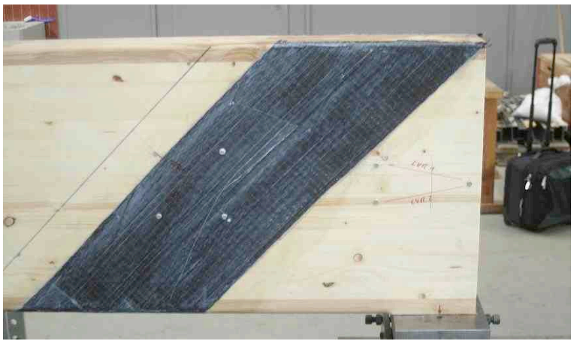

Fig. 2: Beam reinforced with Sika unidirectional CFRP sheets

1.3 Test Methods

All beams were tested under 3–point bending. Initially the beams were tested without reinforcement until one side failed due to a combination of tension perpendicular to grain and shear stresses (notched beams) and due to shear stresses alone (beams without notches). Following the failure, each beam was strengthened in the failed area and again subjected to three - point bending until failure occurred on the other side. After the reinforcement of this side, the beams underwent a third testing cycle in order to determine the strength of the reinforcement. For all tests the loads were recorded as well as the global bending deformation and deformations along the missglued lamellas or notches

2. Results and discussion

Apart from the failure load and the respective bending stress σ_b at midspan the nominal shear stress at failure τ^* and the nominal compression stress perpendicular to the grain at failure $\sigma^*_{c,90}$ were determined in order to highlight the level of the loading. Both nominal stresses are referred to the cross section or the support area without taking into account the shear- and compression reinforcements.

Missglueing the middle lamella had the desired effect in reducing the nominal shear strength τ^* of the beams. Every applied strengthening technique led to a significant increase of the shear strength. However, the ultimate shear strength capacity of the reinforcements could not be determined discretely as most of the beams showed a failure different from shear failure after having been reinforced on both sides. Apart from evaluating different strengthening techniques the future work will therefore also concentrate on finding test set ups that permit a higher shear loading.

However, these tests showed that the required shear strength could be obtained without problems whereas the shear stiffness keeps clearly inferior to initial stiffness.

3. Conclusions

In a test series, several glulam beams with a missglued middle lamella were loaded up to failure and then reinforced with the help of different techniques. In this paper, preliminary results of tests with self-tapping screws and CFRP reinforcements are presented. It could be shown that the reinforcements are effectively increasing the (nominal) shear strength. The high loading of the reinforced beams often lead to failures other than shear failures, so that the shear strength of the reinforcements cannot be stated discretely.

Introducing other material to increase seismic resistance

Roberto Tomasi[1]

Summary

This presentation illustrate some possible technologies adapted to enhance structural robustness and ductility of both new and existing timber structures in seismic zone, by means the use of other materials or components.

1. Increasing seismic performance of new timber structures

Although timber, thanks to the excellent ratio between strength and density, has been traditionally considered an interesting building material for earthquake resistant structures, compared to other building materials (steel or reinforced concrete), it shows a brittle behavior, with a poor attitude to dissipate energy.

Clear wood is an anisotropic material featuring different failure behavior both in compression and in tension according to stress direction. For small specimens loaded in tension parallel to the fiber the stress-strain relationship is linear up to the brittle failure occur. Structural size timber elements, because of the material characteristics, the inhomogeneity and the presence of defects (such as, for instance, knots), behave globally in brittle manner.

The difficulty to obtain energy dissipation in timber elements prompted the use of design approaches achieving some structural ductility via plastic deformations occurring in metallic joints manufactured with dowel-type mechanical connectors (e.g. dowels, nails, screws, and bolts).

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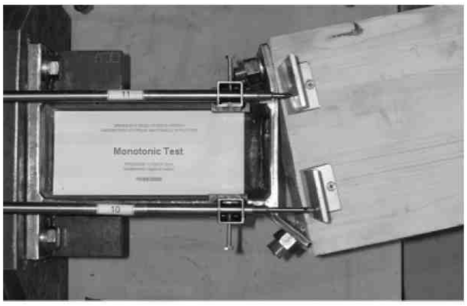

Fig. 1 Detail of a timber to steel semi-rigid connection with glued-in rods

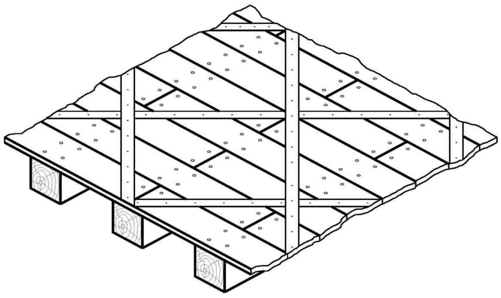

Fig. 2 Diagonal bracing of the existing wood planks by means of light steel plates or FRP laminae

To this aim, the high performance of the glued-in rod joint component can be exploited to assure a global ductile behavior of the steel-to-timber joint. Several efforts were made in the past in order to improve the knowledge of the mechanical performance of glued in rods, also by means national and international research project: notwithstanding a universal standard covering specifically the design is still missing (Stepinac et *al.* 2013). Andreolli et *al.* (2011) presented a "material-adapted" technology to manufacture timber joints with glued-in rods in heavy timber construction endowed with an inherent ductility. This connection system involve several interesting properties in term of mechanical performances and high levels of prefabrication, and its geometry can be properly designed to be used for moment-resistant joints at the corner and foundation of timber frames in seismic zones.

2. Increasing seismic performance of existing timber structures

Old timber floors often need strengthening and stiffening as they were designed to bear moderate loads and may suffer from excessive deflections with respect to current requirements. In case of lateral seismic forces, whereas the floor is not satisfactorily connected to the adjacent walls, or the in-plane stiffness is inadequate, different collapse modes involving overturning of the walls may be observed. Masonry walls can counteract, generally, an insufficient resistance to lateral loads acting out of plane.

In order to adequately model the global behavior of a masonry skeleton, it is fundamental to characterize the in-plane stiffness of horizontal diaphragms, which plays an undeniable key-role in distributing seismic lateral loads to the resisting walls. As a matter of fact it is expected that the more the in-plane stiffness grows, the more the collaboration between systems of piers increases.

The need to increase the in-plane stiffness has induced, in the past, some strengthening solutions which recent earthquakes have demonstrated to be inadequate or, in some cases, even unfavorable. As a matter of fact, the substitution of timber floors with concrete ones, the insertion of a concrete curb "inside" the thickness of the masonry walls, could imply, respectively, a significant self-weight increase and a weakening of the existing masonry walls.

Therefore, learning from the effect of past earthquake on reinforced existing building, some floor refurbishment techniques have been reconsidered: the new Italian standard code on existing buildings for instance suggests new alternative strengthening techniques for the horizontal diaphragms. Some timber diaphragms reinforced with different solutions, as the one presented in Fig. 2, have been investigated in an experimental campaign both in laboratory (Tomasi et *al.* 2009) and situ (Giongo et *al.* 2013).

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Use of CLT in Slovenia on seismically active areas

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Summary

This paper is a presentation of the existing CLT structures, either independent buildings or upgrades, in Slovenia. The chosen examples are all built on seismically active areas. A description of the buildings and technical background is given.

1. Introduction

Slovenia might not be as seismically active as for example Japan, however it has seen a number of quite intensive earthquakes (magnitudes up to 6.2 on the Richter scale) in the past decades. Most of the country is seismically active with ground acceleration of up to 0.25 g, with expected intensity of VIII on the EMS scale. Consequently all the buildings as well as building retrofits and upgrades must be done in accordance with the Eurocode 8 guidelines. That of course also includes CLT buildings. This can be somewhat problematic as the Eurocode standards (neither EC5 nor EC8) do not give any specific guidelines for cross laminated timber design. Nevertheless there is quite a lot of CLT structures in Slovenia, some of them being outstanding cases of timber and seismic engineering. In the following chapter we present a few examples.

2. New buildings

The Ekoprodukt storage hall is the largest timber structure in Slovenia. It was designed in 2010 and built half as high as planned. The building is conceived to allow an easy and quick upgrade when it grows too small for the owner's demands. In its final form it will have a complete volume of over 15,500 m^3. It already boasts with the largest cross laminated timber cantilever. It extends a corner of the building out by 8 m and was designed entirely with the state of the art finite element methods calibrated to experimental response. The seismic concept is an innovative approach of combining CLT with a glulam massive frame. The frame primarily supports the vertical loads. However the CLT panels are screwed onto the frame, consequently forming 'jumbo-size' timber frame with sheathing type of walls. Basically the proven concept of classic timber constructions was applied on a very large scale and by using CLT.

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Fig 1: Ekoprodukt storage hall. Bracing system (left), cantilever (under construction) on the right.

House Crnile is one of the latest CLT structures in the country. It is very large family house with over 280 m^2 of living space. It unique in many ways; the sheer size of the building; the seismic bracing is performed over the CLT roof panels; it has walls, running over the height of two floors, which hold the house and inner gallery in place. And it lies in one of the most seismic prone areas in the country.

3. Upgrades

The upgrade of the Terme hotel in Catez (Slovenia) is the largest upgrade of any structure in a seismically prone area in Europe. A four story reinforced masonry structure was upgraded by an additional 2 stories made in cross laminated timber increasing the hotel's capacity by 50%. By using an upgrade system with small self-weight and removing the unnecessary layers of the existing roof's concrete topping the complete mass of the structure was only raised by 10%. In addition adjusting the stiffness of the upgrade and adjusting its vibration periods to the main structure's the increase in seismic forces was calculated to be negligible. Hence a tremendous amount of resources intended for the seismic strengthening of the existing structure was saved.

Fig. 2: Hose Crnile (left), Terme Catez upgrade (right)

4. Research projects

Seismic strengthening of existing buildings with cross laminated timber panels is a new method that is currently in the process of obtaining two European patents. By using Xlam panels connected to the existing structure at story levels, older buildings, not designed to resist earthquakes, can be sufficiently strengthened against horizontal forces. Testing was performed both quasistatically and dynamically on the shaking table and for the cases of unreinforced masonry and reinforced concrete frames (with and without masonry infill) with very satisfying results. The system offers numerous advantages over the existing seismic strengthening methods, namely also the possibility of energy retrofit with the same panels.

Fig. 3: CLT strengthening system developed in Slovenia

The design of modern, timber antiseismic structures in Eastern Mediterranean Area

Panagiotis Touliatos[1]

Summary

It is a fact that timber structures behave well during dynamic loading. The remarkable strength of timber and its elasticity permit excellent performance to tensile, bending and shock forces. That is the reason of use of timber components reinforcing masonry throughout the history of construction since, at least, the Minoan era (4000 years ago) over the, seismically rich, Eastern, Mediterranean Basin. That is, also, the reason of using preferably timber structure for horizontal constructional parts like floor or roof during the same historic periods.

In modern times wood still is, perhaps, the best material to use for load bearing frames under strong seismic load. Especially in large-span frames. However, during recent experience period of large span timber framed constructions design, it was observed, that during strong seismic shocks, various damages occurred to them, especially around the load bearing frame components connections. Rigid portal frames suffered the most of those local failures.

Three hinged portal frames, presented better behavior but, still, during high seismic forces the rigid connection between column and beam showed some problems. Experimental construction in seismically endangered city of Trikala in Central Greece was designed and built. In that large span construction the steel column was connected to glulam beam by means of a sophisticated system of springs. During normal seismic loadings up to 6,5 Richter's the connection behaves rigidly. After that force, the eight springs permit some deformation absorbing the incoming load. When the seismic movement is reversed the eight springs, using the stored energy, bring back the components to their correct angle.

For more than 25 years, this structure performs successfully under several earthquake events. Second experimental stage was the construction of three hinged glulam arches that framed large span sport halls. Positioned in high seismic areas with, usually weak foundation soil, during several earthquakes (Korinthian Bay 1981, Athens 1999, Lefkas 2003 etc.) these structures behaved very well even when a secondary local fault passed through the building (Loutraki Sport Hall 1981)

Latest projects (2004 and 2012) were large span sport, timber framed structures in seismic areas following a bioclimatic and sustainable performance. The latest one operating since 2012 has a sliding roof opening over the main swimming pool area for natural ventilation and illumination.

All this experience proved and developed some important principles:

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- The initial idea of the overall structures composition must take into account its shape, the mass distribution, the load bearing components system and the materials. The static and dynamic calculation procedure cannot and should not be obliged to correct some initial, architects, wrong decisions.

- The design of the critical connections, especially among the load bearing components should be carefully designed properly for each characteristic case of the overall composition. Frequently, the typical, catalog's connection details should not be applied.

- The interdisciplinary cooperation of the relevant sciences experts: architects, civil engineers, mechanical engineers etc. should be performed from the initial stage of the composition studies.

Some characteristic construction examples will try to prove the above conclusions.

Innovative Timber Composites - Improving wood with other materials

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