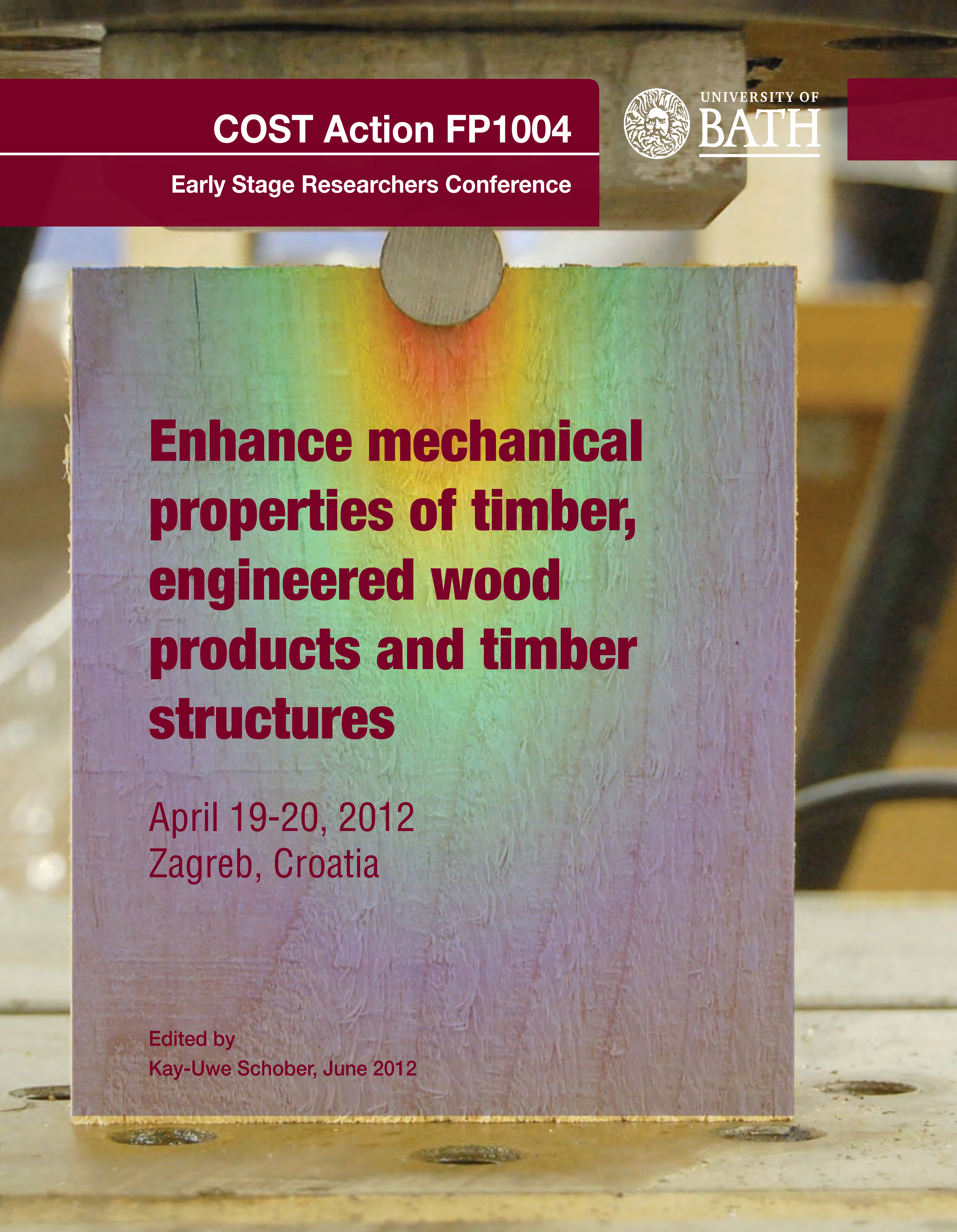

COST Action FP1004

Early Stage Researchers Conference

UNIVERSITY OF BATH

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

April 19-20, 2012
Zagreb, Croatia

Edited by
Kay-Uwe Schober, June 2012

COST Action FP1004

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Preface

COST Action FP1004, Enhance Mechanical Properties of Timber, Engineered Wood Products and Timber structures, has a broad mandate. Clearly timber suffers in comparison with homogeneous, isotropic materials such as steel. There are many ways of addressing the variability timber, which is a result of it being a natural, harvested product. There are techniques for moderating the effects of timber's different strength and stiffness at angles to the grain. The objective of the COST Action is to bring together an understanding of the state of the art in research in these areas and to identify particular gaps in knowledge.

At the first Management Committee Meeting of the COST Action, in November 2012, there were two important decisions. Firstly, the delegates agreed to focus, initially, on a selection of the key issues outlined in the Memorandum of Understanding, which underpin the scope of the project. Secondly, in response to COST Action Strategic intent, delegates agreed to promote a conference that would focus on Early Stage Researchers, giving them the maximum opportunity to become involved in the COST Action and helping them to promote their research and network with one another and to meet experts in their field of work.

The result of these two decisions was the Early Stage Researchers Conference in Zagreb, held on 19 and 20 April 2012. The template for the call for papers was an extended abstract and the papers selected for presentation are published in these proceedings. They form an excellent basis for understanding the State of the Art in these important areas of research. The COST Action will proceed to explore these and wider areas of research in the field of enhancement of performance of timber.

Attendees at the Conference, many of whom were presenting at an international conference for the first time, were able to enter into dialogue with their peers and with international experts. Scientific collaboration will develop from these exchanges and the many delegates will look back on Zagreb as an important milestone in their careers.

Richard Harris

Chairman of COST FP1004

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, modelling 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 programme 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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I would like to express my gratitude to all who voluntarily and with no or limited funding have contributed to this early stage R&D report and conference proceedings within their research area.

I am confident that all who attended would join with me in expressing our thanks to Professor Vlatka Rajcic and her colleagues at the Department of Civil Engineering at the University of Zagreb who so ably hosted the Conference.

A special thank you goes to Kay-Uwe Schober, member of the COST FP1004 core group, who has been vital to the success of this Conference. Not only did he handle management of submitted abstracts but he has also done the excellent work of editing these conference proceedings.

Richard Harris

Chairman of COST FP1004

June 2012

Table of Contents

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

Working Group 1

Enhance performance of connections and structural timber in weak zones

In this area, scientific activities focus on increasing and consolidating the current knowledge of structural behaviour of timber and connections in weak zones and how to improve/enhance performance and reliability.

This scientific area includes:

- Identifying and categorising weak zones (type of failure, relevance) and respective mechanical properties;
- Grouping of connections (load level, type of failure, dissipation of energy);
- Using glued-in rods or self-tapping screws as reinforcements;
- Using densified wood or modified wood;
- Using other Engineered Wood Products (EWP) e.g. plywood, LVL or cross-laminated timber (CLT) as reinforcement;
- Using fibre reinforced polymers (FRPs) as reinforcement
- Evaluation of design models and identification of respective gaps;
- Potential of non destructive test (NDT) methods in identifying weak zones;
- State-of-the art in reinforcing connections and weak zones;
- New jointing techniques (in cross-laminated elements, or in components created with CNC machines (direct timber contact).

Reconstruction and Health Monitoring of Wooden Pavilion Jeka (Echo) in the Forest-park Maksimir Zagreb

Marin Hasan[1], Radovan Despot[2], Jelena Trajkovi[3], Bogoslav Šefc[4], Vladimir Petri[5], Silvije Novak[6]

Summary

The wooden pavilion "Jeka" (Echo) was built in 1843 according to the plans of the Austrian Empire garden architect Franz Schücht. "Echo" pavilion is the only remaining one of ten similar pavilions that were placed in the Forest-park Maksimir during the 19th century. Since 1964 the "Echo" pavilion has been protected by law and declared as cultural monument of garden architecture together with the whole park Maksimir, in which it has been situated. Although the pavilion was restored several times (last time in 1986) it was in very bad condition. The reconstruction and preservation of the complete pavilion's wooden construction were done after a careful analysis of its recent state in 2000. During reconstruction, monitoring system for wood moisture content at three different positions inside the roof construction was installed. Twice a year the pavilion is being carefully visually inspected and data from data logger downloaded and correctness of the equipment verified.

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1. Introduction

As the wood was one of the most used construction materials in the ancient times, many parts of those objects, particularly roof constructions, were made of solid wood [1, 4, 5]. In the last fifteen years, a large number of houses and objects of Croatian's national cultural heritage were restored with fine workmanship.

Fig. 1 Pavilion in the drawing of I. Zasche, 1848

The wooden pavilion "Echo" was built in 1843 according to the plans of the Austrian Empire garden architect Franz Schücht (Fig. 1). Unfortunately the wooden pavilion "Echo" is the last and only remaining one of ten similar pavilions that were placed in the Forest-park Maksimir during the 19th century.

Since the year 1964 the "Echo" pavilion has been protected by law and declared as cultural monument of garden architecture together with the whole park Maksimir, in which it has been situated. When the moisture content was high enough due to construction faults in the roof and in the splash water area close to the ground, decays progressed (Fig. 2 and 3). It was necessary to restore the pavilion several times. From the year 2000 to 2002 necessary substantial restoration was done accompanied by the Forestry Faculty of the University of Zagreb as scientific consultant [4, 5].

2. Evaluation of the state of the art

By inspection of the pavilion «Echo», that was done in May 2000 (Fig. 2) it was found that the wooden parts and joints were mainly made of fir- and spruce-wood. The wooden parts of the construction were severely decayed by soft- and brown-rot (mainly *Gloeophyllum abietinum* and *G. separium*). The construction was not safe (the decayed poles did not hold roof properly, and few wooden panels were displaced). The ceiling did not have the echo effect.

Fig. 2 State of the Pavilion «Echo» before reconstruction

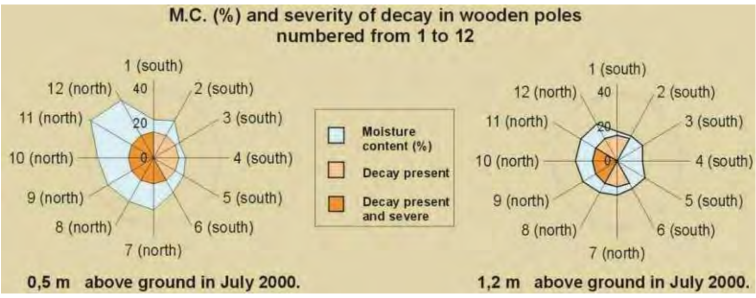


Fig. 3 Moisture content and decay severity in wooden poles

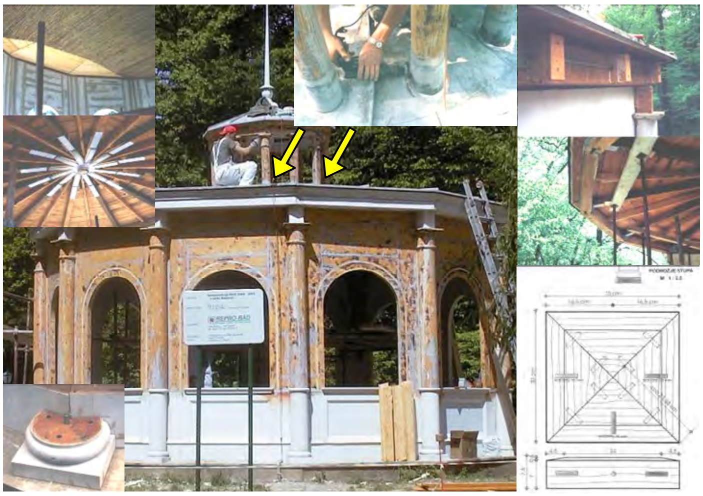
The wooden pavilion "Echo" was built in 1843 according to the plans of the Austrian Empire garden architect Franz Schücht (Fig. 1). Unfortunately the wooden pavilion "Echo" is the last and only remaining one of ten similar pavilions that were placed in the Forest-park Maksimir during the 19^{th} century. After detailed examination, two levels of degradation were established.

On the northern side, due to the lower insulation and water penetrating in, wood was more exposed and susceptible to the biodegradation, particularly in roof construction (lantern) which was chamfered and decayed. Wooden elements were completely decayed by the brown rot fungi, particularly lower parts of poles. Among other decay, the fructification of fungi *G. abietinum*, the most frequent brown rot decay of construction coniferous woods [2, 3], was found. It is also very important to mention that Forest-park Maksimir is known as the region with the highest relative humidity during the whole year in Zagreb (Fig. 3).

The southern, more "open" side of pavilion was damaged mostly because it was exposed directly to the sun light, particularly due to the significantly stronger abiological degradation. Frequent period of faster drying and also faster wetting has caused non-equal swelling and shrinking. Due to the non-equal internal strains in wood, the cracks and splitting appeared on its surface. As a consequence coatings had started scaling and cracking.

3. Reconstruction recommendations

All decayed wooden parts were checked and repaired. Instead of the then used fir and spruce decayed poles, larch and spruce heartwood was used for producing new poles. New poles or new parts of the poles were made of glued spruce core surrounded by larch heartwood. Poles were glued with resorcin-formaldehyde glue, and then turned. The bases and Toscan's bases were made of oak heartwood and protected with water repellent paste (Woodfill and Woodflex). Boron Woodpils were put in the bottom of each panel and pole (Fig. 4). All wooden parts were preserved by three minute immersions, or by spraying and brushing with Tebuconazole solved in white spirit. The wooden pavilion “Echo” was built in 1843 according to the plans of theAustrian Empire garden architect Franz Schücht (Fig. 1). Unfortunately the wooden pavilion “Echo” is the last and only remaining one of ten similar pavilions that were placed in the Forest-park Maksimir during the 19th century. After detailed examination, two levels of degradation were established. After preservation and assembling, the poles and panels were coated with the three – coat alkyd based primer – undercoat – mat system for boats.



The solution for the roof construction was a silicone based permanently elastic sealant to prevent water running into the roof where the little top pillars (yellow arrows in Fig. 4) on top of the roof are connected to the roof sheeting.

Fig. 4 Phase of reconstruction of Echo pavilion (2000)

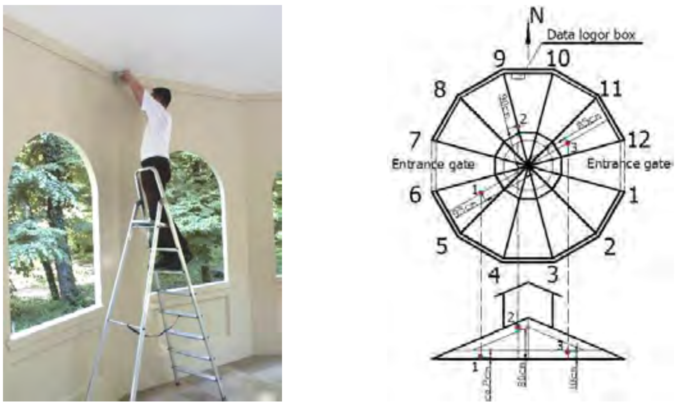


Fig. 5 Installation of moisture monitoring device:
Left: Position of the three measurement points in the roof;
Right: Installation of data logger box

However since “permanently” elastic sealants are in reality never permanently elastic, but only for a limited duration, wood moisture monitoring system was installed at different positions inside the roof (Fig. 5). Stainless steel screws drilled into the spruce roof beams in a distance of 30 mm to each other served as electrodes for the MC measurement based on electrical resistance. Cables were attached to the electrodes and run to a little box in which the data logger (Materialfox mini, Scanntronik Mugrauer, Zorneding) was stored [1].

4. Ongoing and future work

Since the moisture monitoring device has been installed, data logger is delivering daily records of the MC of the roof beams in 3 different positions. For better readability the monthly average values are shown in Figure 6 [1].

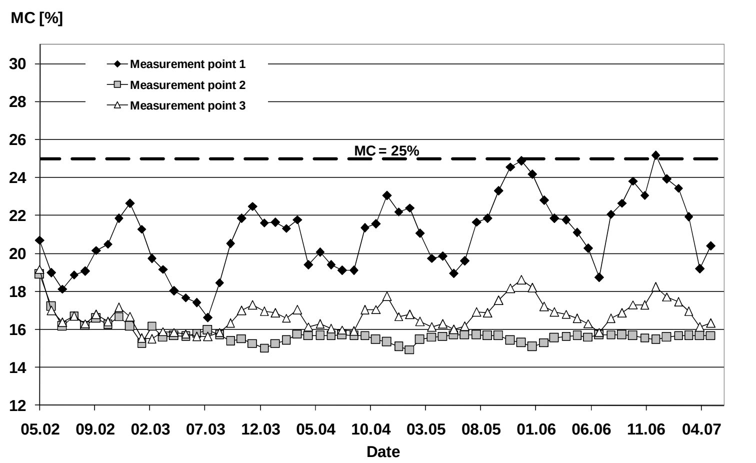


Fig. 6 Wood moisture content MC of the roof construction of the Echo pavilion in Park Maksimir, Zagreb, during five years of monitoring.

Measurement point 1 in the lowest position is indicating highly increased MC, especially during the winter time. November is the month with the highest precipitation in Zagreb. MC readings correspond with the precipitation peaks.

Measurement point 1 is located the lowest within the roof construction, where leaking water is accumulating when leaking through the roof and running down the beam. At the end of the recording period (5 years) the critical MC of 25 % was exceeded. This is a clear signal to open the roof construction now and do a revision including resealing of the connection between the little pillars on top of the roof and the roof sheeting.

5. Conclusion and outlook

Restoration of the “Echo” Pavilion was started because its construction was in poor condition and because the whole pavilion was unsafe. There was a severe decay of the lower parts of the poles, particularly those exposed to the North. Hence it was decided to replace all poles and several panels from the North side, and the lower parts of all South exposed poles. New poles or only the new parts of the poles were made of glued spruce core surrounded by larch heartwood. Organic solvent Tebuconazole and WOODCAP products were used in wood preservation. The restoration of the Pavilion can be seen as an attempt to extend the life time of a severely damaged cultural monument, or at least in those parts of the monument where the amendments were possible.

This example of the “Echo” Pavilion is showing that moisture monitoring systems can considerably contribute to prevent cultural heritage from decay. The installed inexpensive and simple monitoring system of early warning proved to be feasible and saved costs in the conservation of wooden cultural heritage. In times of tight budgets and increasing rates of decay moisture monitoring systems will become increasingly important.

6. Acknowledgements

We are very grateful for support from the public institution »Maksimir«, Zagreb, Croatia (http://www.park-maksimir.hr) and the Croatian State Hydro-meteorological Institute, Zagreb, Croatia, (www.meteo.hr).

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Robustness of structural timber truss systems

Vlatka Rajčić[1], Dean Čizmar[2]

Summary

In the last few years there has been intensely research concerning robustness of structures. As timber is very competitive building material questions about robustness of timber structures arise. However, reliability models applied to timber were always related to individual components but not the systems. In this project a robustness index and different measures of structural robustness, based on structural reliability, will be defined and analysed. Then, for the different timber truss structural systems a robustness will be assessed. Special attention will be drawn upon brittle/ductile modelling of timber and connections. Possible influence of both ductility of timber and connections on the robustness is to be analysed.

1. Introduction

Progressive collapse is characterized by disproportion between the magnitude of a triggering event and resulting in collapse of large part or the entire structure. Robustness of structures has been recognized as a desirable property because of a several large structural system failures, such as the Ronan Point Apartment Building in 1968, where the consequences were deemed unacceptable relative to the initiating damage. After the collapse of the World Trade Center, robustness has obtained a renewed interest, primarily because of the serious consequences related to failure of advanced types of structures. In order to minimize the likelihood of such disproportional structural failures many modern building codes require robustness of the structures and provide strategies and methods to obtain robustness. Robustness requirements are provided in two European documents: Eurocode EN 1990: Basis of Structural Design [1] and EN 1991-1-7 Eurocode 1: Part 1-7 Accidental Actions [2]. The first document provides the basic principles,

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e.g. it is stated that a structure shall be "designed in such a way that it will not be damaged by events like fire, explosions, impact or consequences of human errors, to an extent disproportionate to the original cause". The EN 1991-1-7 document provides strategies and methods to obtain robustness, actions that should be considered and different design situations: 1) designing against identified accidental actions, and 2) designing unidentified actions (where designing against disproportionate collapse, or for robustness, is important). In the JCSS Probabilistic Model Code [3] a robustness requirement is formulated as: "A structure shall not be damaged by events like fire, explosions or consequences of human errors, deterioration effects, etc. to an extend disproportionate to the severeness of the triggering event". In order to attain adequate safety in relation with accidental loads, two basic strategies are proposed: non-structural measures (prevention, protection and mitigation) and structural measures (making the structure strong enough to withstand the loads limiting the amount of structural damage or limiting the amount of structural damage). According to Danish design rules robustness shall be documented for all structures where consequences of failure are serious. A structure is defined as robust when those parts of the structure essential for the safety have little sensitivity with respect to unintentional loads and defects, or that an extensive failure of the structure will not occur if a limited part of the structure fails.

2. Robustness of timber structures – state of the art

In the last few decades there has been intensely research concerning reliability of timber structures but robustness of timber structures has not been shown much attention. One of the reasons for lacking interest / information about robustness of timber structures is that a unified approach for assessing robustness of any material is not available yet. Since timber is a complex building material, assessment of robustness is difficult to conduct. In the frame of the COST E55 Action [4] have made a deterministic robustness analysis of the collapses of both the Siemens Arena and the Bad Reichenhall Ice Arena. The Siemens Arena was build in 2001. as a large span timber truss system. Two of the trusses collapsed without warning at a time with almost no wind and only a few millimetres of snow. The partial collapse happened just a few months after the inauguration of the arena. An investigation showed that the cause of the failure could be localised to one critical cross-section in the tension arch near the support, where the load-bearing capacity was found to be between 25% and 30% of the required capacity. It is noted that the collapse did not occur due to an unknown phenomenon. The design of the trusses was not checked by the engineer responsible for the entire structure due to unclear specification of the responsibility and duties of that engineer. The Bad Reichenhall Ice-Arena built in 1971/1972. is a large span roof structure was supported by 2.87 m high main girders produced as timber box-girders. The box-girders featured upper and lower glued laminated timber members and lateral web boards. On January 2nd 2006, the entire roof collapsed without warning during a period of significant snowfall [5] . The review of the structural calculation revealed severe human errors in design and heavy misuse of building codes. These errors, humidity exposure and general lack of maintenance lead to the collapse of a structure. Based on the robustness framework described above [6] presented a reliability-based robustness analysis of a glued laminated frame structure supporting the roof over the main court in a Norwegian sports centre. Progressive collapse analyses are carried out by removing potential critical elements, and then assessing the reliabilities of the remaining structural elements.

3. Robustness of structural timber truss systems

The robustness analysis in this project is based on the general framework mentioned above [6, 7] and a probabilistic modelling of the timber material proposed in the Probabilistic Model Code [3] of the Joint Committee on Structural Safety (JCSS). The main difference with respect to the work by [6] is that in this paper the robustness of the different structural systems is assessed at two different levels. First the robustness assessment is made on componential level where reliabilities of the remaining components (after failure of one critical element) are compared with the reliability of the intact elements, and next on a system level, where a robustness index is formulated using system reliability measures. General information about this procedure can be found in [6, 7, 8]

Also, beside the robustness index formulated in [8], different robustness measures will be defined and analyzed (robustness factor, normalized robustness index and normalized robustness factor). In total, there will be six different structural systems (one is presented in figure 1). Figure 2. presents overview of experiments that will be conducted. First part relates to investigation of two spam beams and their material characteristics in order to asses possible material ductility in bending. Next step is to asses characteristics of the metal fasteners (punched plates and screws). Final step of the project will be to investigate full scale timber truss systems.

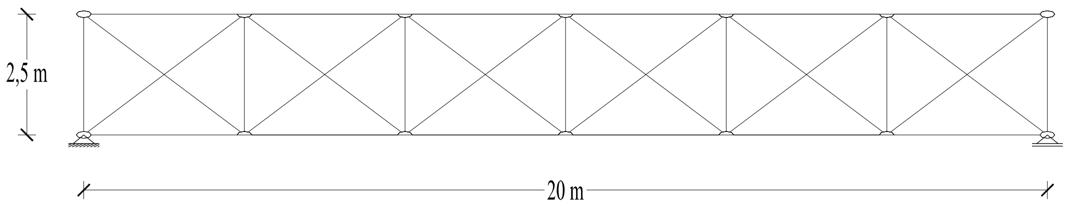

Fig. 1 Example of investigated systems (truss no. 2.).

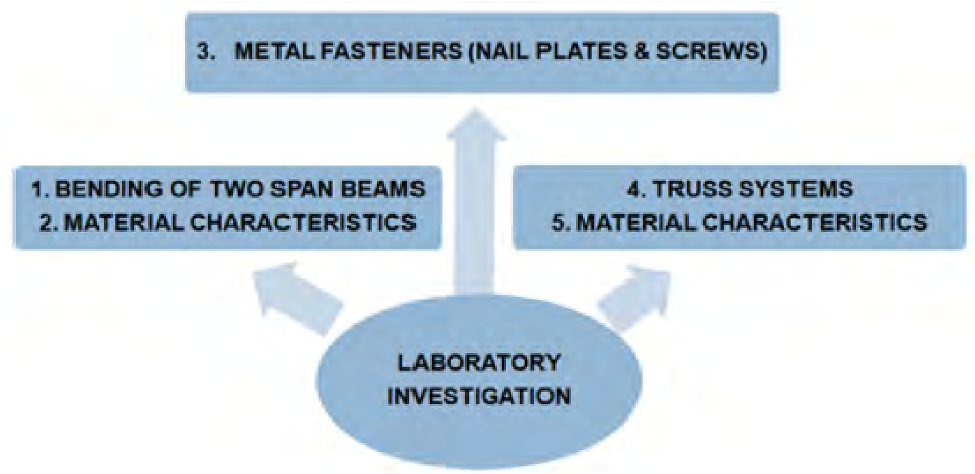

Fig. 2 Overview of experimental investigations.

Fig. 3 Truss system with punched metal plates.

4. Conclusion

A primarily aim of this project is to asses and quantify robustness of different structural timber truss systems. Method for robustness assessment is based on a general framework given by [7] and more in detail explained in [8].

Except from theoretical models, huge experimental program is planned. Based on data derived from experiments (which will be used for probabilistic analysis and for structural models), the current probabilistic model will be updated and the influence of both material ductility and ductility of fasteners could be defined and investigated.

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Composite structural systems timber-structural glass and steel-structural glass

Vlatka Rajčić[1], Mislav Stepinac[2]

Summary

The concept of research is a new composite system of glass and timber (or glass and steel), with both components acting as the supporting system. The results of the project will present the advantages and the problems related to such structures. Aim is to create new generation of structural composite systems of glass and timber with high aesthetic, economic and load-carrying values.

1. Introduction

Today combination of wood and glass is frequently used for facades, winter gardens and similar structures. They are not treated as structural composite system because of the unacquaintance of their composite behaviour. So, evaluation of load-carrying capacity and serviceability of composite systems are the main goal of the research.

2. Objective and Basic Problems

Aim of the research is to achieve a composite timber – glass panels that will have wide application in construction. To achieve that goal we need to find the best combination of these two materials. The most critical part will be joints, so design of joints is primary objective. Several different ways of connections between timber and glass will be examined and the best one will be chosen for further experimental examination. Connection between timber and glass can be made by gluing, mechanical connectors or by fitting (inserting) glass in sliced timber element. All methods will be examined in the laboratory.

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3. Past and On-going Research

3.1 Modelling and testing of joints of composite structural systems timber-glass obtained by gluing – shear test

Preliminary FEM analysis has proved to be decisive in the planning of laboratory tests. The analysis was performed with COSMOS / M software package, 2-D and 3-D model were made. In 2-D model SHELL4L plane elements have been used. Laboratory tests were made on composite beams to establish behavior of joints.

Fig. 1 Shear test on composite system timber-glass obtained by gluing and type of failure

Beams were exposed to shear. There were five glued samples and five samples made with steel bolts. The first group of samples was made in a way that tempered glass was glued to the wooden beams with Sikadur adhesive, while the 2nd group of samples was made in the way that timber beam and tempered glass were connected with L-profiles and steel bolts. Cross section of timber beam was 12/14 cm and length was 80 cm. Tempered glass was 10 mm thick. Testing was made in laboratory of Faculty of Civil Engineering in Zagreb. Shear test was made on hydraulic press with 0,4 mm/min input load speed. Tempered glass collapsed at the value of force of 186 kN (connection between glass and timber obtained by gluing). 2nd group of samples were also tested and didn't show very good because displacements of joints were too big. Laboratory examination showed that bonding is very good way to connect glass and timber and global failure is expected to happen at very high loads and not because of the failure of joints, but because of the mechanical properties of glass. Also, mechanical combination of glass and timber with steel L-profiles and bolts is not satisfactory because even at moderate loads huge displacements occur.

3.2 Laboratory testing of glass samples and composite panels – compression and bending test

After completion of tests in Zagreb and obtained results, next step of the research started. In the laboratory of Faculty of Civil and Geodetic Engineering in Ljubljana and in cooperation with University of Ljubljana, samples were made and prepared for compression test.

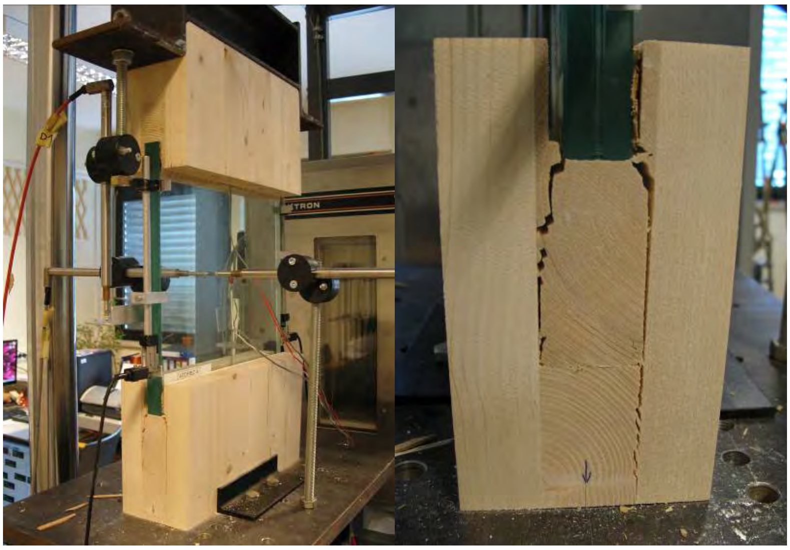

Fig. 2 Compression test of composite panels and model of failure

Profiles were made in a way that the flanges are from cross-laminated timber and web is double tempered glass thickness of 2 × 10 mm (figure 2). Tests proved that failure always occurs in timber elements as shown in the pictures.

At load of 50 kN, compression perpendicular to grain causes huge damages in timber flanges. For mentioned panel there were several questions to be answered; how to simulate realistic boundary conditions, how to simulate realistic response mechanism, what is the influence of vertical load and horizontal loading protocol. Conclusion is that vertical load-bearing cappacity of laminated glass sheet enable development of structural panels that are able to carry up several floors above.

3.3 Laboratory testing of composite frames

The central part of the research is to evaluate a large composite frame. Frame consists of 2-ply glass which is tightly engraved in timber elements that surround it. Panels represent cross-laminated timber consists of strips of spruce stacked on top of each other and glued together forming large-sized solid cross-laminated boards. Strips thickness is between 19 and 40 mm and maximum thickness of the boards is 60 cm consisting of 3, 5 or 7 layers. The moisture of these technically dried strips glued together is 12% (±2) which disables any pests, fungi or insects attacks.

Fig. 3 Frame prepared for testing

Fig. 4 Opening of studs

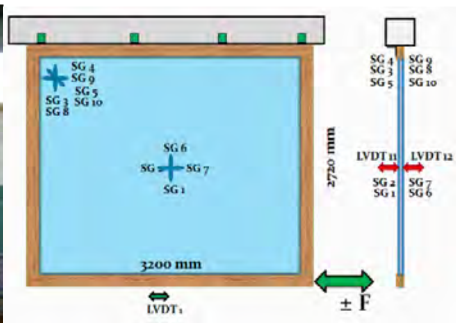

Fig. 5 Frame dimensions

Cross lamination of the timber strips has many advantages. It minimizes swelling and shrinkage in the board plane, considerably increases static strength and shape retention properties and enables load transfer across the entire plane of panel.

Mutual connection of timber elements is made in several ways; one screw in the corner, two screws in the corner or with punched nailed steel plates on one or two sides (with screws). The research aims is to get the cyclic response of frame and buckling deformations of glass.

Fig. 6 Failure when one screw was used

Fig. 7 Joints with screw and punched nailed plate

Three main simulations were made: shear cantilever (one edge of panel is supported by the firm base while the other can freely translate and rotate), constrained rocking (one edge of panel is supported by the firm base, the other can translate and rotate as much as allowed by the ballast; balast can translate only vertically without rotation) and shear wall (one edge of panel is supported by the firm base while the other can translate only in parallel with the lower edge and rotation is fully constrained). The critical part of the frame has proved to be the angle (connection of timber elements). Failure has occurred in the form of opening of studs. Also, tests where timber components are connected with punched nailed steel plates and screws were conducted.

5. Future work

Significant problem that should not be neglected is to examine performance of composite frames in earthquake conditions. After laboratory work in Ljubljana frame with best characteristics will be chosen and tested on shaking table to show behavior in earthquake. These tests will be made with partners Institute for Earthquake Engineering from FYRO Macedonia. Controlled laboratory studies will be conducted to investigate cracking and fallout resistance of composite frames. Quantitative data from these studies will be summarized, along with qualitative observations regarding the various failure modes exhibited by composite timber-glass frames under simulated seismic loadings.

6. Conclusions

This project predicts significant improvements in insight of the structural composite timber- constructive glass systems behavior. The investigation field represents the improvement of structure joint connections as well as development of their theoretical behavior models and failure mechanisms. The joints are an extraordinary important factor of the structure's bearing capacity and serviceability. The research aims to get to new concepts of bearing capacity and applicability of described joints with emphasison understanding of their behaviour. The procedure would involve the comparison of laboratory materials andglue testing as a constitutive joint element with computer model results.

Many problems of the timber-glass joints especially with respect to long term behavior and related realistic modelling/design are today unresolved. For the composite system load-carrying capacity, serviceability and factor of the stability will be determined. More experimental and analytical research is needed before the practical use of structural glass walls and composite timber-glass panels can be introduced in construction practice.

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Composite joint of timber truss girders

Vlatka Rajčić[1], Krunoslav Pavković[2]

Summary

In this paper the possibility of joining elements of timber truss girder with "built-in" connection will be explored. The joint will be based on glued-in steel rod, glass and/or carbon fiber-reinforced timber element and built-in steel tube which takes the tensile force. Fibre-reinforce textile will be implemented locally between the timber layers, in adhesion layer. Glue laminated timber elements will be laminated horizontally as opposed to the usual used vertical lamination.

The study of this joint will be conducted experimentally and numerically. Experimental testing will be conducted on a somewhat smaller number of specimens with applied load at angle 0°, 45° and 90° to the grain and on the prototype of truss girder. Fields not explored by experimental laboratory testing , will be explored by examining the complex numerical models.

1. Introduction

The study is planned to extend the investigation of timber structures built-in joints with large diameter fastener. The concept of the joint will be based on the glued-in steel bars and laminated timber with glass and/or carbon fiber-reinforce. Large number of studies were made on glued-in steel rods, and because of that this study will carry out only preliminary investigation of glued-in steel rods. Tests on the glued-in threaded steel rods will be carried out to establish the correlation between the resistance determined according to EC5 norms and the actual resistance obtained by experimental tests.

Reinforced glued laminated timber with glass fiber fabric is one of the main goals of this research. Specifically, research that was carried out for connection in

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reinforced glued laminated timber have shown significantly increasing of ultimate strength comparing to connection in unreinforced glued laminated timber. Experimental tests have shown that raising of ultimate strength is in correlation with reinforce factor and angle between applied load and grain [1-3].

Reinforcing of the timber elements is planned to be executed during the lamination process in a way that the fabric is placed between the layers, in the adhesive layer. Reinforcing of the timber elements in place of connection have been already explored, but the fabric in these studies was specifically knitted and the elements were studied with applied tensile force parallel to the grain. Also, previous studies were conducted with small diameter fasteners (screws, nails and studs), while in this research the plan is to use steel pipe with 50 mm in diameter or more. This research should provide new insights into the behaviour of this connection. Only a few researches were carried out for a connections with a large diameter fasteners [1, 4-6].

2. Planed joint research

2.1 Experimental research

Experimental studies are planned to carry out specimens in three series, which would be divided by applied force compared to the grain. Also, within these series are planned sub-series of the reinforced and unreinforced samples; in other words one serie would consist of three reinforced and three unreinforced sample.

The angles of applied force compared to the grain which will be tested on the samples, will be 0°, 45° and 90°. Fig. 2 shows preliminary test on reinforced specimen with applied load perpendicular to the grain. Because small quantity of samples for determination of stiffness, strength and ductility, results will be used for the calibration of numerical models.

After completing the tests on small samples with dimension of the $100 \times 12 \times 20$ [cm], and determining the resistance of the joint through the numerical models, will be cared out the tests on the four prototypes of truss girder. Prototypes of girder will be divided into two groups, those with a local reinforcement and without local reinforcement connection.

Fig. 1 Reinforcing gluelam timber specimens

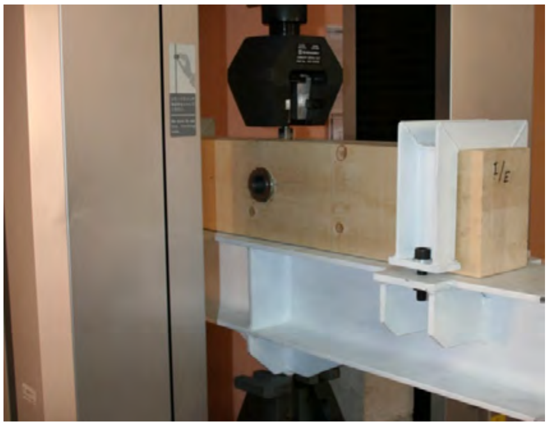

Fig. 2 Testing of reinforced specimen

Wood samples for testing the joints will be made from laminated timber GL24h. Bonding of laminated timber and fabric will be conducted with melamine adhesive. The fabric with dimension of 200 mm × 400 mm manufactured by Kelteks will be inserted in place of the future connection. The type of fabric that will be used is WR 900HA 145 weight 900 g/m^2. Glass-fibre textile will be glued between the timber layers, in the glue layer. Reinforcing of the upper and lower chord with fabric will be performed in classical lamination process (Fig 1).

All steel components will be made of high strength steel. Steel pipe will be made of steel with a yield strength of 450 N/mm^2, and a thread rod and connectors will be made of steel with a yield strength of 650 N/mm^2. The main element is a M16 screw, quality 12.9 which is mounted on the underside of the chord, passing through the steel pipe with diameter of 50mm and on the other side of the chord enters in the diagonal, in which is mounted by screwing. Mechanical property of threaded rod which is glued-in diagonal element of truss girder will be equivalent to screw 8.8., and at its end will be constructive sleeve into which the screw mounted.

2.2 Research on numerical models

Experimental researches on specimens, because of the limited number of samples, will be expanded numerical models. It will be shown the impact of numerical model complexity to the result accuracy. Numerical model, in which the wood is modelled as an orthotropic elasto-plastic material and with crack propagation for tension stress perpendicular to the grain will be compared with numerical model in which the wood is modelled as a elastic material.

Research on parametric numerical models will be explored areas that experimental research has not included. Fig. 3 and 4 are the first numerical models with applied force at angle 90° and 45° compare to the grain.

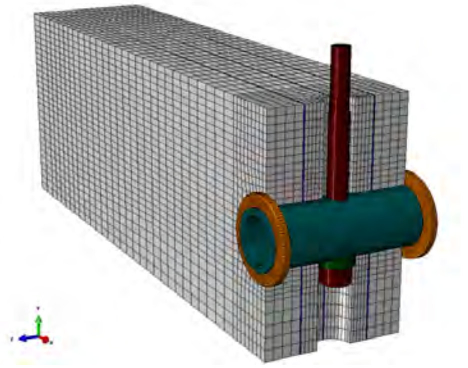

Fig. 3 Numerical model with applied load perpendicular to the grain

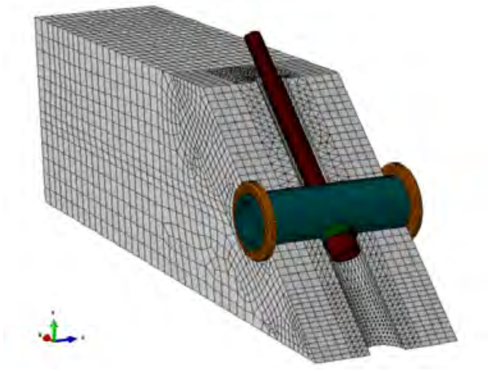

Fig. 4 Numerical model with applied load angle 45° compared to the grain

2.3 Expected research impact

It is expected that the new connection with steel pipes and glued-in rods will replace the commonly used steel plates in truss girders. Quick installation with this type of connection should give greater advantages to the truss girder against the solid timber girder. Results of parametric analysis will give the complete picture of the behaviour of these joints. It is planned that the research will give a ultimate load of these connection depending on the coefficient of reinforcement, the size of the steel pipe and angle of applied load.

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Seismic behaviour of timber frame multi-storey buildings

Daniele Casagrande, Paolo Grossi, Tiziano Sartori, Roberto Tomasi[1]

Summary

CHI-QUADRATO is a company consortium which cooperated with DIMS (Structural and Mechanic Department of University of Trento) in order to investigate the seismic behaviour of timber frame multi-story building. The research program has been divided in three different phases of work in order to test the individual building components and to quantify the interaction of them during seismic loading of full-scale timber construction.

The first phase the object has been dedicated to the investigation of the behaviour of the nailed connections between the timber studs and the board sheathing, necessary to assure the lateral stability of the timber frame walls, and of the connection systems to foundations.

In the second phase tests on full scale timber frame walls under quasi-static and cyclic lateral loads were performed. In order to study the interaction between the individual walls during seismic loading of a timber frame building, a full scale 3 – story timber frame building shake table test has been performed. The 1979 Montenegro Earthquake ground motion, recorded at Ulcinj-Hotel Albatros, was selected as the ground motion for seismic tests. The maximum peak ground acceleration was scaled to 0.07g, 0.27g, 0.5g. 0,7g and 1g in order to evaluate the performance of the building at different levels of seismic inputs.

A frequency and damping evaluation tests were used before and after each seismic test to identify natural frequencies, modal shapes and equivalent viscous damping ratio, exciting the structure with a low amplitude white random noise. After each seismic excitations the specimen was inspected for evidences of damage. The building designed for a 0,28g PGA representing a hazard level of 10% probability

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of exceedance in 50 years or a return period of 475 years, showed no visual damages at all the stage of the tests. The illustrated experimental campaign represent an effort towards a better comprehension of the seismic behaviour of multi-storey timber building in seismic zones. Within a frame of an European project, specimens with the same geometry but with other timber technologies (CLT, log house system) will be subjected to a similar seismic test. Moreover a specimen with timber frame system complete with external lining (insulation, door and windows) will be tested on the shacking table.

1. Introduction

During the last years many research projects and tests have been performed in order to understand the seismic behaviour of timber frame buildings. These projects were made almost exclusively in U.S.A., Japan and New Zealand, where the timber frame structural system is widespread and an high seismic hazard is present, showing good performances of the timber frame building. However there are a lot of differences between the European timber frame system and the others countries ones. Firstly, the timber elements have larger dimension, getting a more massive building; secondly, the system is characterized by the process of prefabrication.

Fig. 1: Timber frame wall assembying

Fig. 2 :Timber frame walls positioning on site

2. Seismic Tests of Chi-Quadrato Research Project

2.1 1st phase: connection systems tests

CHI-QUADRATO is a company consortium which cooperated with DIMS (Structural and Mechanic Department of University of Trento) in order to investigate the seismic behaviour of timber frame multi-story building. The research program has been divided in three different phases of work in order to test the individual building components and to quantify the interaction of them during seismic loading of full-scale timber construction.

The first phase the object has been dedicated to the investigation of the behaviour of the nailed connections between the timber studs and the board sheathing, necessary to assure the lateral stability of the timber frame walls, and of the connection systems to foundations. A lot of connection systems and different sheathing boards were tested to understand resistance, stiffness and ductility of each of them.

Fig. 3 Hold Down Test

Fig. 4 Stud – sheathing nailed connection Test

2.2 2nd phase: wall tests

In the second phase tests on full scale timber frame walls under quasi-static and cyclic lateral loads were performed. The main object of this phase has been to study the interaction of individual components, already tested in the previous phase, in a full-scale wall. Load - displacements diagrams have been obtained for each test configuration.

Fig. 5 Full-Scale timber wall test

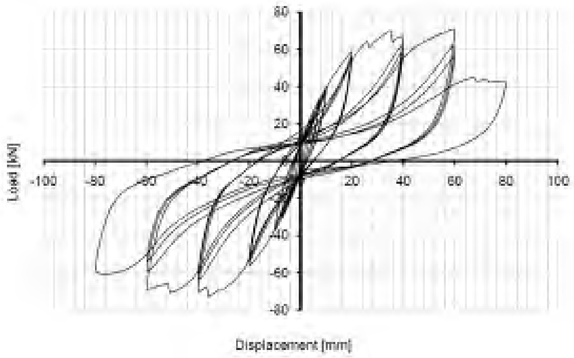

Fig. 6 Load - Displacement diagram for a cyclic test

2.3 3rd phase: full scale 3-storey timber frame building shaking table test

In order to study the interaction between the individual walls during seismic loading of a timber frame building, a full scale 3 – story timber frame building shake table test has been performed. The design was prepared based on the engineering design provisions of the Eurocode 8 and Eurocode 5 for a 0.28 g Peak Ground Acceleration and assuming a q-factor of 4. Six seismic tests were performed scaling the Ulcinj – Hotel Albatros Montenegro Earthquake accelerogram in order to investigate the behaviour of building for different limit states: a PGA of 1.00 g was reached during the 6th test.

The test structure was instrumented with nearly 100 digital instruments to measure forces (Hold down load cells), displacements or deformations (LVDT, string potentiometers, strain gauges) and accelerations. In addition a vision system was used to measure position of markers attached on a west side of building

Fig. 7 Full-Scale timber building shake table test

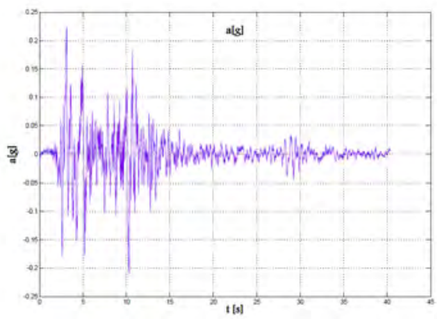

Fig. 8 Acceleration Time-History of Unscaled Ulcinj – Hotel Albatros Ground Motion

Before and after each seismic test a frequency evaluation test was performed in order to identify the natural frequencies and mode shapes of the test structure. During these test the structure was excited using a low amplitude flat white noise.

3. Analytical and numerical model

An analytical formulation and an elastic numerical model were developed to predict the elastic lateral displacement of timber frame walls, take account of all components of deflection: sheet shear, sheet nail slip, wall rocking, rigid wall translation.

The analytical formula was used to create a simplified wall numerical model, useful to make an elastic building model. The forces distribution between the walls was studied.

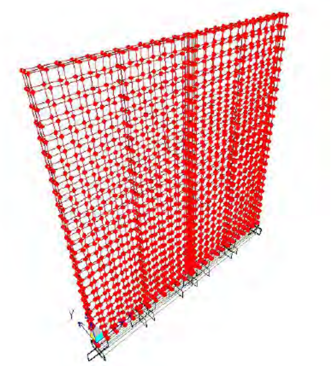

Fig. 9 Complete elastic wall numerical model

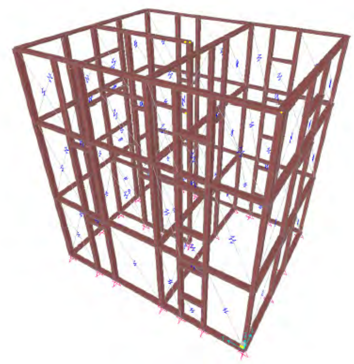

Fig. 10 Elastic model of a 3-story building

4. Conclusion and future work

The illustrated experimental campaign represent an effort towards a better comprehension of the seismic behaviour of multi-storey timber building in seismic zones. Within a frame of an European project, specimens with the same geometry but with other timber technologies (CLT, log house system) will be subjected to a similar seismic test. Moreover a specimen with timber frame system complete with external lining (insulation, door and windows) will be tested on the shacking table.

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Monotonic and cyclic in-plane behavior of CLT panels tested by using different types of metal devices

Ermanno Acler, Paolo Endrizzi, Roberto Tomasi[1]

Summary

This paper deals with the in-plane behaviour under monotonic and cyclic loads of cross laminated timber panels mounted by using novel types of connection systems to the ground. This research directly addresses the problem of the behaviour of connection systems in CLT structures. An extensive experimental full-scale campaign on a large number of possible configurations is presented. Tests were carried out in the laboratory of the Department of Structural and Mechanical Engineering of the University of Trento, Italy. An F.E.M. modelling has been moreover carried out to predict the force-displacement behaviour of the wall on the corresponding real configuration; it focuses on explaining the single contribution that must be considered to describe in all the components the wall behaviour.

1. Introduction

Several CLT (cross laminated timber) structures have already been built around the world during the last few years. This type of constructions is gaining a lot of popularity because of his speed of construction, the possibility of a high industrialization of the production process and the good performance in resisting to lateral loads when the connection systems are optimized. In these systems the resistance to horizontal actions - wind and earthquakes - is totally entrusted to the metal connector systems arranged in such a way as to absorb the horizontal forces and to prevent the uplift effects of the wall. Angle brackets are used as special connections to ensure the transfer of shear actions, and the so called "hold-down" adopted in order to counteract the tense actions causing the rocking behaviour of the wall.

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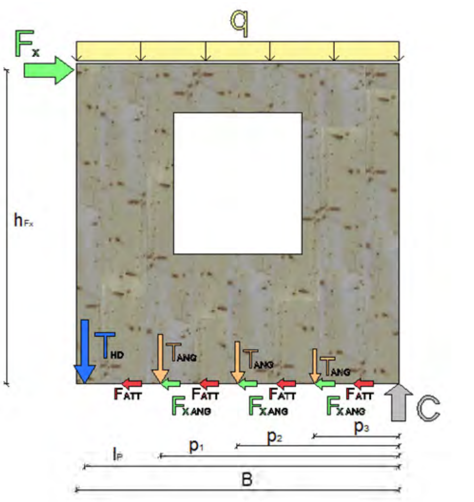

Fig. 1: Configuration system of the forces providing the equilibrium of the walls

The behaviour of the entire structure is directly influenced by strength and stiffness of metal elements. Because of the high in-plane stiffness of the wooden panels, it is reasonable to consider the contribution of the connector deformability on the global displacement of the wall as fundamental. The necessity to characterize the behaviour of the single wall as part of an entire CLT structure is one of the reasons that led to perform the experimental campaign which is herein presented.

2. Materials and Test Methods

Test specimens were square wall segments 2.50 m by 2.50 m, 3-layers CTL panels and were used for all the tests. The total thickness of the panels was 90 mm, each layer being 30 mm thick. The panels were delivered in standard production dimension. The specimens were processed directly in the laboratory to obtain the final test size. The panels were manufactured according to the European Technical Approval 08/0271 standards.

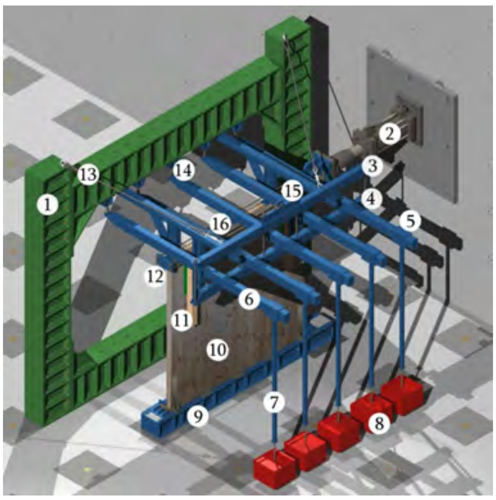

Fig. 2: Special full scale experiment apparatus for monotonic and cyclic tests on CLT walls

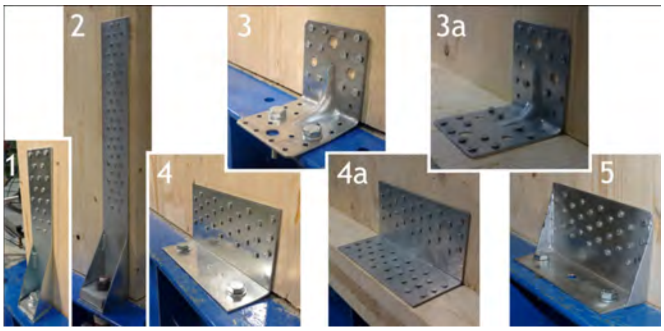

Fig. 3: Steel metal devices (from standard industrial production and new types) used on the testing campaign

1) hold-down type 1(340 mm heigh) 2) hold-down type 2 (620 mm heigh) 3 -3a) steel angle bracket type 1 4 -4a) new angle bracket type 1 5) new angle bracket type 1 with ribs

All the tests were performed with three angle brackets spaced at 625 mm, with two hold – down at the endings. All metal connection devices were placed only on the front side of the wall. During all the tests the horizontal load was applied by an hydraulic actuator set in displacement control. Two different vertical load configurations were used: the first one without vertical load and the second one by using a 20 kN/m vertical load configuration. All the tests were carried out both in the case of monotonic and cyclic test protocol. A total amount of 4 linear variable displacement transducers (LVDTs) were positioned on the wall in order to measure all the displacement contributions which characterize the global displacement. LVTD nr.1 was used to get the overturning displacement data of the wall when the load is monotonically applied; LVTD nr.2 was used to measure the same data only during the cyclic tests; LVTD nr.3 measured the horizontal relative displacement between the ground and the bottom surface of the panel; LVTDs nr.4 were applied to estimate the shear deformation of the CLT panel in his plane

3. Tests results and Analysis of the displacement contributions

3.1 Qualitative findings

For all tests the in-plane shear deformation of the CLT panels could be considered absolutely negligible. Referring to the failure of specimens it was observed that collapse never occurs on the panel itself but involved in different ways the brackets or the hold-down, depending on the test. In all tests the maximum wall horizontal displacement, measured at the bottom surface at the same step in which global failure occurs was less than 3.5 mm.

It was observed that starting from a lower level of horizontal external load the wall uplift occurred. As the horizontal load increased the wall uplift became more evident. Failure of specimens, as a consequence of this ratio between the height and span of the walls, never involved exceeding the maximum shear capacity of the brackets, in the case of the lighter *type 3) – figure 1* bracket as well.

As expected the behaviour of the wall, both in terms of initial stiffness and ultimate load, is strongly influenced by the hold-down. For the case of *type 1) – figure 2* hold-down failure occurred because of a composed mechanism that involved exceeding the shear-withdrawal capacities of the vertical nailed connection. A strong difference for the case of the stronger *type 2) – figure 2* hold-down was observed. In all those tests, no matter the type of the bracket and the load configuration, a brittle fracture on the vertical steel plate occurred because of the higher total strength of the nailed connection compared to the axial strength in tension of the plate itself *(figure 4).* The vertical load acting at the top of the panel seem not influence the behaviour of the walls in terms of the ultimate collapse mechanism.

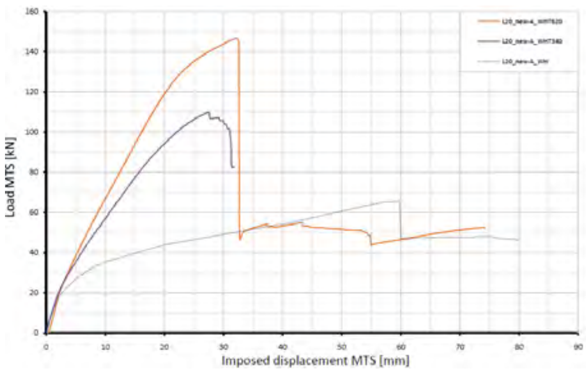

Fig. 4: Role of the hold down on the global force displacement behaviour of the wall

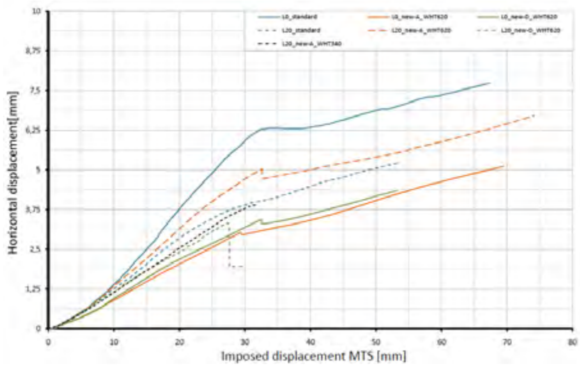

Fig. 5: Horizontal base displacement versus imposed displacement curves

It is to be noted that initial stiffness does not present large differences between tests with the same hold-down type; this underline one more time that the global stiffness of walls is completely influenced by the hold-down behaviour. That being noticed and predicted hold down axial contribution has been investigated in depth during the first push – out campaign [1]; this campaign was carried out on the specific purpose to evaluate the force – displacement diagrams of the single metal devices. Those data allow now to calibrate some specific non *linear links* in FE modeling in order to describe the global force – displacement of the wall connected to the ground by using several different types of steel angle brackets. It is moreover reasonable to suppose that during the first steps of the test, friction and axial stiffness of the brackets play a significant role in reference to the initial global stiffness of the walls.

3.2 Numerical modeling of the in-plane behaviour of the wall

In order to describe the wall behavior under monotonic loads several numerical models were carried out. That process has been made by using special *non linear links* calibrated taking into account the experimental force displacement diagrams from the push-out tests. This models take into account all the contributions that were supposed to have an important role in describing the global behavior. Several factors were considered in order to perform those F.E. models: the shear strength and stiffness of the angle brackets, the axial strength and stiffness of the hold-down and 2 other contributions like the axial stiffness of the brackets and the friction contribution.

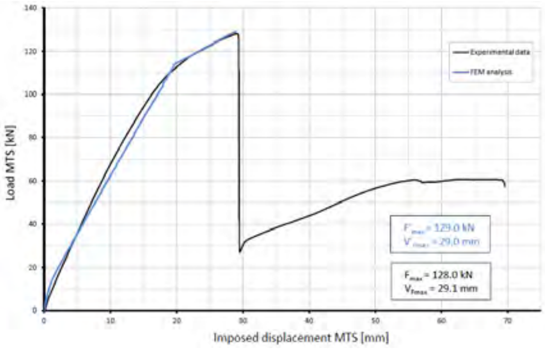

Fig. 6: Numerical force-displacement monotonic diagram for test with type 4) bracket and type 2) hold-down – 20 kN/m as distributed load

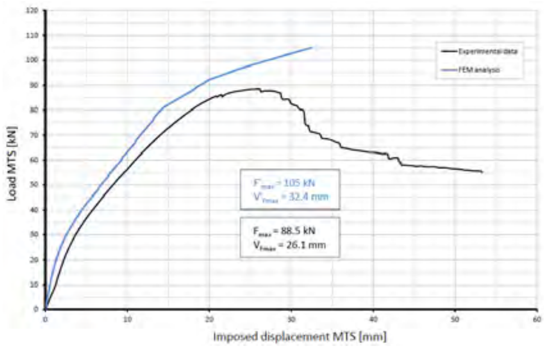

Fig. 7: Numerical force-displacement monotonic diagram for test with type 3) bracket and type 1) hold-down – 0 kN/m as distributed load

It is to be noted that F.E. modeling allowed to understand the role of the friction between the wall and the concrete slab and the axial stiffness of the brackets. A real F.E. prediction of the global behaviour of the wall could not be done without taking into account those contribution.

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Axial glued-in rods in ductile moment resistant steel-timber connections

Mauro Andreolli[1], Roberto Tomasi[2]

Summary

A recent research campaign investigated the mechanical characterization of a joint, suitable for different configurations within a heavy timber frame, consisting of a wooden element connected to a steel stub by means of an end-plate and glued-in steel rods. This connection system has some interesting properties in terms of mechanical performance, versatility and high level of prefabrication.

An analytical model to predict the joint response in terms of its key parameters (e.g. failure mode, ultimate resistance, stiffness and rotation capacity) is proposed and validated through an extensive experimental campaign. The component method, originally proposed for semi-rigid joints in steel frameworks, is adapted in order to set up a feasible general model for steel to timber joints, enabling application of the capacity design approach and offering the required ductility for applications in seismic zones.

The tests carried out indicate satisfactory agreement between theoretical and experimental results: the reliable prediction of joint failure modes allows design of moment resistant connections that can sustain high plastic deformation without brittle rupture, with a remarkable degree of global ductility and energy dissipation under alternate loading.

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1. Introduction

Glued-in bars can be employed successfully both in restoration and in new construction, exploiting their good performance in terms of strength and stiffness. Several authors investigated theoretically and experimentally the mechanical response of the single glued-in bar, inserted both parallel and perpendicular to the grain (for a comprehensive state of the art, see del Senno et al., 2004). Within the frame of the European Community financed GIROD research program, there has been an international effort to develop this knowledge (Bengtsson and Johansson, 2001). The outcomes of these studies are some proposed expressions to estimate the pull-out resistance. On the code side, no recommendations for the design of glued-in steel bars are provided by European standards, while detailed rules are reported in DIN National Annex (DIN EN 1995-1-1/NA:2010-12).

A possible use of glued–in rods is in moment resisting frame connections in heavy timber structures: these solutions involve some interesting properties in terms of mechanical performance and aesthetic appearance (since the steel bars are embedded in the timber elements). The development of prefabricated connection systems also favours the reduction of structure erection time and limits work at the construction site.

In Tomasi et al. (2008) a joint for different configurations within a frame was presented, where a timber element is connected to a steel stub by means of an end-plate and glued-in steel rods. An analytical model to predict the joint response in terms of its key parameters (e.g. failure mode, ultimate resistance, stiffness and rotation capacity) is proposed and validated through an extensive experimental campaign. The joint is conceived and designed to allow plastic flexural deformation of a steel end-plate and to preserve the elastic brittle failure mode of the bar, in order to obtain a connection able to sustain high plastic deformation without brittle rupture.

The moment connection is presented in Fig. 1: a stub of steel section (1) is connected to a glued laminated timber element (2) by means of: an end-plate welded on the steel profile (3); steel bars glued-in parallel to the grain (4); a glued-in steel plate inserted in a slot cut in the timber element (5).

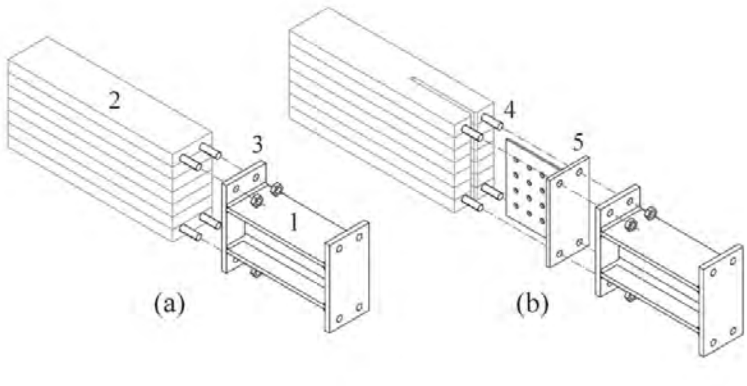


Fig. 1 The end-plate steel to timber joint: without (a) or with (b) a glued-in steel plate inserted in a slot grooved in the timber element.

The configuration of the joint assures transmission of the bending moment (through end-plate and steel bars) and of the shear force (through the glued-in steel plate) between the timber element and the steel section.

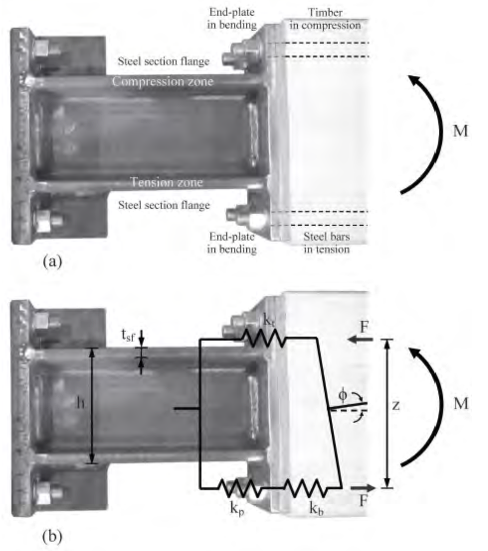
When shear forces are low, the joint can be manufactured without the glued-in steel plate: in this case the transmission of lateral loads can be directly supported by the steel bars.

2. The component method

The overall rotational response of the joint is simulated by means of a rotational spring, whose behaviour is approximated by a simplified trilinear moment-rotation curve. In accordance with the European standard EN 1993-1-8 (CEN, 2005), such a model is fully defined by the following parameters:

- moment resistance $M_{j,Rd}$
- initial rotational stiffness $S_{j,ini}$
- rotation capacity ϕ_{Cd}

The "component method" (Jaspart, 2000, EN 1993-1-8, 2005) considers the joint as an "assembly" of components and it enables evaluation of the strength and stiffness of the joint, on the basis of the response parameters of each component. The application of the component method requires the following steps: 1. identification of the basic joint components; 2. mechanical characterization (strength, stiffness and deformation capacity) of each component; 3. "assembly" of the components and computation of the global joint parameters. The timber-steel joint components in the tension zone and in compression zone are identified in Fig. 2a (step 1).



The evaluation of the mechanical characteristics of each basic component (step 2) should account for the presence of the "timber components" (wood in compression, glued-in steel bars), which are not dealt with by the European standard for steel structures (EN 1993-1-8, 2005, Andreolli et al., 2011).

Fig. 2 (a) Basic joint components for the steel to timber joint analyzed; (b) model for the internal force distribution and related stiffness.

3. Experimental analysis

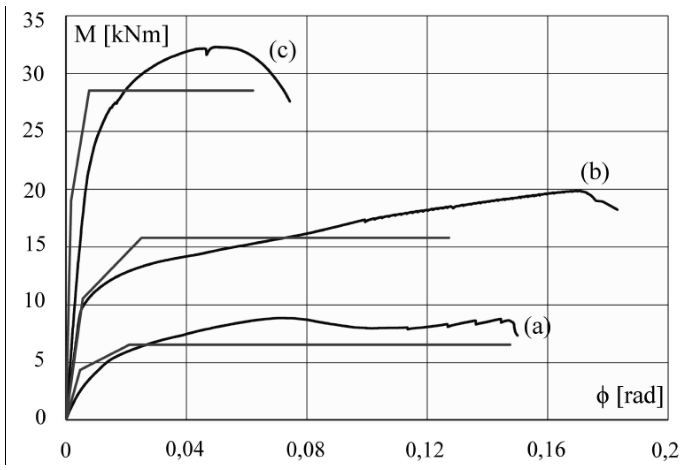


Fig. 3 Experimental and theoretical results for moment-rotation relationship: specimen with thickness of the end-plate t_f = 6 mm (a); t_f = 10 mm (b) t_f = 20 mm (c)

Eight type of specimens, with the same geometry for timber section (120×240 mm) and for steel element (HEB 120), were prepared for monotonic and cyclic tests. In order to investigate the behaviour of the connection, with special regard to the joint ductility and to the influence of the different failure modes on the capacity in terms of ultimate rotation, the thickness t_f of the end-plate (steel grade S235)was varied from 6 to 20 mm (t_f = 6 mm; 8 mm; 10 mm; 15 mm; 20 mm).The theoretical moment-curvature relationship was obtained using the analytical method referred to above (Andreolli et al., 2011): for three specimens chosen as representative of the three theoretical failure modes, Fig. 3 illustrates the experimental moment–rotation relationships compared with the trilinear theoretical curves.

4. Conclusion and future work

This connection system involves some interesting properties in terms of: mechanical performance, since semi-rigid connection can be designed without significant reduction of bending capacity; versatility, since various geometries can be adopted for construction of moment resistant joints (e.g. corner and foundation frame joints, see Fig. 4); high level of prefabrication, since the construction assembly can be done using steel structure methods, reducing the erection time of heavy timber structures and limiting work in situ.

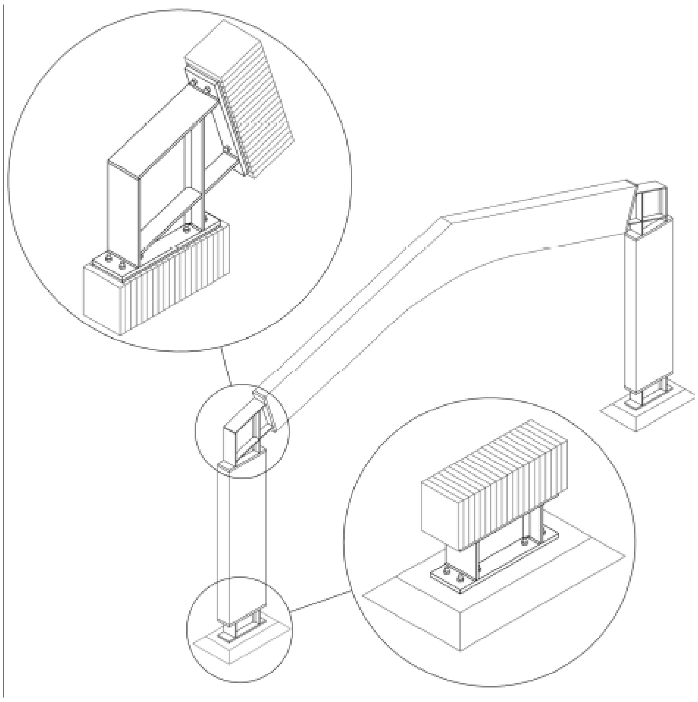


Fig. 4 Possible applications of the analysed joint in timber portal frames for moment-resistant joints at corners and foundations

This study stresses the need to have a reliable design model for practitioners and researchers in order to correctly predict the mechanical behaviour of the joint also in terms of capacity design. The illustrated experimental results validated the approach proposed by the authors, where the component method, originally proposed for steel structures, was adapted for steel to timber joints.

Future research efforts will be devoted to numerical investigation of the proposed steel to timber joint in single and multi-storey heavy timber frameworks. Non-linear seismic analysis (e.g. push-over analysis) will also be performed in order to determine the behaviour factor to be adopted in design as a function of the inherent ductility of the joints.

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Glued-in-rods for roundwood structural applications

Telmo Morgado[1], Alfredo Dias[2]

Summary

In Portugal, given the large proportions of natural regeneration pine stands, a large amount of young trees must be removed from the forest, in order to assure the quality of mature trees and to decrease the risk of fire. The main objective of the project is to provide the basis for the use of small diameter Maritime pine poles in structural applications. The study has two main tasks. The first task comprised the determination of mechanical properties, the establishment of visual and mechanical grading procedures, and is already completed. The second task concerns the development of connections to promote the use of roundwood in construction. This paper presents results within the second task, namely, connections using glued –in –rods.

1. Introduction

The performance of the structural applications of roundwood greatly relies on the performance of the connections. The Portuguese Maritime pine roundwood due to its mechanical properties has great potential for use in structural applications. One of the most significant difficulties are connections. Stern [1] affirms that "Effectively connecting small-diameter timber roundwood is a major problem that must be mastered before small-diameter timber can be used in its most effective manner".The existent connections for roundwood are expensive, hard to implement, have aesthetic problems and must be created for each particular case, preventing mass production of connections. Wolf [2] states the need to "focus research on the development of economically feasible connections to transfer axial loads and bending moment".

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The most common used roundwood connections are the block joint (sometimes using a dowel instead of a steel block) and the central plate. Other type of connections are the glued – in-rods based solutions. This type of connection has a superior aesthetic when compared with the other connections. They have been the focus of various studies in other applications, mainly with glulam. This paper presents a study regarding the application of bonded-in-rods in small roundwood timber elements.

2. Experimental procedure and results

The experimental procedure had two phases. Within the first phase the length of anchorage was studied. In the second phase the configuration of connections was defined and experimental tests performed.

The study of the anchorage length was done in 17 round timber elements of large diameters, between 186 mm and 241 mm, such configuration allowed various pull out test to be performed in each one. The density of timber varied between 496 kg/m^3 and 697 kg/m^3. The rod had a nominal diameter of 10mm and a failure load around 31 kN. The glue used was Icosit K101 TW due to its low viscosity, an important gap filling property.

In the wood elements holes of 12mm diameter and 50mm, 75mm, 100mm and 125mm depth were drilled. The rods were glued in the holes and tested 72hours later. The pull out test is presented in *fig.1*. In order to minimize the interference in area surrounding the rod, a disc with an internal diameter of 50mm was used. It was placed below the bar, that is used for the pull out test (*fig.2)*, assuring that no compression near the glued in rod. The results obtained are presented in *table 1*.

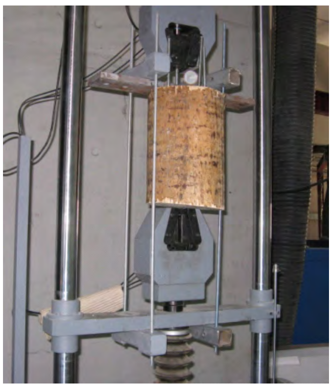

Fig. 1 Pull out test

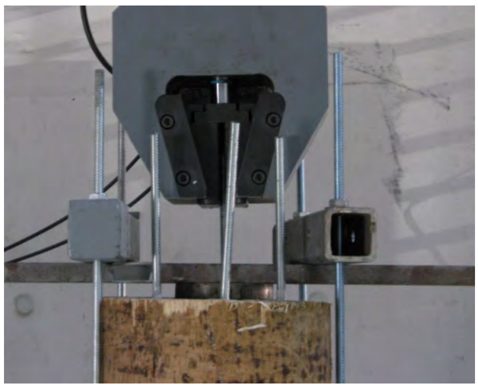

Fig. 2 Pull out test detail

The pull out tests with a depth of 50mm showed, mainly, failures in the wood (*fig.3*), nevertheless, in 5 tests the failure ocurred in the glue (*fig.4*). The tests with a depth of 75mm presented failures mainly by the rod (58%), being the remaining failures in the wood (26%) and in the glue (16%). In the tests with the depth of 100mm and 125mm the failures were only in the rod. Based on these results, it was concluded that, for the conditions used here, when a glue length of 10 times the diameter or higher is used, the failure will occur in the rod. This is a preferable failure due to its ductile characteristics.

Table 1 – Results of pull out tests

Depth (mm)	Failure load (kN)				N.º Tests
	Mean	Max	Min	Std	
50	23,82	30,65	19,21	3,33	22
75	30,54	32,50	26,55	1,31	38
100	31,37	33,85	28,63	1,16	36
125	31,67	32,85	30,59	0,67	20

Fig. 3 Failure in the wood

Fig. 4 Failure in the glue

The final configuration of the connections consisted of 4 glued-in-rods, placed in timber elements with a diameter between 120mm and 140mm. A minimum spacing of 25mm between the rod and the outer circumference of the section and a minimum spacing of 50mm between rods, in a square configuration, were assured. The same quality rods of that used in the pull out tests were used, together with a depth of 100mm and a hole diameter of 12mm. The test followed standard EN-26891 [3] procedure. The test arrangement is presented in *fig.5*. A hinge was used in order to assure a similar load distribution in the four rods (*fig.6*). The results of failure load and stiffness are presented in *table 2.*

Fig. 5 Test arrangement of the connections

Fig. 6 Failure of a connection

As it was expected the failure occurred in the rod. There was only one exception caused by a fissure next to a rod, in this situation the failure occurred in the wood. Due to the connection configuration the failure load is around four times the one obtained in the pull out tests of single bars. The stiffness presented a high value, which was to be expected, due to the configuration of the connection and to the stiffness of its components.

Table 2 – Results of the test of 10 mm glued-in-rod connections

	Failure load (kN)	K_s (kN/mm)	ρ_{12} (kg/m^3)
Mean	126,5	128,2	573,0
Max	133,3	277,3	682,8
Min	111,9	55,6	417,0
Std	4,0	47,1	60,1
N.º Tests	49	84	50

Further tests, but with a lower number of specimens, were performed with rods of 8 mm and 12 mm diameter. The configuration of the rods was similar to the one presented before, being the depth of the holes 10 times de rod diameter and the diameter of the hole equal to the rod diameter plus 2 mm. For the connections with 8mm rods, timber elements with a diameter between 100 mm and 120 mm were used, whereas the 12 mm rods were glued in round timber elements with a diameter between 140 mm and 160 mm. The results for these connections are presented in *table 3*.

As in the previous connections the failure was mainly in the steel rod, confirming that 10 times de rod diameter with a glue thickness of one millimeter assures this type of failure. Likewise in the 10mm rod connections, the stiffness presents a much higher variability than the one obtained for the failure load.

Table 3 – Results of the test of 8mm and 12mm glued-in-rod connections

	Failure load (kN)		K_s (kN/mm)		ρ_{12} (kg/m3)	
Rod diameter	8 mm	12mm	8 mm	12mm	8 mm	12mm
Mean	74,1	155,3	109,2	171,6	554,6	574,3
Max	75,6	158,9	217,3	301,9	674,6	677,2
Min	71,5	151,0	48,7	103,2	465,1	506,0
Std	1,1	2,2	46,9	54,7	66,3	52,3
N.º Tests	19	15	28	24	20	20

3. Conclusions and future work

This paper presents the results from the study of three configurations of connections using glued-in-rods in small diameter Portuguese Maritime pine. The results obtained showed that this connection has great potential, since presents good aesthetics and shows a high failure load and stiffness. Future work will focus on the development of nodes for this type of connection and later on the development of prototype structures using Maritime pine round timber members.

4. Acknowledgments

The authors would like to thank the support from Fundação para a Ciência e Tecnologia through PhD scholarship SFRH/BD/44909/2008 and research project PTDC/ECM/099833/2008.

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Durability of bonded-in rod connections

João Custódio[1], Helena Cruz[2], James Broughton[3]

Summary

This project aims to provide a better understanding of the performance and durability of bonded-in rod connections used for the in-situ repair and reinforcement of historic and contemporary timber structures. The specific objectives of this study are the: (a) determination of typical environmental service conditions expected for these rehabilitation systems; (b) assessment of the effect of preparation and service conditions on the performance and durability of the bulk adhesives; (c) investigation of possible ways of improving the durability of adhesively bonded structural timber joints; (d) development of an experimental methodology to assess the durability of bonded-in rod connections with relation to realistic working loads and the effect of realistic thermal and moisture effects; (e) provision of recommendations for test procedures and acceptance criteria for the selection of candidate repair system materials for specific applications.

1. Introduction

Connections and reinforcement employing rods bonded in to timber have been used for many years. Although these connections have been found to exhibit high initial bond strengths to timber, and excellent durability in applications with other substrates, no test standards or commonly accepted specifications exist today for assessing and approving adhesives to be used for bonded-in rod applications. Because of this, concerns about their reliability still persist. The project here

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presented was created to address these concerns so that the better understanding of the performance and durability of these bonded systems and the development of a test method and associated acceptance criteria for the selection of candidate repair system materials for specific applications may contribute to their wider exploitation and to the development of models to predict the long-term behaviour of bonded-in rod connections.

2. Project overview

2.1 Environmental service conditions

Today's prevailing belief of practitioners is that, since the bond-lines in a structural joint are hidden in the interior of the timber element, they experience considerably lower temperatures compared to the ambient climate due to the low thermal conductivity and specific heat of wood. Epoxy adhesives, while not ideal, are currently the best generic adhesive type for in-situ repair operations. However, as they soften and lose strength at relatively low temperatures [1], epoxy repaired timber under the action of fire or under high service temperatures could therefore be potentially hazardous. Thus, because of the sensitivity of epoxy to heat at relatively low temperatures (in the range of 30-80 ºC depending on the epoxy formulation), the performance of an epoxy-repaired joint during a fire would depend primarily on the insulating properties of the wood and the distance of the bond-line from the surface.

For establishing suitable recommendations regarding this problem, the following questions need be answered: (a) What kind of service temperatures and relative humidity may be expected?; (b) What kind of temperatures will be reached inside a timber member in case of fire and in the case of normal summer conditions?; (c) Can we predict it?; (d) What are the consequences of heating a cured bonded joint on its immediate strength?; and (e) on its long-term behaviour or durability? This section of the project deals with the first three questions, whilst (d) and (e) are dealt with in the next two project topics, respectively "Bulk adhesive performance and durability" and "Long-term stressed joint testing". The details on the experiments conducted to investigate how effective the insulating properties of the timber are against excessive heating of the bond-line in a joint inside a structural timber element, as well as the results are not presented, but can be found in [2].

2.2 Bulk adhesive performance and durability

In applications such as the repair and/or strengthening of buildings using structural adhesives, the bonding is done in-situ in an uncontrolled environment. In such applications, the adhesive system and the ensuing composite is subjected to a range of environments including exposure to humidity, temperature variations, water over extended periods [3, 4]. In addition, due to their in-situ use, it is also vital to assess their glass transition temperature and the effect that the preparation conditions have on their mechanical properties. Thus, the experimental campaign carried out in this part of the project involved testing four commercial two-component structural epoxy adhesives to evaluate the effect that type of mixing, curing and postcuring temperature have on the viscoelastic properties of the adhesives; to evaluate the effect of water on the adhesives' physical properties; and to assess the long-term effects of service environment on the adhesives' durability. Again, the details on the experiments conducted and results obtained so far are not presented here, but can be found in [2, 5].

2.3 Long-term stressed joint testing

This part of the project involved the development of a test method to assess the long-term durability of bonded-in rod connections. The experimental campaign consists of exposing loaded test specimens to natural (NW) or artificial accelerated weathering (AAW), and then to compare the periodically evaluated deformation and residual bond strength. The AAW cycle used was specifically developed for this purpose and uses temperatures and relative humidities based on the data collected from field measurements. The test campaign was designed to evaluate several parameters that could influence the performance and durability of the bonded joints, namely substrate type, adhesive type, load and weathering environment. The tests were performed in specifically designed loading fixtures that were created with the objective of testing several replicates at the same time, using the minimum possible amount of materials, the space occupied by the set up was to be as small as possible, and the minimal effort required for the deformation measurements during the ageing of the specimens.

The rig developed consisted of a timber specimen (having a rectangular prism shape, measuring 80 mm in length, 80 mm in width and 10 mm in thickness, which derives from that described in the standard proposal (CEN

TC193/SC1/WG11 N48) bonded to a zinc plated threaded steel rod (having 10 mm in diameter) through a 2 mm thick bond-line (figure 1).

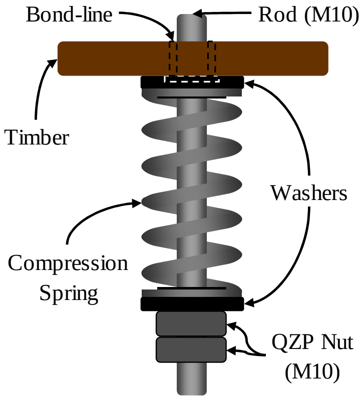

Fig. 1 Loading rig.

Two timbers were used, namely, maritime pine (*Pinus pinaster* Ait.) and European oak (*Quercus robur* L.). These species constitute the most common wood species used in Portugal and in the United Kingdom for the structural timber frames used in construction, where the repair/reinforcement techniques involving these structural adhesives are normally used.

The adhesives used in this study were both commercial 2-component structural epoxy adhesives. The details on the experiments conducted and results already obtained can be found in [2, 6].

2.4 Effect of surface treatments on joint performance

Epoxies form highly durable bonds with many substrates. However, they are usually not considered capable of forming completely durable bonds with timber. Because of that, epoxies are generally classified as adhesives not suitable for exterior wood bonds. Thus, epoxies have generally been limited to wood bonding markets where the other adhesives do not perform well, *i.e.*, applications where the bond is made at room temperature, with low clamping pressures and without intimate contact between adherends, *e.g.*, rehabilitation of timber structures.

The use of primers, coupling agents, and other surface treatments to enhance adhesion is now commonplace in the aerospace, automotive, and plastics industries, where they are used to develop highly durable bonds to metals, advanced composites, ceramics and plastics. However, such treatments are virtually non-existent in the wood products industry [7].

The experiments conducted in this project section were performed with the purpose of verifying if any of the surface pre-treatments studied would lead to an improved durability of adhesively bonded structural timber joints. The details on the experiments conducted and results obtained so far can be found in [2, 8, 9].

3. Conclusions and future work

From the project results obtained up to now, it was found that the service conditions to which bonded-in rod connections might be subjected to may involve long periods exposed to extreme temperatures and relative humidities (≥ 50°C and ≥ 80%rh). The preparation conditions and cure schedule were found to have a profound effect on the viscoelastic properties of the adhesives. Service conditions had significant combined effect on joint performance. The tests performed here prove that surface modification methods for adhesion promotion can be adapted to cellulosic substrates with significant improvements in bonded joint durability. Finally, the data collected so far, concerning the long-term stressed joint testing, appears to indicate that the developed method is predicting correctly the end-use performance for different adhesives and timbers. The most important features of the method developed to assess the durability of bonded-in rod connections are: (a) the direct evaluation of all the components involved in the repair/strengthening system, thus accounting for all the possible cross-influences of the different materials bonded together; (b) the exposure to a realistic load and realistic ageing conditions; (c) the use of small scale bonded assemblies requiring minimal space so that a number of different systems can be tested simultaneously in an environmental cabinet; (d) the absence of complicated jigs (labour intensive and with complex parts) and weathering equipment (*e.g.*, vacuum and pressure chambers); and (e) the relative short-time period needed to obtain a result for unsuitable materials.

Although this research has advanced the state of knowledge about the durability of bonded-in rod connections, the flowing topics require further investigation: the realistic performance assessment test (RPAT) should be used with other material combinations; the RPAT results should be compared with the behaviour of full-sized specimens; and typical service conditions should be assessed in more structures and in countries other than Portugal and the UK.

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Compression strength perpendicular to grain - full-scale testing of glulam beams with and without reinforcement

Roberto Crocetti[1], Per Johan Gustafsson[2], Daniel Ed, Fredrik Hasselqvist[3]

Summary

The result of 38 beam tests carried out to determine the compression strength perpendicular to grain at the supports of glue laminated beams are summarized in this extended abstract. The tests were carried out at the laboratory of Structural Engineering, Lund University, and they comprise both non-reinforced and reinforced beams. The reinforcement at the supports was achieved by means of (i) glued in steel rods, (ii) glued-in wooden rods and (iii) lateral screwed steel plates.

The main findings were:

– The strength decreases considerably with increasing support length

– The strength is significantly improved by glued-in steel or wood rods

– Although a higher load bearing capacity can be achieved by steel rod reinforcement, wood rods give higher stiffness at common service load levels

– Steel plates screwed at each side of the beam support, which are often used in practice to improve the lateral stability of timber beams, considerably increase the compression strength perpendicular to the grain

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1. Introduction

Timber has very low strength and stiffness perpendicular to the grain. The structural design of many structural details is therefore governed by perpendicular to grain compressive stress. The strength for compression perpendicular to the grain is determined on the basis of testing standards, which are substantially dissimilar in different parts of the world. In Europe, for example, the standard EN408 is adopted for determination of the basic material properties. Such standard prescribes for compression perpendicular to grain a method where a block of timber is loaded in uniform compression over the entire loading surface. In EN408 is then the strength defined as the stress corresponding to a deformation equal to 1% of the height of the specimen. This testing method leads to characteristic values for perpendicular to grain strength in the range $f_{c90,k}$≈2-3 MPa for specimens made of coniferous species, with no significant discrepancies between higher and lower strength classes. Some design codes, e.g. Eurocode 5 gives the possibility to increase the basic characteristic value of perpendicular to grain strength $f_{c90,k}$ both (i) directly by the adoption of a magnification factor, referred as $k_{c,90}$, which depends upon the support conditions and (ii) indirectly by assuming an increased support length. However, in Sweden structures have been designed during the past three decades using a characteristic value for perpendicular to grain strength of 7 MPa for structural coniferous timber and 8 MPa for glulam, which are well above the values suggested by EN408 and Eurocode 5. Despite the adoption of such high strength values, to the authors' knowledge, neither failures nor evidence of damage due to too high compressive stresses perpendicular to grain have ever been observed in Sweden. An important task is therefore to understand whether the design according to European praxis, i.e. starting from "basic material properties" for perpendicular to grain strength, is reasonable or not.

2. Material and Methods

To investigate the perpendicular to grain behavior 38 full-scale laboratory tests of glulam beams with different sizes were carried out at the laboratory of Structural Engineering, Lund University, Sweden. Spruce glulam of strength class GL30c was used for all specimens. The mean density of the tested specimens was in the range 434-477 kg/m^3 and moisture content was approximately 12%. The load was displacement controlled and applied in a quasi-static fashion by means hydraulic jack. In order to create a more distributed load, thus reducing the risk for possible bending failure, a rigid steel beam was placed on the top of the tested timber beams. The timber beams were placed on two timber blocks with vertical grain direction. These blocks rested on Teflon plates to ensure low friction and negligible horizontal reaction force, as it commonly is in real structures. A total of four sensors measured the vertical deformation at each end of the beams. The sensors were placed in the middle of the supports, measuring the vertical deformations from mid- height of the beam to the upper surface of supporting timber block, see Fig. 1.

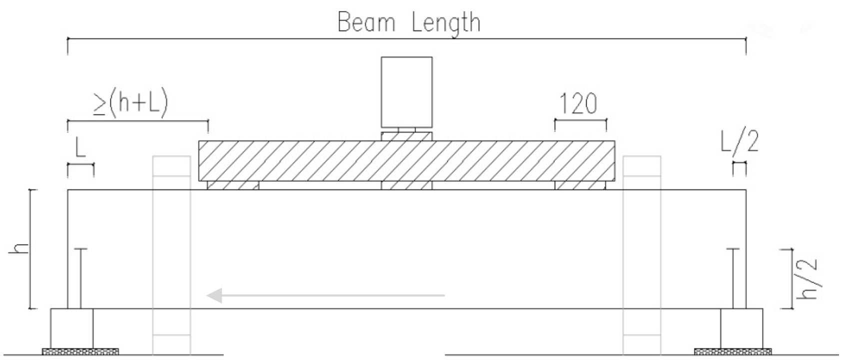

Fig. 1 Test setup of non-reinforced beams

The beam lengths were: 1,6 m for specimens with cross sections 90 x 270 mm^2 and 90 x 360 mm^2 and 2,6 m for of the beams with cross section 115 x 630 mm^2, see Table 1.

Tab. 1 Beams tested

Beam type	Length	Width	Height	Number of beams	Support lengths and (number of reinforcing rods)
Non-reinforced I	1600 mm	90 mm	270 mm	4+4+4	60 90 120
Non-reinforced II	1600 mm	90 mm	360 mm	1+1+1	60 90 120
Non-reinforced III	2600 mm	115 mm	630 mm	1+1+1	60 90 120
Steel plates I	1600 mm	90 mm	270 mm	3+3	60 90
Steel plates II	1600 mm	90 mm	360 mm	2	120
Wooden rods	2600 mm	115 mm	630 mm	2+2+2	60(2) 90(4) 120(6)
Steel rods	2600 mm	115 mm	630 mm	2+2+2	60(2) 90(4) 120(6)

In addition to the 38 beams, six number blocks of timber were tested in uniform compression. The cross section of the timber blocks were 90 x 90mm^2 and 115 x 115mm^2. The height of the blocks was 135mm, i.e. equivalent to three laminations. Eighteen beams were unreinforced, whereas the remaining twenty were equipped with some kind of reinforcement at the support areas. The reinforcement consisted of either (*i*) glued-in M12 4.6 threaded steel rods or (*ii*) glued-in smooth rods of hard wood or (*iii*) steel plates screwed at each side of the beam, connecting the beam to the underlying support. Numbers of tests are indicated in Table 1. The material of the threaded steel rods was 4.6 steel, the nominal diameter $d_s = 12$ mm and the length $l_s = 400$ mm. The material of the hard wood rods was birch (estimated mean compression strength $f_c \approx 55\text{-}60$ MPa), the diameter d_w=19 mm and the length l_w=400 mm. In both cases the rods were glued by polyurethane glue. The number of glued-in rods was: (i) two for support lengths L = 60 mm, (ii) four for support lengths L = 90mm and (iii) six for support lengths L = 120 mm. For the beams reinforced with steel plates, for each plate 18 screws were used to connect the plate to the beam and 18 screws to connect the plate to the wooden support. The dimension of the screws was 40 x 4,0. The dimensions of the steel plates were 2,5 x 80 x 240 mm^3. Fig. 2 shows the studied reinforcement methods.

(i) Reinforcement with glued-in threated steel rods

(ii) Reinforcement with glued-in birch rods

(iii) Reinforcement with screwed steel plates

Fig. 2: Types of tested reinforcement

3. Results and discussion

3.1 Unreinforced beams

The unreinforced beams with cross section 90 x 270mm^2 and 90 x 360mm^2 showed very similar performance, see Figure 3. The large beams with cross section115x630mm^2 showed considerably less stiffness, see Fig.3.

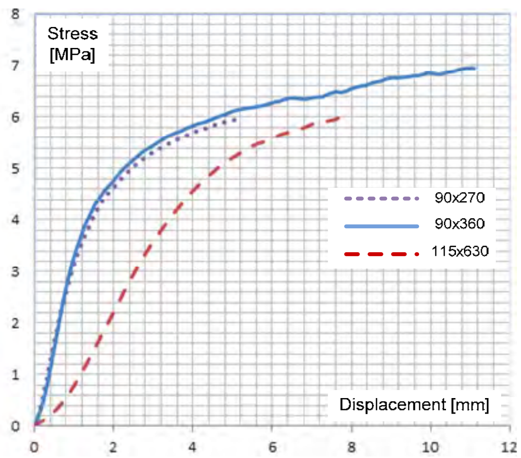

Fig. 3: Stress vs. displacement at the support. Support lengths 90 mm. Mean values

This discrepancy may partly be because - for a given strain - deeper beams show larger relative displacement than shallower beams. Another important observation is that the stiffness in terms of stress vs displacement considerably decreased with increasing support lengths. The slope of the stress-displacement curve in the range from 1 to 3 MPa was approximately 35% greater for support length L = 60 mm than for L = 120 mm.

3.2 Reinforced beams

Reinforcement by means of glued-in rods gave a significant enhancement of both stiffness and ultimate load capacity. At low load levels, the beams reinforced by glued-in wooden rods showed a greater stiffness than similar beam reinforced with glued-in steel rods. The opposite was observed at higher load levels, see Fig.4. Such a behavior may be the result of the manufacturing process. In the case of wooden rods a very smooth surface can be achieved at the support, e.g. by re-planing of the beam after the insertion of the roads. The same smoothness is normally not achieved in the case of steel rods. Therefore, relatively large deformations are needed at the support before all the steel rods will start to carry load. In both cases the failure of the specimens was due to buckling of the rods, which always occurred close to the bottom edge of the beam. Table 2 shows the enhancements of stiffness given by rod reinforcement for beams with cross section 115x630 and support length L = 90 mm. At small deformation the beams reinforced with wooden rods carried more load than the similar beams reinforced with steel rods. However, at arger deformations (i.e. larger than 5 mm for this specific case), steel rods performed better than wooden rods.

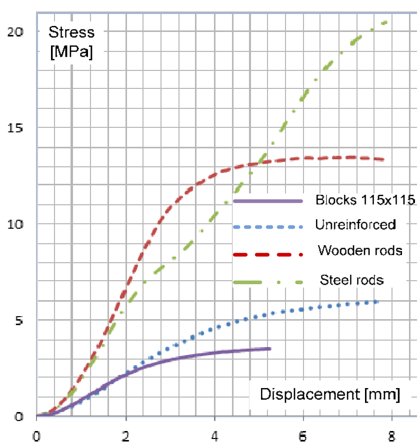

Fig. 4: Stress vs. displacement at the support. Beams 115x630. Support length 90 mm. Mean values

Table 2: Stress levels at different deformations of the supports. Beam115x630. Support length L = 90mm

Specimen	**Deformation**						
Type	**1 mm**	**2 mm**	**3 mm**	**4 mm**	**5 mm**	**6 mm**	**7 mm**
Unreinforced beam							
Stress (MPa)	0.8	2.2	3.6	4.6	5.2	5.6	5.9
Steel rods							
Stress (MPa)	1.9	5.7	8.1	10.4	13.2	16.6	19.2
Stress increase	144 %	157 %	125 %	128 %	154 %	198 %	228 %
Wooden rods							
Stress (MPa)	2.1	6.7	10.8	12.5	13.2	13.4	13.5
Stress increase	166 %	201 %	203 %	175%	153 %	141 %	130 %

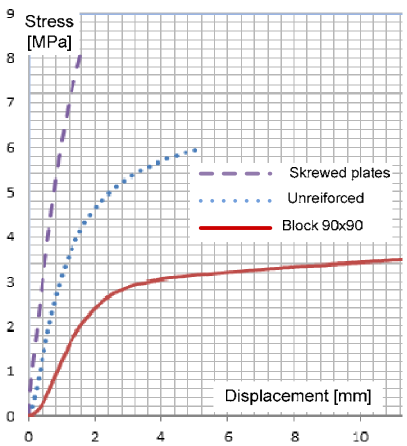

Fig. 5 Stress vs. displacement at the support. Beams 90x270. Support length 90 mm. Mean values

Also the reinforcement with steel plates screwed at each side of the beam gave a significant increase of both strength and stiffness. This is shown in Fig. 5 for the specimens with cross section 90x270 mm^2 and support length 90 mm. The main reason for the enhancement of stiffness and load bearing capacity is presumably that the screws carry a part of the support load by shear. Moreover, the lateral steel plates prevent the timber beam from expanding in the transversal direction, which to some extent may improve the resistance to compression.

It is interesting that lateral steel plates are almost always used in column-to-beam connections and if screws or nails are used as fasteners then the stiffness and strength of the beam support are apparently much greater than anticipated by strength design analyses not taking into account the influence of the plates.

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Comparison of different techniques for the strengthening of glulam members

Robert Widmann[1], Robert Jockwer[2], Roman Frei[3], Rafael Haeni[4]

Summary

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 which fits well into the scope of WG1 within COST FP 1004 (and also within FP 1101). The strengthening techniques include self tapping screws, glued-in rods, refilling adhesives in delaminations and CFRP tissues glued externally, where the first and last mentioned will be discussed here. In a later phase also other strengthening methods will be considered. In the course of the tests some of these methods are not only used to repair delaminated glulam beams subjected to tension perpendicular to grain and shear but also to reinforce weak parts of the members, e.g. at the supports and the loading points. As the project is strongly related to practical issues the work does not only focus on mechanical parameters but also on application issues in practice.

The project now is in an intermediate stage and the preliminary results obtained so far will be presented. The authors are also interested in discussions with other researchers about this topic in order to learn from their experience which might influence the future work in the project.

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1. Introduction

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.

2. Material and methods

2.1 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 x 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. Another series contained 10 beams with notches which represented the weak part. The length of the notched beams was 3.00 m while the missglued beams were 2.50 m long. 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.

2.2 Reinforcements

Two different reinforcement methods were used so far: self tapping screws 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. 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.

For the notched beams screws with different dimensions (13 mm x 800 mm and 16 mm x 800 mm) were used as reinforcements. The screws were arranged in two different orientations, perpendicular to the span and in an angle of 45° to the span.

The beams without notches were reinforced with self tapping screws, applied from the tension edge under an angle of 45° (Fig. 1). 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. 2). These sheets were also glued using an epoxy under an angle of 45° to the beam.

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

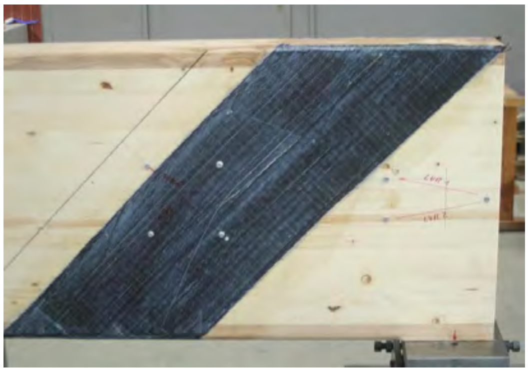

Fig. 2: Beam reinforced with Sika unidirectional CFRP sheets

2.3 Test methods

All beams were tested under three – 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.

3. Results and discussion

For the results of the tests with notched beams it should be referred to a different paper: Jockwer et al: Structural behavior of glulam beams with notches at the support or with holes. The preliminary results of the tests with the beams containing the missglued middle lamella are summarized in the following table 1.

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 $\sigma^*_{c,90}$ are indicated 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. The relevant stresses were calculated as follows:

$\tau^* = 1.5 \cdot 0.5\ F_{\max}/(b \cdot h)$

$\sigma_b = M_{\max}/W$

$\sigma^*_{c,90} = 0.5\ F_{\max}/(a \cdot b)$

Table 1: Preliminary results from tests with reinforced glulam beams

Beam No.	Reinforcement	Failure Mode	$F_{\max}$ kN	τ^* N/mm²	σ_b N/mm²	$\sigma^*_{c,90}$ N/mm²
1a	None	shear	160	1.43	10.8	3.57
	1 x 1 CFRP	shear	218	1.95	14.7	4.87
	2 x 1 CFRP	compr. perp.	303	2.71	20.5	6.76
1b	None	shear	240	2.14	16.2	5.36
	1 x 2 CFRP	shear	251	2.24	20.0	5.60
	2 x 2 CFRP	compr. perp.	436	3.89	29.5	9.73
2a	None	(delaminated)	0	0	0	0
	1 x 4 SFS 13	shear	220	1.96	14.9	4.91
	2 x 4 SFS 13	compr. perp.	361	3.22	24.4	8.06
2b	None	shear	180	1.61	12.2	4.02
	1 x 4 SFS 13	shear	244	2.18	16.5	5.45
	2 x 4 SFS 13	compr. perp.	290	2.59	19.6	6.47

Table 1: Preliminary results from tests with reinforced glulam beams (continued)

Beam No.	Reinforcement	Failure Mode	$F_{\max}$ kN	τ^* N/mm^2	σ_b N/mm^2	$\sigma^*_{c,90}$ N/mm^2
3a	None	shear	200	1.79	13.5	4.46
	1 x 2 SFS 13	shear	240	2.14	16.2	5.36
	2 x 2 SFS 13	bending	372	3.32	25.1	8.30
3b	None	shear	235	2.10	15.9	5.25
	1 x 2 SFS 13	shear	329	2.94	22.2	7.34
	2 x 2 SFS 13	shear	398	3.55	26.9	8.88
4a	None	shear	208	1.86	14.1	4.64
	1 x 1 SFS 13	shear	290	2.59	19.6	6.47
	2 x 2 SFS 13	shear	376	3.36	25.4	8.39
4b	None	shear	196	1.75	13.2	4.38
	1 x 1 SFS 13	shear	245	2.19	16.6	5.47
	2 x 2 SFS 13	bending	360	3.21	24.3	8.04

From the table it can be seen that 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 all 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.

4. 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.

5. Acknowledgements

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Influence of Varying Material Properties on the Load Bearing Capacity of Glued Laminated Timber

Gerhard Fink, Andrea Frangi[1], Jochen Koehler[2]

Summary

In the current project the influence of varying material properties on the load bearing capacity of glued laminated timber (glulam) is analysed. Therefore, the material properties of board sections containing knots and without knots are analysed experimentally. Based on the results models are developed which describe 1) the between and within member variability of timber boards and 2) the interrelation between material properties and easy measurable indicators; such as knot dimensions or eigenfrequency. Further, glulam beams with well-known local material properties will be produced. On those the influence of selected configurations of knot clusters is analysed. Based on the results a probabilistic model for the estimation of the characteristic value of the load bearing capacity will be developed.

1. Introduction

Timber is a natural grown material. Thus, compared to other building materials, timber properties demonstrate higher variability. Due to the highly inhomogeneous structure of timber, this variability is pronounced not only between different structural elements but also within single elements. The variation between elements results from different growth conditions and the cutting process. The within member variation is highly related to knots and knot clusters. In Nordic spruce timber specimens knot clusters are distributed over the length of the board with rather regular longitudinal distances. Considering the trunk of a tree the average distance between the clusters is directly related to the yearly primary growth of the tree.

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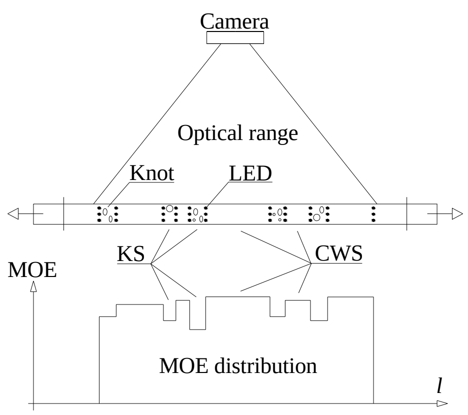
Within glulam the variation is slightly reduced through homogenisation. However, because of the relatively regular distance between knot clusters the joint appearance of knot clusters from different lamellas in the same cross section is relatively frequent. Glulam beams with a critical coincidence of weak sections in highly stressed areas show relatively low load bearing capacity.

2. Material Properties of Glulam Lamellas

2.1 Experimental Investigations

The investigation takes place on lamellas of two grade classes L25 and L40 (Norway spruce, 200 specimens each grade class, dimension: 126 x 44 x 4000 mm). Lamellas of this grade classes fulfil the requirements for the production of the glulam grades GL24h and GL36h, respectively [1, 2]. The grading of the lamellas is performed by the GoldenEye 706 grading device. From all timber boards the dimensions and the position of every knot with a diameter larger than 10mm is assessed and recorded. Additional, several indicators to predict the mean stiffness (eigenfrequency, ultrasonic runtime and density) and the moisture content are measured.

On 100 specimens, of each strength grade, nondestructive tension tests are performed to estimate the stiffness properties within a board by using an infrared camera system. Prior to the experiment every board is subdivided into two types of sections: Knot sections (KS) and clear wood sections (CWS). KS representing sections containing knot clusters or single major knots and CWS representing sections between the KS. Knots with a diameter less than 10mm are neglected. At the beginning and at the end of a KS three high frequent infrared light emitting diodes (LEDs) are mounted (Fig. 6 right).



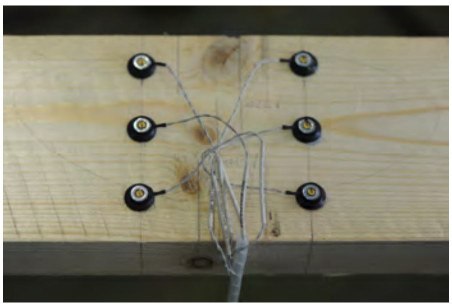


Fig. 5 Left: Schematic illustration of the test set-up. Right: LED arrangement around a KS.

In addition to the LEDs around KS three LEDs are fixed at the beginning and the end of the total measured area, respectively. The boards are clamped in a tension machine and loaded with an axial tension force. During the tensile test the LEDs send light impulses with a frequency of $20Hz$. The position of the LEDs during the tensile test is measured with an infrared camera. Fig. 6 left illustrates the test procedure. Based on the results the relative displacements between the LEDs are computed and the strains of the board sections (KS and CWS) are estimated. In order to ensure comparability of the test results, all tension tests are performed with standard moisture content according to EN 408 [3]; i.e. equilibrium moisture content of the specimen in standard climate: (20 ± 2)°C and (65 ± 5)% relative humidity.

In the last experimental phase the deformation and failure behaviour of significant knot cluster is analysed. Previously, all specimens are prepared with a speckle pattern (Fig. 6 left).

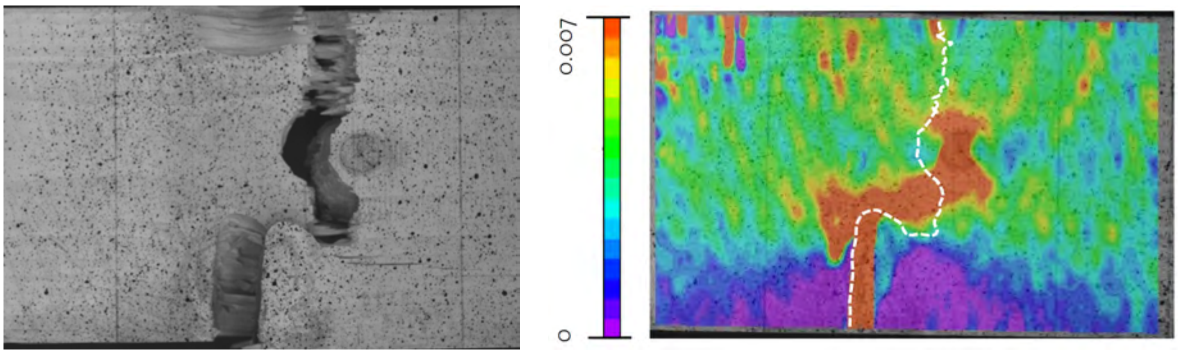


Fig. 6 Left: Fracture pattern. Right: Longitudinal strains before the fracture.

During the tensile test pictures with a frequency of $0.5Hz$ are taken. Based on the pictures, relative displacements within one knot clusters and thus, the strain distribution on the surface are calculated with digital image correlation software. Based on the results the influence of knots and their arrangement on the deformation behaviour will be analysed. Furthermore, an interaction of local strain peaks and the fracture patters will be investigated. The deformation and failure behaviour of knot clusters is analysed with the aim to gain additional knowledge about the influence of knot arrangements. Improved knowledge can be used for a more efficient prediction of the load carrying capacity which is the basic principle of the grading process and for modelling the material behaviour of structural timber products like glulam. For a more detailed description of all experimental investigations see [4].

2.2 Hierarchical Stiffness Model

Based on the results of the experimental analysis a hierarchical model for the variability of the stiffness properties is developed and the statistical model parameters are identified. To simplify the model, KS are modified into sections with constant length of $150mm$, so called week sections (WS). The stiffness of the WS and the stiffness of the CWS are described by two hierarchical levels, respectively [5, 6]: Meso scale and micro scale. The meso scale describes the stiffness variability of a single board within a sample of boards. The micro scale describes the stiffness variability within one board. In Equation 1 the stiffness model for the WS is given (the stiffness model for the CWS is analogue). Furthermore, the correlation between the WS and the CSW within one member is considered in the model. In addition to the stiffness properties the distance between WS is described. Therefore a Gamma distribution is assumed. A more detailed description of the model is given in [7]. With the presented model timber boards with well-known stiffness properties can be generated. Thereby the location of the knot clusters and the stiffness properties of each section are described. A model like this can be used as the basis for a glulam model.

$$MOE_{ij,WS} = \exp\left(\mu_{WS} + \tau_{i,WS} + \varepsilon_{ij,WS} \right) \tag{1}$$

where $MOE_{ij,WS}$ is the MOE of the WS j in a board i. $MOE_{ij,WS}$ is a lognormal distributed random variable.

μ_{WS} is the logarithm mean of all WS within a sample of boards. μ_{WS} is considered to be deterministic.

$\tau_{i,WS}$ is the difference between the logarithm mean of all WS within one board i and μ_{WS}. $\tau_{i,WS}$ is represented by a normal distributed random variable with mean zero and a standard deviation $\sigma_{i,WS}$.

$\varepsilon_{ij,WS}$ is the difference between WS j in a board i and the logarithm mean of all WS within one board i $(\mu_{WS} + \tau_{i,WS})$. $\varepsilon_{ij,WS}$ is represented as a normal distributed random variable with mean zero and a standard deviation $\sigma_{j,WS}$.

2.3 Correlations between Knots, Tensile Strength and Stiffness

Based on the shapes, sizes and arrangements of knots, together with the eigenfrequency, ultrasonic runtime and density a model is developed to describe the stiffness properties of a knot cluster. Therefore several knot parameters (e.g. knot area ratio) are calculated and their influence on the stiffness properties are analysed. Following the most efficient will be identified and combined. Furthermore a model is developed to predict the tensile capacity of timber boards based on the same input parameter. Therefore the data from a parallel research project are used. For a more detailed description of the knot parameters and the models see [8]. The major gaol of the herewith described models is the prediction of material properties based on the results of easy-useable, nondestructive test methods.

3. Load Bearing Capacity of Glulam

3.1 Experimental Investigations

Based on the results of the experimental analysis of the glulam lamellas those are glued together to 24 glulam beams. Hereby the position of each lamella and each finger joint will be defined, previously. Thus, glulam beams with well-known local material properties are produced. KS will be arranged specifically, so that their influence on the deformation and failure behaviour can be analysed. Thereby it is particularly focused on the investigation of 1) the lamination effect and 2) the influence of KS lying upon each other on the load bearing capacity.

3.2 Model to Identify the Characteristic Values of the Load Bearing Capacity

Based on the results of the experimental investigations a probabilistic model will be developed to identify the characteristic value of the load bearing capacity of glulam. Therefore timber boards are generated (based on the hierarchical stiffness model) and glued together. Based on a sufficient amount of simulations the probability of inappropriate configurations such as KS lying upon each other can be identified. From those the load bearing capacity has to be estimated (e.g. with FE-Models). Based on this, the characteristic value of the load bearing capacity can be identified.

4. Summary & Outlook

The described research project is dealing with the influence of varying material properties on the load bearing capacity of glulam. Thereby the following research areas are taken into account:

– Experimental investigation of local material properties of glulam lamellas.
– Development of a hierarchical stiffness model which describes the between and within member variability.
– Development of a model to predict material properties based on the results of easy-useable, nondestructive test methods.
– Experimental investigation of the lamination effect.
– Development of a model to identify the characteristic value of the load bearing capacity of glulam.

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Design of glulam beams with notches at the support

Robert Jockwer[1], René Steiger[2], Andrea Frangi[3]

Summary

The load carrying capacity of glued laminated timber beams is reduced considerably by notches at the support on the tension side of the beam. Stress concentrations at the notch corner together with low shear and perpendicular to the grain tensile strength may lead to brittle failure. For the prediction of the load carrying capacity of notched beams, based on experiments and/or theory several design approaches have been made available each of them using different material properties as part of the model. However, specimen sizes in experimental tests, specifications of material properties used in theoretical approaches and investigations on the influence of ambient conditions on notch capacity are often not in line with the needs of the application in practice.

A safe and practical design approach has to take into account all relevant parameters as e.g. geometrical and material properties, ambient conditions and load characteristics in a reliable way.

1. Introduction

Notches at the support as shown in Fig. 1 can be necessary for architectural or structural reasons. Due to an abrupt change in cross-section stress concentrations occur. The low strength of the timber in shear and tension perpendicular to the grain considerably reduces the load carrying capacity of the beam with

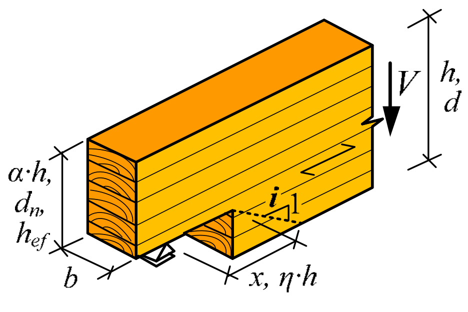

Fig. 1 Notched glulam beam

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notches on the tension side. Since furthermore the failure behaviour is extremely brittle, notches have to be avoided. If this is not possible they have to be reinforced. Within a small geometrical range unreinforced notched beams can be applied in practice, this however being possible only, if the decrease in strength is taken into account. For the design of both reinforced and unreinforced notched beams adequate models are to be used.

2. Design approaches for notched beams

2.1 Empirical design approaches

Scholten [1] proposed one of the first very simple empirical design approaches: the shear capacity of the beam had to be reduced with a ratio equal to the one of the notch α. This design approach is still used and gives a rough estimate of the load carrying capacity of notched beams. Later Mistler [2] found a bilinear reduction of shear capacity to fit the experimental test results and studies based on stochastic failure models best.

2.2 Design approaches based on theory

Design approaches based on theory as listed in Table 1 use fracture mechanics concepts. The stress concentration at the notch corner can be determined by means of linear elastic fracture mechanics. The design approach in AS 1720-1997 (Eq. (1)) takes into account the fracture mechanics size effect by a coefficient g_{40}. It is based on studies by Leicester [3].

Table 1: Selection of approaches for the design of notched beams

AS 1720-1 [4]	FPL Wood Handbook [5]
$\frac{6M^*}{bd_n^2} + \frac{6V^*}{bd_n} \leq \varphi\, g_{40}\, k_1 k_4 k_6 k_{12} f'$ (1)	$\sqrt{h}\left(A\left(\frac{6M}{bh^2}\right) + B\left(\frac{3V}{2bh}\right)\right) = 1$ (2)
EN 1995-1-1:2004 [6]	CSA O86.1 [7]
$\tau_d = \frac{1.5\,V_d}{bh_{ef}} \leq k_v f_{v,d}$ (3) $k_v = \min\left\{1\ ;\ \frac{k_n\left(1 + \frac{1.1\,i^{1.5}}{\sqrt{h}}\right)}{\sqrt{h}\left(\sqrt{\alpha\left(1-\alpha\right)} + 0.8\,\frac{x}{h}\sqrt{\frac{1}{\alpha} - \alpha^2}\right)}\right\}$	$F_r = \varphi F_f A K_N$ (4) $K_N = \left\{0.006d\left(1.6\left(\frac{1}{\alpha} - 1\right) + \eta^2\left(\frac{1}{\alpha^3} - 1\right)\right)\right\}^{-\frac{1}{2}}$

Murphy [8] observed crack opening (Mode 1) and crack shearing (Mode 2) of slits and notches and published a failure criterion based on Stress Intensity Factors (SIF). The design approach according to Eq. (2) in FPL Wood Handbook [5] is based on Murphy's findings and gives conservative strength values A and B for most softwood species.

Gustafsson [9] investigated the energy state of notched beams and proposed a design approach based on fracture energy (Eq. (3)) leading to the reduction factor k_v for shear capacity in EC5 [6]. Smith and Springer [10] suggested to neglect an additional term used by Gustafsson and proposed an approach for notch capacity as given in CSA O86.1 [7] for solid timber (Eq. (4). In the Swiss design code for timber structures SIA 265:2012 [11] in addition to the EC5 approach a simplified version of the EC5 approach is given, valid for quadratic notches.

2.3 Experimental tests on notched beams

Experimental tests have been carried out to verify the design models and to evaluate material property values used in the approaches. In Fig. 2 the specimen sizes used in tests published so far are compared with the range of relevancy of cross-sections used in practice. Most tests have been carried out on small solid timber specimens for reason of simpler tests procedure and cost efficiency.

However, size effects play an important role in the determination of material properties. Not only fracture mechanics and Weibull size effects have to be taken into account but also size effects resulting from different types of grading, way of production and care in both handling and installation.

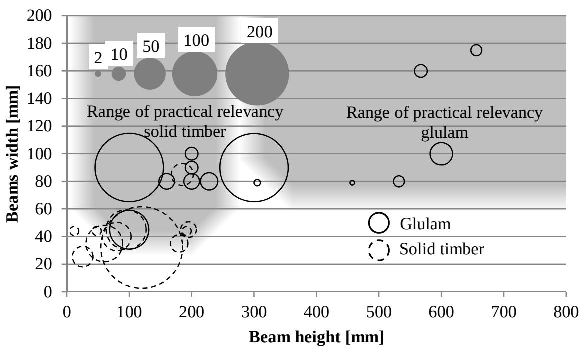

Fig. 2 Experimental tests on notched beams compared to the range of practical relevancy

3. Parameters influencing the notch strength

When applying the design approaches mentioned above precise information on material stiffness and strength properties is needed. Some of these properties are exclusively used in design approaches for notched beams, others are used generally in structural design and can be found in product standards.

Fracture energy $G_{f,\mathrm{I}}$, modulus of elasticity (MOE) and shear modulus are the main strength and stiffness properties used in the EC5 design approach (Eq. (3)). The approach actually was reformulated as a verification of shear strength. The stiffness properties are specified in product standards for different strength classes of solid timber and glulam. The probability density distribution of the stiffness can be estimated from the given mean and characteristic values of the respective strength class by assuming a distribution function as suggested in [12]. The effect of moisture and duration of load on stiffness properties is neglected in EC5.

Fracture energy as a key strength property of the model neither is given in the product standards or the design code nor is a test method specified to derive this parameter way it is used in the design model. For the implementation of Gustafsson's approach into EC5 Larsen et al. [13] assumed a correlation of density and fracture energy $G_{f,\mathrm{I}}$ determined from single edge notched beam tests. With the values specified at that time, the material factor as the product of $G_{f,\mathrm{I}}$ and MOE over the square of shear strength was found to be constant for different strength classes leading to different material constants for glulam and solid timber. Nowadays when basing the calculation on these material constants, the notch strength is overestimated due to changes in the shear strength values in the product standards in recent years. In addition hardly any correlation has been found between density and fracture energy $G_{f,\mathrm{I}}$ within the range of practical relevancy as it was shown by Jockwer et al. [14].

The characteristic value of shear strength given in the product standards has run through some changes in recent years. Though shear strength has no direct impact on the notch capacity it has one on the estimated load carrying capacity of the notched beams calculated with the design approach in EC5. Thus adequate reliability of the design is not given in any case.

4. Need for research

The impact of the material properties on the estimated notch capacity and on the reliability of the design are to be evaluated. In order to increase the reliability of the design of notched beams, a better estimate of the key material properties should be made and parameters affecting these properties should be checked in detail. Further influences on the load carrying capacity of notched beams are to be determined as e.g. the influence of moisture. In design codes regulations regarding notch geometries and ambient conditions are to be formulated. Geometrical limits where notches should be avoided in practice for sake of low reliability are to be defined. Design approaches finally implemented in design codes should be free of material property values not directly connected to notch strength.

The size of specimens for experimental tests should be chosen according to the range of practical relevancy as can be seen in Fig. 2.

Possibilities for the enhancement of strength of notched beams are to be evaluated. Reinforcing notches is of special interest, since it helps in getting reliable structures without arguing the correctness of neither notch design models nor material properties.

5. Methods and Outlook

Different techniques can be used to analyse the impact of different parameters influencing the load carrying capacity of notched beams. The impact of varying material properties can be determined by means of a sensitivity analysis and the theoretical approach itself can be evaluated with a Finite Element model assuming a predefined crack path. The variation of the parameters can be accounted for by using Monte Carlo simulated material properties in the FE-model. Both theoretical and FE-models can be benchmarked to experimental tests. Differences between predicted and measured values can be taken into account by a model uncertainty, possibly expressed by a bias with a certain probability density function. Effects neither taken into account by the theoretical model nor by experimental tests are to be considered in a more sophisticated FE model. E.g. the influence of variation in moisture can be studied in a 3-dimensional FE model. In combination with a material model allowing for single element failure, the effect of varying climate conditions on the progressive failure of notched beams can be studied.

Reinforcing notches is another way of providing adequate reliability of notched members. The interaction of the resistance provided by the wood itself and the reinforcement can be studied by means of FE models and the structural behaviour of reinforced beams finally can be described by an analytical approach. Parameters with key impact on the load carrying capacity of reinforced notched beams can be identified and the reinforcement itself can be optimized.

6. Acknowledgement

The support by the Swiss National Science Foundation (SNSF) is gratefully acknowledged.

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Experimental Investigation on Full-Scale Single Large-Dowel Connections

Peter Kobel[1], Roberto Crocetti[2], Andrea Frangi[3]

Summary

In the frame of a Master Thesis at Lund University a full-scale test series was carried out to investigate the behaviour of reinforced single large-dowel connections. Fifteen spruce glulam specimens were tested in load controlled tensile tests. Besides three different configurations of self-tapping screws also lateral prestressing was used as a reinforcement measure. It was found that the load-carrying capacity of the connection was significantly increased by applying self-tapping screws. Reinforcing screws effectively impeded splitting of the timber if placed near the loaded end of the connection, where major lateral deformations occurred. However, the screws could not prevent the formation of a shear plug, which was observed to be the ultimate failure mode. Lateral prestressing of the connection also proved to be an effective reinforcement measure, as splitting was prevented completely. By applying large lateral prestresses the failure mode changed to a combined shear and tensile failure which resulted in higher load-carrying capacities.

1. Introduction

For spans of more than about 30m, usually truss structures are adopted. However, the competitiveness of large span timber truss structures is reduced due to the expansive and complex joints that have to be adopted. Normally, such joints consist of single or multiple slotted-in steel plates in combination with a number of dowels, which imply a rather complex manufacturing process and also the use of a large amount of steel.

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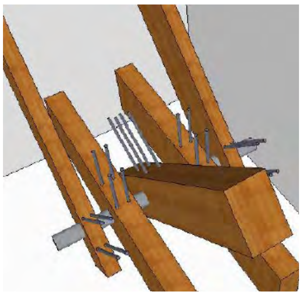

Fig. 1 Principle of the truss joint

As a consequence, truss structures in timber are often not able to compete economically with solutions in other materials (e.g. steel) for large spans. In order to find a more efficient and economical solution for timber joints, a truss joint consisting of a single large-diameter dowel has been developed [1]. As the minimal end distance of $a_3 = 7d$ suggested by EC5 becomes unrealistic for large-diameter dowels, the end distance was reduced to half this value, i.e. $a_3 = 3.5d$. This, however, increases the risk of splitting of the timber along the grain, which makes the use of lateral reinforcements crucial. The study included a full-scale test series to investigate the overall behaviour of the joint under tensile loading, including load-carrying capacity, stiffness values and the determination of failure modes [2]. As premature splitting is a key aspect, different reinforcing measures for the connection were investigated, i.e. three different configurations of self-tapping screws as well as lateral prestressing.

2. Material and Method

To investigate the behaviour of reinforced single large-dowel timber joints full-scale laboratory tests were carried out at the SP Trätek in Borås, Sweden. Fifteen spruce glulam specimens (GL30c) with a cross section of 140mm×405mm and a length of 2.3 m were tested in load controlled tensile tests.

Fig. 2 Test setup

As a consequence, truss structures in timber are often not able to compete economically with solutions in other materials (e.g. steel) for large spans. In order to find a more efficient and economical solution for timber joints, a truss joint consisting of a single large-diameter dowel has been developed [1]. As the minimal end distance of $a_3 = 7d$ suggested by EC5 becomes unrealistic for large-diameter. Local densities obtained from the failure area after testing varied in the range of $\rho = 356 \div 566$ kg/m^3, the corresponding moisture contents were within MC = $8.9 \div 12.4\%$. For all experiments the load was applied parallel to the grain. The employed dowels were steel tubes with an outer diameter of 90 mm and a wall thickness of 30 mm. The dowels could be considered as rigid.

All fifteen specimens were each equipped with an identical design on both ends, meaning that one specimen contained two connections. The specimens were then divided into five groups of three specimens i.e. six connections per group. In four of the groups, the connections were reinforced by a different strengthening method, to avoid splitting along the grain, see Figure 3. Group “Basic” was the non-reinforced reference group.

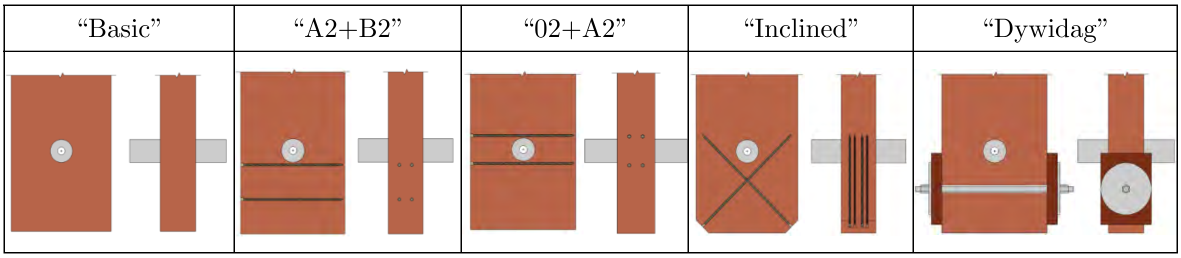

Fig. 3 Test specimens

The connections in groups “A2+B2”, “02+A2” and “Inclined” were reinforced with self-tapping screws (Konstrux 10×400 mm and SFS 9×500 mm). In group “Dywidag” the timber was prestressed perpendicular to the grain using Dywidag rods. Hardwood plates were used to distribute the prestressing force uniformly over the section between the dowel and the loaded end, resulting in lateral prestresses of approximately 3.1MPa. The tests were carried out as load controlled quasi-static tensile tests. The load was applied by a hydraulic jack according to the loading procedure after European Standard EN26891 for timber structures. The load was increased until failure was reached in one end of the specimen. During testing the dowel slip in the connection as well as lateral deformations in the specimen were continuously measured. The lateral deformations were measured in two different positions between the dowel and the loaded end. One sensor was installed 50mm from the dowel, the other sensor was placed at the loaded end, see Figure 2. The lateral deformations were measured from edge to edge, i.e. the recorded values were the total deformations occurring over the height of the specimen. To continuously measure the prestressing force in group “Dywidag”, load cells were installed.

3. Results

Table 1 summarizes the main test results. It should be noted that due to the small sample size of six connections per group the results do not provide statistically reliable values, but they allow for a first comparison of the tested configurations.

As shown in Table 1 the load-carrying capacities of the connections were significantly increased by the applied reinforcement measures. The gains compared to non-reinforced connections ("Basic") were +77% for "A2+B2", +40% for "02+A2", +71% for "Inclined" and +128% for "Dywidag". The ultimate failure mode was a shear plug for all groups and failure. The width of the shear plug was usually slightly smaller than the diameter of the dowel (Figure 4).

Tab. 1 Summary of test results (load-carrying capacity F_{max}, coefficient of variation CoV of F_{max}, dowel slip $\delta(F_{max})$, stiffness in load direction k_{slip}, splitting load F_{split}, lateral strains ε_{split} at F_{split}, lateral stiffness k_{lat}, density ρ and moisture content MC)

Group (n=6)	**Parallel to the grain**				**Perpendicular to the grain**				
	F_{max} [kN]	**CoV [%]**	**(F_{max}) [mm]**	**k^{slip} [kN/mm]**	**F_{split} [kN]**	**ε_{split} [‰]**	**k^{lat} [kN/mm]**	**[kg/m³]**	**MC [%]**
"Basic"	134	4.1	0.48	308	105	2.76	93	523	12.1
"A2+B2"	237	8.4	1.06	336	199	4.05	115	463	11.6
02+A2"	187	8.9	1.24	360	168	4.39	95	473	10.9
"Inclined"	229	8.6	1.28	336	204	4.12	121	400	11.2
"Dywidag"	306	9.4	1.56	311	-	-	100	392	9.2

An exception from this rule was group "Dywidag", where the shear plug was usually wider than the dowel diameter, implying also a change of the failure mode from shear failure to combined shear and tensile failure (Figure 5).

Fig. 4 Typical shear plug failure

Fig. 5 Combined shear and tensile failure in prestressed

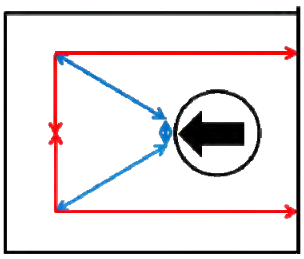

Fig. 6 Qualitative strut-and-tie model

As for the stiffness values k^{slip} a slight increase of 9÷17% was observed for the groups reinforced with screws ("A2+B2", "02+A2" and "Inclined") compared to the non-reinforced group. The prestressed connections in group "Dywidag" did not show any significant increase in stiffness. However, the dowel slip at failure was the largest in group "Dywidag" (F_{max} = 1.56 mm), which was more than three times the slip in group "Basic" (F_{max} = 0.48 mm). Groups reinforced with screws also showed values of more than twice the slip in non-reinforced specimens at failure. Table 1 also shows the mean values of the load when splitting occurred (F_{split}) and the corresponding lateral strains ε_{split}. The groups which involve reinforcement screws placed towards the end ("A2+B2", but also "Inclined") showed higher values of F_{split} compared to group "Basic" without reinforcement, but also compared to group "02+A2" with reinforcements only near the dowel. In the prestressed specimens of group "Dywidag" no splitting was observed. The indicated lateral stiffness k^{lat} connects the load applied to the dowel with the lateral deformations measured at the loaded end of the connection. It can be seen that the groups with reinforcement screws towards the end ("A2+B2", "Inclined") showed significantly higher values for k^{lat} than groups without screws in that area ("Basic","02+A2","Dywidag"). As opposed to the significant lateral strains near the end grain, almost no deformation was measured near the dowel.

4. Analysis

The observation that significant lateral expansion occurred at the end grain and almost no expansion or even compression was measured near the dowel can be explained by using a simple strut-and-tie model (Figure 6) to describe the stress distribution within the connection. Consistently, it was observed that splitting had its origin at the loaded end of the connection. To increase the load-carrying capacity of the connection, splitting has to be prevented, as splits constitute weaknesses in the material which promote the formation of a shear plug. Accordingly, reinforcing screws are most effective if placed near the end, which is confirmed by the fact that the highest load-carrying capacity for connections reinforced with screws was obtained for group "A2+B2". The influence of the reinforcing screws is illustrated in Figure 7, where the splitting behaviour of the reinforced connections of group "A2+B2" are compared with the non-reinforced connections of group "Basic". It can be seen that splitting in the reinforced connections occurred at a higher load F_{split}, but also at a higher lateral strain ε_{split}. This is due to two effects: Firstly, the lateral stiffness k^{lat} is enhanced by the screws, leading to lower strains ε at a given load F. Secondly, the screws even out the natural scatter in the material properties of the timber. Thus, local strain peaks are flattened and a larger average strain ε can be reached before splitting occurs. Furthermore, the screws provide lateral strength in the material even after splitting, restricting the crack propagation. The largest increase in bearing capacity however was achieved by lateral prestressing of the timber as in group "Dywidag". Prestressing at approximately 3.1 MPa ensured that no lateral tensile stresses occurred during the whole procedure of testing. During prestressing the timber was compressed to a strain of around 3.5%, whereas the expansion during testing reached values of around 1%. Thus, splitting of the timber could be completely prevented. Prestressing also increased the shear strength of the material along the grain, enhancing the resistance against shear plug failure. This led to a change in the failure mode, from a pure shear plug to a combined shear and tensile failure (Figure 5). The observed failure modes in this study were brittle, due to the fact that load controlled tests were performed. However, similar *displacement controlled* tests performed on small-scale specimens have shown significant ductility of this type of reinforced connections [1].

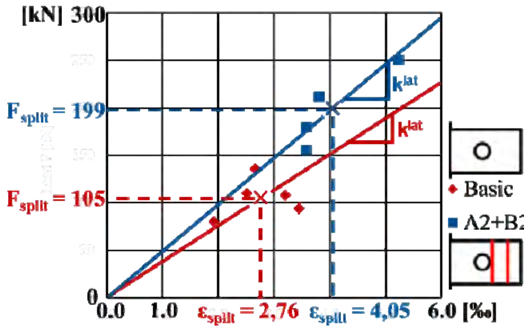

Fig. 7 Comparison of splitting behaviour of groups "Basic" and "A2+B2"

5. Discussion

From the results of the conducted full-scale test series for single large-dowel connections the following conclusions can be drawn:

1. Major lateral strains occur at the loaded end of the connection and thus splitting is initiated at the end grain.
2. Self-tapping reinforcement screws inserted perpendicularly to the grain impede splitting and can thus significantly enhance the load-carrying capacity. The screws are most effective if placed near the end grain, where the main lateral deformations occur.
3. Reinforcing screws cannot prevent the formation of a shear plug, which is the ultimate failure mode.
4. Lateral prestressing is suitable for preventing splitting and enhancing the load-carrying capacity of a connection.

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Thin Structural Toppings for the Upgrade of Existing Timber Floors

Jonathan Skinner[1], Richard Harris, Kevin Paine, Pete Walker[2], Julie Bregulla[3]

Summary

Timber-concrete composites (TCC) are underutilised in the UK. Research at the University of Bath aims to counter this by focusing on upgrading existing timber floors with thin toppings. Stiffness rather than strength has been identified as the performance criteria most in need of upgrade. In particular timber floors often suffer from poor vibration and acoustic performance and although the addition of a topping has been shown to provide a solution, [1], it has also led to a detrimental reduction in vibration performance [2, 3]. This paper demonstrates that if an appropriate topping thickness is identified and the influence of the connector behaviour on the stiffness and damping properties of the composite are understood then TCC's can indeed be used to upgrade a timber floor to improve vibration response.

1. Introduction

In the UK there is a trend to living in older properties requiring upgrade and refurbishment. Despite this, refurbishment of existing buildings is not as well supported with designed solutions as new build. This is because modern expectations are for buildings which perform at a higher standard than in the past. In particular, apartments require acoustic separation and excessive vibration is unacceptable. For building reuse, floors below modern standards require upgrade to stiffen, reducing excessive vibration, and provide acoustic separation. To this end a TCC solution, with a novel thin topping is being developed at the University of Bath. Previous TCC solutions have utilised relatively thick toppings (40-80 mm) but thinner toppings have notable advantages. The change in finish floor level to

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ceiling height is minimised, propping of the floor can be avoided and as will be demonstrated, a significant improvement in serviceability performance can be realised.

The human perception of a vibration is influenced by its frequency, acceleration and duration. Vibrations of high frequency, low acceleration and short duration are regarded as least perceptible. These attributes are determined by the mass, stiffness and damping properties of the vibrating structure. The primary focus of the research project is to investigate how the upgrade of a timber floor can be optimised through altering the floor's mass, stiffness and damping properties to realise the best vibration response. For a TCC it will be shown that these properties are largely determined by the thickness of the topping and the type and spacing of the connectors.

2. Thin Topping

Conventional wisdom suggests that the vibration performance of a timber floor is easily improved by adding a very thick topping to minimise the accelerations of transient vibrations. However it seems unnecessary to add a large depth of material as the mass of even a thin topping (15 mm) is greater than that of a typical UK timber floor. To better rationalise the topping thickness an alternative approach has been developed.

2.1 Optimising the topping thickness

A method based on improving the frequency of the first mode of vibration considers the frequency of a vibrating beam, governed by the relationship expressed in Equation 1, where EI_l is the stiffness of the beam and m is the mass per unit area.

$$f_1 \propto \sqrt{\frac{EI_l}{m}} \tag{1}$$

When a topping is added to the timber beam both the stiffness and mass of the beam change, causing the composite beam to vibrate at a different frequency to the timber beam. The relationship between, the change in stiffness divided by the change in mass, from timber to composite beam is plotted against topping thickness in Figure 1. Gamma, γ, the shear bond coefficient, describes the extent of

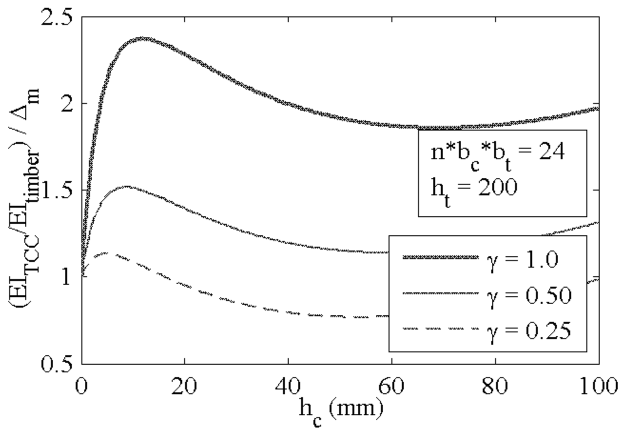

Fig. 1 Relationship between the change in stiffness and mass of a TCC T-beam and topping depth

composite action, n is the modular ratio and b_c, h_c, b_t, and h_t are the dimensions of the concrete and timber elements of the T-beam. As Figure 1 illustrates, for a beam from a typical UK floor, the improvement in frequency is greatest for a topping thickness between 10 and 15 mm.

The only benefit of additional material is to reduce the magnitude of the accelerations. Of the assumptions made in the analysis, γ has the greatest effect. Achieving reasonable composite action is key to a successful upgrade; utilising the beneficial stiffness of the topping and thus mitigating against the negative effect of its mass.

2.2 Acoustic performance of thin toppings

A thin topping is unlikely to provide sufficient acoustic separation for a satisfactory upgrade but it could be used in conjunction with a proprietary acoustic flooring product to achieve better overall performance. A proprietary acoustic flooring product alone would be insufficient as the additional mass of the product would be detrimental to the vibration response of the floor.

3. Connectors

Increasing the frequency at which a floor vibrates is not the only means of reducing the perception of a vibration, shortening the duration is also effective. The duration is determined by the ability of the structure to dissipate energy. One of the additional mechanisms to dissipate energy, available in a TCC floor but not a timber floor, is the connectors between joist and topping. Chui, [4], previously proposed that fixings in timber floors could be designed to provide mechanisms for energy dissipation but the damping available in these types of joints has never been quantified. To determine the damping behaviour of TCC connectors experimentally, the test configuration for a standard pushout test has been adopted and a new loading-protocol devised.

Tests are conducted by repeatedly cycling the load about a non-zero point to replicate the type of loading cycle a connector might experience due to footfall induced vibration. From Figure 2 it is observed that there is hysteretic behaviour and that connectors in TCC floors dissipate significant quantities of energy. To compare how different connector types, loading ranges and frequencies affect the damping behaviour, the energy dissipated can be expressed as a proportion of the total energy in the system. Types of connectors being investigated are illustrated in Figure 3 and have been chosen because of their ability to maintain a mechanical connection in a thin topping.

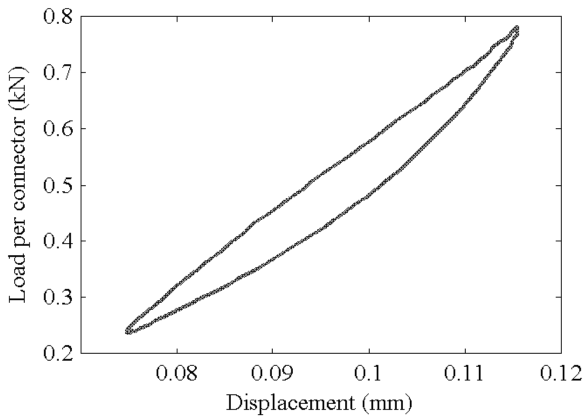

Fig. 2 Hysteretic behaviour of an inclined screw in a thin topping.

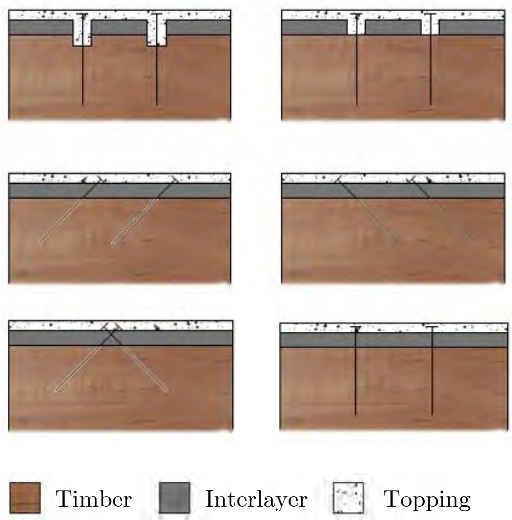

Fig. 3 Thin topping connector types.

4. Damping in TCC floors

The connectors in a TCC floor are not the only mechanisms by which energy dissipation, in addition to that of a timber floor, takes place. Energy is also dissipated through friction between the interlayer and topping as the components slip past each other due to the changing curvature of the vibrating floor. The third mechanism is by the internal material damping of the topping.

Although the damping mechanisms of a timber floor are complex and variable, the additional mechanisms of energy dissipation available in a TCC floor are easily identifiable; providing the opportunity to construct a model to predict the total change in energy dissipation of a TCC T-beam due to an upgrade.

4.1 Damping model

The effect that the topping material damping has on a vibrating beam is simple to implement in a damping model whilst the remaining mechanisms require the slip at the interface to be known along the length of the beam. The slip, w, experienced as a beam vibrates is a function of the strain difference, e, at the interface of the cross-section (Figure 4) and is defined by Equation 2. The slip is also described by Equation 3 where q is the shear force at the interface per unit length and s and K are the spacing and slip moduli of the connectors respectively. Figure 5 demonstrates that the slip along the length of a beam subjected to a span/2000 deflection is small.

By knowing the slip along the beam, the energy lost through friction at the interface and the connectors joining the components can be summated by integration. Equation 4 describes the portion of energy lost at the interface by friction. By assuming the coefficient of friction between the topping and interlayer is 0.2, the magnitude of the energy dissipated by friction between the surfaces is compared with that of the connectors. It is found that for thin toppings that as the coefficient of friction and normal force are both small, the energy dissipated by the connectors has a much greater significance than that of the interfacial friction.

$$e = \frac{dw}{dx} = \frac{s}{K}\frac{dq}{dx} \tag{2}$$

$$w = \frac{sq}{K} \tag{3}$$

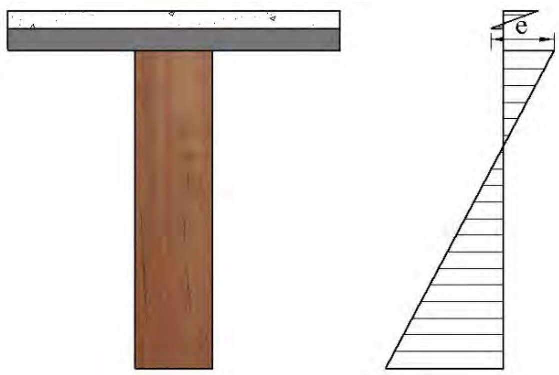

Fig. 4 Strain difference for a TCC section

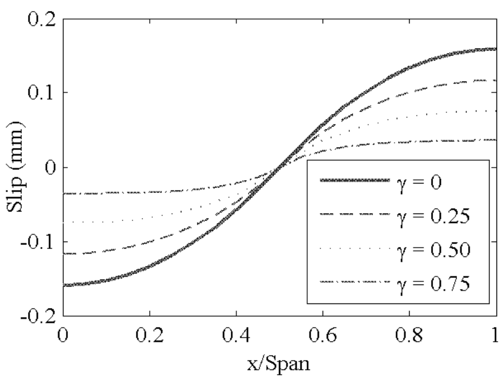

Fig. 5 Slip along a beam when δ=span/2000

$$E_{d,friction} = \mu_k \int F_N\left(w\right)dx \tag{4}$$

5. Conclusions and Future Work

Thin toppings can provide an elegant solution to vibration and acoustic problems of timber floors. Upgrade provides an opportunity to better estimate the damping in TCC floors compared to timber floors. Connectors influence not only the stiffness of TCC floors but also the damping behaviour. To date the majority of TCC vibration testing has been comparing TCC T-beams but the actual vibration performance of a floor can not be discerned from these tests instead, full scale floor tests will be conducted to allow for the effects of load sharing and transverse stiffness.

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Timber Portal Frame Joint using Glued in Rods

James Walker[1], C.S. Chin[2]

Summary

The use of glued in rods as a method for connecting timber has received much attention over the past decade with an increasing number of examples of their use in industry. The objective of this study is to develop guidelines for the design of moment resisting portal frame connections using glued in steel or FRP rods. A series of pull out and bearing tests will be carried out, based on the findings from the LICONS and GIROD projects. These tests will help to gain an understanding of the strength characteristics and failure mechanisms of glued in rod connections. Finite element analysis using ANSYS will be carried out in an effort to numerically model the pull out and bearing tests. A complete finite element model of a portal frame connection will be simulated with the aim of predicted the load and mechanism of failure. A portal frame connection will be tested to destruction and compared to the numerical analysis. The finite element model will be extended to the design of more complex statically indeterminate structures. All results will be compared to the latest guidelines in the Eurocodes.

1. Introduction

With the advancement of manufactured timber such as Glulam and LVL, the design of efficient connection systems are becoming increasingly important. Today's timber engineering industry is demanding more than can be fulfilled by traditional wood connectors such as bolts, dowels, nails or screws. In order to advance the current use of timber as a structural material, a reliable system for connecting members is required. “Connections in timber structures are often the critical part of the design. It is often a balance between what is needed to transfer the loads and what is visually acceptable.” [1]

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Requirements for a suitable timber connection system [2] [3]:

- Strength Equivalent to the timber members being joined.
- Adequate Stiffness to limit deflections to those defined in the Eurocodes.
- Low cost in terms of raw materials, manufacturing and assembly.
- Transfer loads without relying upon contact between members.
- Avoid undesirable behaviour in environments with changing moisture content.
- Avoid significant loss of strength in case of fire.
- Allow for inaccuracies in manufacturing.
- Allow for possible deformation of members during transportation.
- Minimal cutting or drilling of the main members.
- Display a ductile failure.
- Allow for rapid assembly and disassembly.
- Concealment or aesthetically pleasing Joint.

Over the past four decades much research has gone into the manufacture of glued in rod joint connections. Most studies have focused on the strength characteristics of the rod, adhesive and timber interface; attempting to define guidelines for a suitable material for the rod, embedded rod length, bondline thickness and minimum spacing of rods [4] [5]. The most common rods in use are threaded steel or FRP. There are few examples of finite element analysis predicting the behaviour of these joints [6].

With a view on sustainability, engineers are now starting to specify engineered timber products where previously steel beams would have been used. Fig 1 shows an example of an LVL portal frame connection. The connection between members is made using a simple halving joint, glued and bolted together.

Fig. 1 LVL Portal Frame Connection (Courtesy of Webb Yates Engineers)

The aim of this project is to design a series of experiments to demonstrate the failure characteristics of moment resisting joints in timber using glued in rods. The experiments will be modelled numerically using finite element analysis and extended to more complex structures. Finally the design implications of using glued in rod connections will be discussed.

2. Testing

Engineered timber "Kerto-S" LVL will be used for all experimental testing to improve the homogeneity of the timber and provide more consistent mechanical properties. Currently "Timberset" and "RSA" are being investigated to find the most appropriate adhesive to use for testing; both have been used widely in previous studies of glued in rods.

All the specimens constructed for the test campaign will be designed in accordance with the guidelines in the "manual for the design of timber building structures to Eurocode 5" [7]. Testing will be carried out in three stages:

2.1 Pull Out Tests

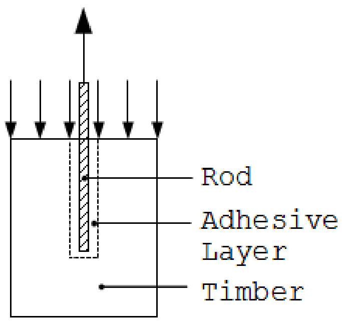

Fig. 2 Pull Out Test Configuration

The first series of tests will be pull out tests using tension in the rod and compression on the surface of the timber, fig 2. Tests will be carried out using both steel and FRP rods, with rods inserted at a variety of angles to the grain (0°, 30°, 45°, 60°, 90°). The advantage of using steel rods is that the connection can be designed so that a ductile failure will take place due to yielding of the steel, rather than a brittle failure in the resin timber interface. FRP rods have advantages over steel in terms of cost, durability and resistance to fire.

2.2 Bearing Tests

The second series of tests will be bearing tests. The portal frame connection will be constructed so that the rods will be fixed to a plate concealed between the horizontal and vertical timber members, as shown in fig 4. This arrangement will improve the bearing capacity of the timber as the rods will distribute the load

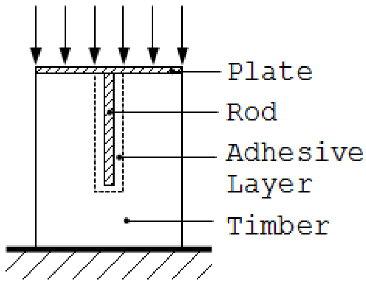

Fig. 3 Bearing Test Configuration

along their length and the plate will provide a greater bearing area. It is therefore important to understand and model this behaviour. The tests will consist of a rod attached to a plate fixed into the timber at a variety of angles to the grain (0°, 30°, 45°, 60°, 90°). A compressive load will be applied until failure fig 3.

2.3 Portal Frame Tests

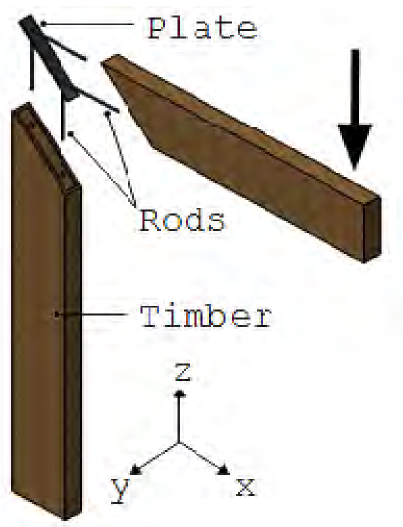

Fig. 4 Exploded Portal Connection

The final tests will be carried out on the portal frame connection. The portal frame connection will be constructed as shown in fig 4. The load will be applied at the end of the horizontal member in both the positive and negative sense. This loading will simulate the positive and negative moment experienced by the connection under self weight loading conditions and situations where wind loading becomes critical.

Following the test campaign, comparisons will be made to the design guidelines given in Eurocode 5.

3. Finite Element Analysis

Finite element analysis will be carried out using ANSYS mechanical APDL. ANSYS has been chosen for its ability to model the orthotropic properties of timber, the non linear fracture mechanics of the adhesive layer and the non linear plastic yielding and large deformation of steel.

The finite element model will be calibrated by attempting to model the extensive testing campaign carried out and reported as part of the LICONS project Task 4.1 [5]. Findings from this study will be used to predict the behaviour of the timber, rod and adhesive connection in both the pull out and bearing tests before experimental testing is carried out. Following the experimental testing, the finite element model will be updated in an effort to improve the accuracy of the predictions. The full portal frame connection will then be modelled to predict the failure loads and mechanisms of the connection before testing. Again, following the experimental testing, the finite element model will be updated with a view to better modelling the observed behaviour from experiment. Finally the finite element model will be extended to model more complex statically indeterminate structures.

4. Conclusions

Currently there are very few conclusions as the research is still at very early stages. The first series of pull out tests are due to take place at the beginning of March 2012. A finite element model is currently being constructed to compare with the experimental testing carried out as part of the LICONS project [5]. The emphasis of this study is to produce guidelines for the design of a portal frame connection using glued in rod technology and compare with current guidelines in Eurocode 5.

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Working Group 2

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

In this area, scientific activities focus on increasing and consolidating the current knowledge of reinforcing techniques used for heavy timber structures, in particular for timber bridges and how to improve/enhance performance and reliability.

This scientific area includes:

- Identification of properties to be enhanced;
- More effective timber decks as a result of effective pre-stressing;
- Increase stiffness and strength by reinforcement;
- Energy dissipation capacity of structures.

Development of High-tech Timber Beam

Karl Rautenstrauch[1], Markus Jahreis, Martin Kaestner, Wolfram Haedicke[2]

Summary

The High-tech Timber Beam (HTB) was developed for economic design of structures with long span and high load-carrying capacity like bridges or large roof-constructions as well as for structural rehabilitation. It is a composite element consisting of timber and high-performance materials. The main material is Gluelam completed with Laminated Veneer Lumber (LVL). The compression zone at the top of the beam is reinforced by an epoxy-based polymer concrete (PC). The tension zone at the bottom of the beam is strengthened by Carbon fiber reinforced plastics (CFRP). Furthermore the bearings with high transverse compression stress and the areas with high shear stress are strengthened with glued in steel- or FRP-bars in combination with epoxy based polymer mortar. The HTB consists mainly of the renewable material wood and the PC can be made with bio-based resins. Therefore it is an ecologically and economically promising alternative to conventional long-span structures based on reinforced or prestressed concrete or structural steel. Accordingly the HTB is particularly interesting for infrastructural buildings in threshold countries or industrial countries with good timber industry.

Fig. 1 Specimen in test after tensile-failure of the wood

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1. Introduction

Timber is the most sustainable building material coeval with a high load capacity. Also it is easily available with a low demand on primary energy. However, wood is a natural material with a spread of properties. To advance the applicability for light weight construction the timber must be new conditioned or reinforced. For the purpose of homogenization the timber is sliced into boards with similar properties and glued together for new beams. The technology of gluing wood has been used with high success for more then 100 years.

Next step to increase the load capacity or the span of a wooden structure requires the use of high performed materials [1]. The demands on those materials are high stiffness, high stress or compression capacity and a good connectivity to timber. With this features it is possible to reinforce the parts of the system with the highest stress. For the case of the reconstruction of historical buildings or damaged structures a polymer resin compounded concrete with high compression stability and a stiffness, which is twice as high as the stiffness of wood, was developed. This PC can be used to reinforce the compression zone of timber beams. The polymer resin has a high bonding affinity to the wood surface, whereby a rigid connection will be achieved. To strengthen the tension zone of the beam, FRP can be used. With a surface mounted FRP-Laminate it is possible to equalize defects as knots and finger joints in material structures and achieve a higher load capacity. CFRP are reliable especially for use with timber structures because of their very low dead weight, the high stiffness and tension strength. Based on these characteristics it is effectively possible to reinforce the structure and create a new high performance composite element.

2. Design and Construction

2.1 Compression and Tension Zone

There is a high compression stress in the upper part of the bending beam. For stabilization a mineral compound can be used, which is bonded by an epoxy resin (see Fig. 2). The resin helps connecting the grid and mounts the Polymer Concrete to the surface of wood. In order to the required properties the PC can be modified by the ratio of the components resin and filling and by the grit size. The characteristic value of the compression strength is about $f_{c,k} = 88.5$ MPa and the Young's Modulus at $E_{mean} = 22.8$ GPa. New epoxy resins are developed on base of phytogenetic oils with high strength and durability. By replacement of the petrochemical resins, used in actual tests, with "bio-based" resins the HTB can be produced with more than 95% of renewable products.

Fig. 2 Compression Zone of the HTB reinforced with Polymer Concrete

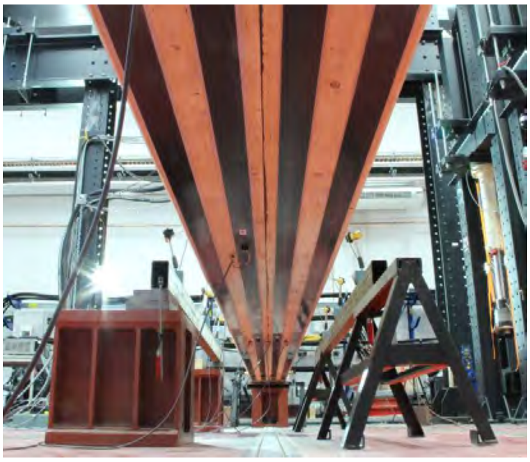

Fig. 3 Tension Zone of the HTB reinforced with CFRP-lamellae

To reinforce the tension zone lamellas of unidirectional CFRP with a width of 50 mm and a thickness of 1.4 mm were used (see Fig. 3). Due to their high MOE of 210 GPa the lamellas can force the timber beam significantly. The most important factor is a stiff connection between the timber and the reinforcing material. Therefore the best solution is surface mounting with sand filled epoxy resins. The glue line moderates the stress distribution and transmits the tension completely.

2.2 Bearings, Anchorage Area and Shear-Reinforcement

Long span and high load on the beam has the consequence of high forces at the bearings. At points where high additional loads are applied, the same problem exists. The forces are transferred transverse to the grain into the timber. This direction of wood has low load capacity and low stiffness. Therefore it is necessary to reinforce these areas. Polymer Concrete, with a high compression resistance, helps to distribute the forces constant to the timber. Additional screws can lead the forces deeper and over a larger area inside of the beam.

Under service conditions the glued on CFRP-lamellas work as a continuously bonded reinforcement. However, due to the extreme difference in stiffness of the bonded materials stress peaks in the ending area are unavoidable, so an effective end-anchorage of the lamellas is needed. Additional, in case of failure the end-anchorage receives particular importance. The process of delamination between CFRP and timber starts after the tensile strength of the wood is reached and the first cracks occurs. Simultaneously the tensile stress in the lamellas increases abruptly. In the worst case the entire tension load has to be carried and transmitted back into the wood by the end-anchorage. The lamellas were mounted over a smooth crown into a bearing block made of PC in investigations. They are completely embedded in the compound over a length of 20 cm (see Fig. 4).

Fig. 4 End-anchorage of the CFRP-lamella / Bearing

With the reinforcement of compression zone and tension zone, the stress at these areas will be amplified. This causes higher shear forces inside of the timber beam. Especially near to the end of the girder, the shear stress increases. This can define the load capacity of the beam, because of the low shear strength of wood.

Glued in rods, made of steel or FRP, can reinforce the timber beam and deflect the force to the bearings.

3. Design concept and numerical Simulation

In case of a direct stiff connection it is acceptable to calculate with the overall cross section amalgamate of the different part sections [2]. Even now the different materials can be assessed with their stiffness inside the composite. For the combination the coefficient for the materials will be found by the comparison of the MOE's of the main material and the additional material. Equation (1) gives the calculation of the moment of inertia (MOI) of the total cross section at a reinforced timber beam section.

$$I_{eff} = (I_t + A_t \cdot e_t^2) \oplus n_{re}(I_{re} + A_{re} \cdot e_{re}^2) \tag{1}$$

with I_t MOI of timber beam section

I_{re} MOI of reinforcing component

$n_{re} = \dfrac{E_t}{E_{re}}$ ratio of MOE of Timber and reinforcing Material

A constitutive numerical model for wood allows the approach of longitudinal shear stress under tension or compression perpendicular to the grain as well as the localization of the area of plasticization. Additionally, the generated three dimensional FE-Modell of the HTB contains all reinforcements with their typical elastic-plastic material properties so that deformation behavior, failure mechanisms and failure load can be calculated reliably. A comparison of the test results, partially achieved with 3d-photogrammetric measurement technology, with the simulation results enables the calibration of the FE-Modell for further parameter studies [3].

4. Tests

Short time bending tests with 8 m long structure elements were arranged for the determination of the load bearing capacity and the deformation performance of the hybrid beams. Tests were carried out as 6-point-bending tests partly according to DIN EN 408. This kind of load set-up induces a distribution of bending moment like under linear load in usual service. During the tests the load as well as deflection, deformation and strain at several points of the beam and at every component were recorded (see Fig. 5).

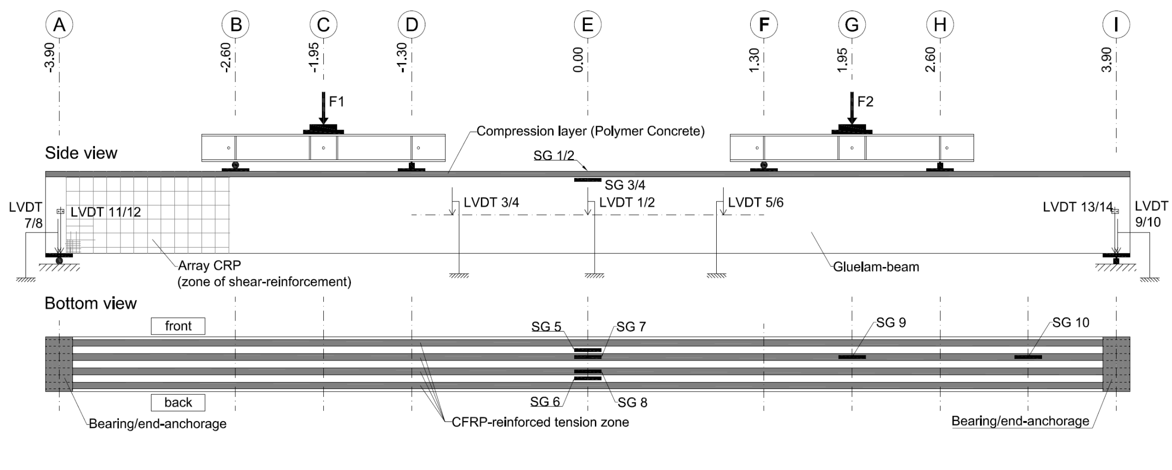

Fig. 5 Six-point bending test of a eight meter long specimen, here: CFRP-reinforced tension zone

As result the stiff connection between the single material layers could be shown and the equation was verified for validation. The hybrid structure element shows an almost linear deflection behavior up to the collapse. The breakdown is caused by tensile-failure of the wood next to the bottom. After the collapse the deflection increases and a residual strength of about a quarter of the main strength remains. The compression zone keeps in solid conditions, while the tensile forces are carried completely by the CFRP-Lamella so that the static system almost corresponds to a girder with carriage strap.

Contactless testing measurement by close-range photogrammetry (CRP) was used to determine the allocation of strain distribution at different specimens during the tests . The CRP-measurement gives the possibility to locate whole range areas of strain. By the connection of CRP and FEM, it is possible to create a realistic model and estimate the load capacity of the system.

5. Discussion, Conclusions and Acknowledgements

The investigations on hybrid timber beams show good results regarding the enhanced load capacity by reinforcements with modern high-capacity materials. Further studies on important details with high loads are necessary. Those are the bearings and points with external load as well as the areas of shear.

Thanks are going to the Bennert GmbH as cooperation partner at the investigation project and the European Union (EFRE) for co-funding.

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Fatigue Behavior of Timber-Concrete Composite Road Bridges

Karl Rautenstrauch[1], Jens Mueller[2]

Summary

The project summary presents approach and first results of a research project that concentrates on the fatigue behavior of a connector used for timber-concrete composite road bridges. In this context the conventional configuration of the shear joint and a modified design with a layer consisting of polymer concrete were investigated. In consequence of a new test setup at first short-time shear tests were accomplished, which had served as basis for later tests under cyclic loading. The content of the here presented topic belongs to FP1004 working group 2, which deals with heavy timber buildings and bridges.

1. Introduction

With the construction of the Birkberg-Bridge near Wippra (Fig. 1) in the German federal state Saxony-Anhalt the first timber-concrete composite (TCC) road bridge in Germany was build and opened for traffic nearly three years ago [1]. Therefore a suitable connector, which is able to transfer the occurring shear forces between the concrete deck and the log-glued laminated timber main girders, has been developed in a previous research project at the Department of Timber and Masonry Engineering of the Bauhaus-University Weimar [2]. The so called stud connector consists of a 3 cm thick steel plate with welded shear studs on the concrete side.

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Fig. 1 Log-glued timber main girder with stud connectors (pilot project Birkberg-Bridge)

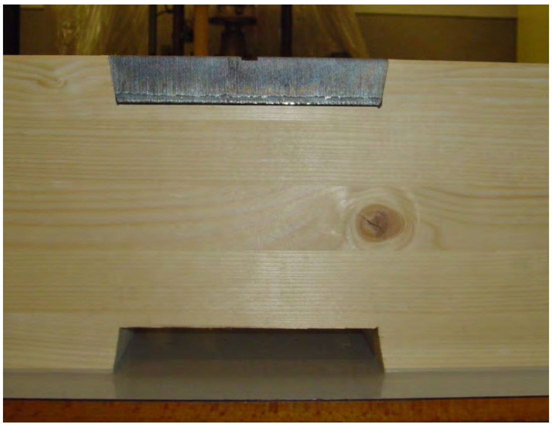


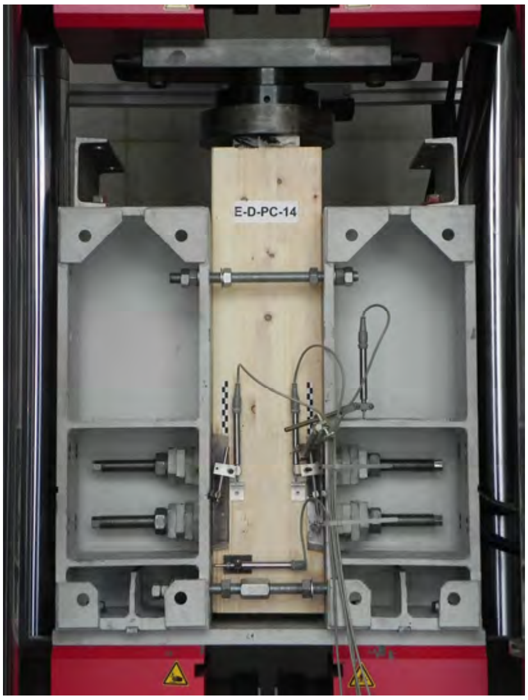
Fig. 2 Conventional design of the joint (series E)

So the bond to concrete follows the accepted principles of steel composite bridges, while the bond to timber uses the principle of form-closure and works like a traditional step joint. Up to now the stud connector has not been tested extensively under cyclic loading, but especially for an application in bridge building it is necessary to know the fatigue behavior of the connection. For this reason the stud connector was tested under cyclic loading and analyzed with the help of numerical simulations to get a reliable evaluation within this project.

2. Experimental tests

2.1 Short-time shear tests

First of all, eleven short-time push-out tests have been conducted according to DIN EN 26891:1991. In addition to the conventional construction of the joint with direct contact between steel and timber in the facing area (series E, Fig. 2) a new kind of design was applied with the arrangement of a layer consisting of polymer resin compounded concrete (PC) between stud connector and timber (series E-PC, Fig. 3). PC is a final product with general technical approval composed of a matrix of mineral additives and a polymeric resin system as binder. Because of its compound PC also offers high compression stability. Compared to the conventional construction this new type of joint possesses essential advantages, because PC equalizes all tolerances due to manufacturing, so that the stud connector can be embed accurately fitting into the milled notch.



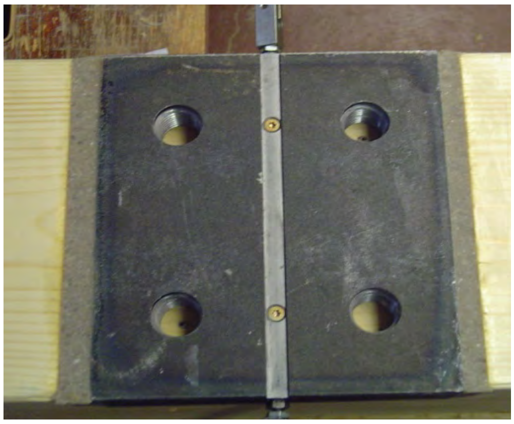


Fig. 3 (above)
Modified design of the joint (series E-PC)

Fig. 4 Test setup and measuring equipment for short-time shear- and fatigue tests

The primal intention of the research project was the investigation of the transmission of the shear force from the stud connector into the timber cross section, both under static and cyclic loading. The transmission of the shear force into the concrete can be evaluated as far as possible with the codes regarding composite constructions. So the test setup was simplified because the concrete was substituted for steel profiles, but the test-principle of the so called push-out test remains. The test setup was fitted into a servo-hydraulic testing facility with a capacity of 1.000 kN, which was used for the tests under static as well as cyclic loading (Fig. 4). For all tests the classical measuring method with dial indicators was used. Additionally a contactless photogrammetric measuring system was applied for the first time for such shear tests. The photogrammetry makes it possible to detect local strains in detail, for example in the facing area about the whole test duration. Then these results serve as basis for following numerical simulations.

The investigations and analysis show explicitly the advantages of the assembly of a layer consisting of polymer concrete in the facing area so far. The initial slip between stud connector and timber, which always occurs in different intensities with the use of the conventional design of the joint, can now be eliminated because of the fitting accuracy. So this effect leads to duplication up to a triplication of the stiffness of the joint in comparison to the specimen with direct contact between connector and timber.

2.2 Fatigue tests

The intention of the fatigue tests are the determination of S-N-lines for both designs of the joint and to get reliable information on the behavior of the connection under cyclic loading. Based on the results of the short-time tests, a total of 24 tests – nine tests for series E and 15 tests for series E-PC – have been arranged under pulsating stress. In this context three different load levels (40, 50 and 60 % of the average value of the ultimate loads of short-time shear tests as maximum load) have been investigated. The minimum number of load cycles a specimen has to achieve to count as a run-out is defined according to practical aspects and based on a literature research with 2 million cycles. The stress resp. load ratio, which is defined as the relation between minimum and maximum load, is arranged with 0.1. Consequently the stress amplitudes, which can be expected in building construction, can be reliably represented. The frequency of the applied pulsating compressive load amounts to 3 hertz. So all boundary conditions together lead to a test duration for a run-out of about ten days. Altogether eight of the 24 fatigue tests arranged so far failed within the cyclic loading. Analogue to the short-time tests the final failure mechanism was always shear failure of the timber in front of the step joint (Fig. 5). Prior to this there is a local compressive failure under an angle to the grain of 80° in the load bearing area as a consequence of the advancing compression of the truncated fibers (timber cells).

Fig. 5 Failure due to shearing of timber in front of the joint

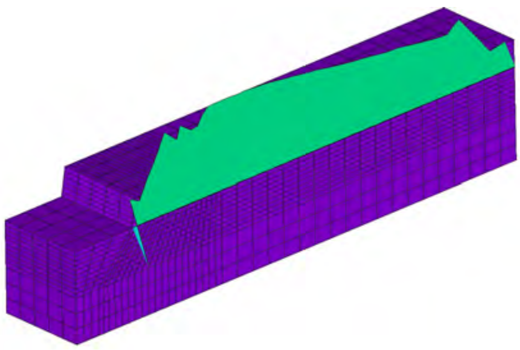


Fig. 6 Shear stress in the area of the length of timber in front of the step joint as a result of the numerical simulation

A very important result of the tests under cyclic loading is the elaboration of S-N-lines for both designs of the shear joint. Based on a regression analysis (Fig. 7), which considers only the failures within the fatigue loading, the fatigue strength of the shear joint can be estimated to nearly 68 % of the corresponding ultimate load of the short-time tests for the specimen with direct contact between steel and timber (series E). For the test pieces with a layer consisting of polymer concrete in the facing area (series E-PC) the fatigue strength was calculated to about 51 % of the respective short-time load. Further information see in [3]. The S-N-lines shown here were calculated with the average values for the results, why the proposals for the fatigue strength, especially for series E, seem to high. For a better evaluation a statistical analysis will be absolutely necessary in the next step regarding to safety reasons. For a better appreciation and classification of the results the graph additionally contains the run-outs of each series and the k_{fat}-line according to DIN EN 1995-2:2010, the Euro–pean code for timber bridges.

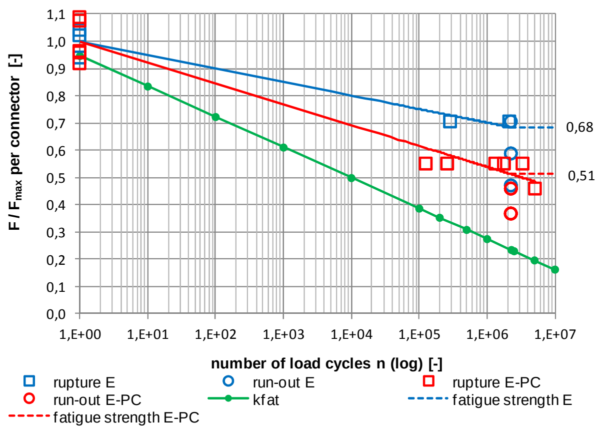


Fig. 7 Proposal for S-N-lines for series E and E-PC in comparison to the k_{fat}-line

As shown in the graph both test series with different designs of the joint observed the requirements of the fatigue verification according to the code with sufficient safety distance.

3. Results, conclusions and further on-going work

The results of the research project demonstrate clearly the advantages of the arrangement of a PC-layer in the load-bearing area, because the use of PC equalizes all tolerances due to manufacturing, makes an accurately fitting assembly possible and causes a very high stiffness of the joint. Furthermore a consistent load distribution over the whole contact area of the notch can be achieved, so that the variances of the experimental results are significant lower, which leads to a higher safety.

Furthermore, a first proposal for the fatigue strength for both designs of the joint can be presented, but at this point more precise information on the basis of a statistical analysis are necessary. The main focus of the further on-going work will be – as shown in Fig. 6 – the numerical simulation to use the extensive results of the short time- and fatigue tests.

4. Acknowledgements

The presented results are elaborated in the line with the research project "Load-bearing-, deformation- and fatigue behavior of special connectors used for timber-concrete composite bridges", which is financed by the German ministry for economy and technology via the AiF in collaboration with the International Association for Technical Issues related to Wood (iVTH). These institutions are gratefully acknowledged.

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Rigid Joints for Large Timber Structures

Kjell A. Malo[1], Haris Stamatopoulos[2]

Summary

The main objective of the project is to develop technology for high stiffness of joints in large timber structures. Emphasis will be on rotational stiffness and moment carrying joints as well as systems for prefabrication allowing easy mounting on the building site. A series of applications will benefit from the new technology. Such applications are rigid beam and post connections for large urban multistory timber structures, cantilevered beams, clamped columns at the footing in large timber buildings, sideway stiffening of arches at the footing of bridges and splicing of large arches and joining wood components to steel or concrete parts. The project will focus on splicing and clamping of glulam members by use of large threaded screws or rods and metallic coupling parts.

1. Introduction

The competitiveness of timber structures is largely governed by the design and effectiveness of its connections. The manufacturing costs and the extent of labor can make timber structures rather expensive and thus exclude them from further consideration in comparison to other building materials. The ease of erection may also be jeopardized by improper design of the connections.

In a connection, if the interfacial zone is softer than the adjacent components, the overall structural stiffness will be severely reduced and additional rotations or displacements will occur at the joint. This often occurs in practical design of wooden structures since usual connections have only minor rotational stiffness, if any at all. Consequently, the use of wood is in many situations hampered by lack of rigid connections. To overcome this limitation separate stiffening structural

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components are usually added, like x-bracing or some type of sheeting. However, the bracing or sheeting put strong restrictions on the exterior appearance as well as on the interior use on the building.

In short, high stiffness of joints is necessary in order to transfer moment action between wooden building components. Too low stiffness will lead to ineffective moment transfer or excessive deformations. Too low overall stiffness in building structures give flexible structures, unacceptable deformations and possibly instability and uncomfortable movements and vibrations.

The approach in this project is the use of large threaded rods and metallic coupling parts. Long self-tapping screws and threaded bars may, to some extent, play the same role in timber structures as reinforcement bars do in concrete structures. These devices offer new opportunities for development of rigid joints in glulam post and beam systems. Threaded bars and long self-tapping screws are connectors that can transfer large forces along the screw axes. Pre-installation in components make them suited for industrialized building concepts.

2. Research Tasks

2.1 Experimental Investigation

The experimental framework of the project consists of the following activities:

- Experimental uniaxial withdrawal tests of single rods. Parameters will be the length of the rod and angle between the rod and the grain.
- Experimental withdrawal tests with parallel rods. Number of rods will be limited. Parameters are number of rods, spacing between the rods and the angle to grains.
- Experiments to study the interaction between several rods installed in parallel. Uniform displacements at the fixation will be imposed and force in the individual rods to be measured. Parameters might be spacing, rod length and angle to grain.
- Experimental testing of moment carrying connection, either using a spliced simply supported beam or a cantilevered beam. Parameters will be number of installed rods and height of beam. A possible additional parameter is angle of rod-axes to grain.

- Experimental testing of moment carrying connection, using either a spliced simply supported beam or a cantilevered beam. Parameters will be number of installed rods and necessary stiffness and strength of additional metallic parts to ensure plastic distribution of forces in the rods and a certain amount of ductility of the joint.

2.2 Numerical Modeling

Numerical modeling will be used for validation of the experimental outcome. Moreover, numerical modeling can give insight about the stress and strain distributions along the rods and surrounding wood, quantities which are difficult if not impossible to measure in the experimental tests. Numerical modeling consists of the two following parts:

- Numerical modeling of threaded rods installed at an angle to the grain in wood. The experimental results obtained by the tests of rods will be used as validation tests of the models.
- Numerical modeling of experimentally tested connections based on numerical modeling of threaded rods and experimental results of connections for validations.

2.3 Analytical Modeling

A further task of the project is the development of analytical models for the two following cases:

- Analytical models for the withdrawal strength of threaded rods installed at an angle to the grain. Models shall cover both stiffness and ultimate strength.
- Simplified analytical design methods suited for design of moment-carrying rigid joints in large timber structures. The model should provide both the stiffness properties and the strength of the connection. The model should be candidate for an ETA approval and/or a proposition to design codes like Eurocode 5 (EC5) [1].

3. Expected Results

Modern timber buildings and bridges are highly engineered products at the end of the value chain. The most important contribution to the knowledge basis for structural use of timber is the development of rigid joints that will make impact primarily on urban multi-storey buildings as well as more specific structures like bridges. With the documentations of the strength, stiffness and the general structural behavior, more and larger timber structures can be made. Rigid joints will give more competitive structures and increased possibilities for further industrialization. In general, it is important that wooden structures, as a renewable environmentally friendly and sustainable structural material, can be a competitive alternative for urban buildings as well as other large structures like bridges.

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Study of behaviour of timber footbridges by static and dynamic load testing

Vanessa Baño[1], Julio Vivas[2], Soledad Rodríguez[3]

Summary

Vibrations and risk of resonance in timber pedestrian bridge are studied in this project. Different structures and timber elements will be evaluated in order to determine the influence of designs typologies in the natural frequencies of structures. Several types of supports in timber beams are being analyzed to determine the influence of that in the natural frequency and the degree of similarity with the expected theoretical behavior. Three boundary conditions are being studied: hanging beams simulating a free-free condition, simply supported beams on neoprene and pinned-pinned supported beams.

Dynamic analyses are being developed in timber bridges in order to know their natural frequencies and their risk of resonance. Currently simply-supported, two hinged arch and truss timber footbridges are being analyzed. Once performing dynamic tests on several bridges will be possible to know their stiffness, and the influence of different designs on vibration of the structure.

1. Introduction

The demand of timber pedestrian bridges in Spain has recently increased with respect other materials like concrete or steel. It is common the realization of in situ testing on metal and/or concrete road bridges to determine the modal parameters of the structures in order to control their vibration. Although dynamic tests in timber bridges are less frequent, there are recent research works on this subject [1][2][3][4]. The lightness of timber bridges, compared to those made of

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other materials, and the trend towards designs with increasing span, means that natural frequencies are lower, resulting in an increased risk of resonance.

Table 1. Frequency range classification according to IAP-11 for vertical vibration

Frequency (Hz)	Range
1st frequency $<$ 1.25	No risk
1.25 - 4.60	Critical risk
1st frequency $>$ 4.60	No risk

The recent review of the regulation of design loads for road bridges (IAP-11) includes a section about the limit state of vibration in footbridges as a serviceability limit state [5]. The IAP defines a range for vertical frequencies (Table 1).

According to the IAP, the natural frequencies of pedestrian bridges must be outside this range (1.25-4.60 Hz) to comply with the serviceability limit state. Furthermore, the IAP requires verification by specific dynamic tests when materials different from concrete or steel, such as timber, are used.

The *Service d´Études techniques des routes et autoroutes* establishes the ranges in which these frequencies are situated to assess the risk of resonance entailed by pedestrian traffic and to can verify the comfort criteria [6]. Table 2 shows the vertical direction frequency corresponding to the risk of resonance classification.

Table 2. Frequency range classification according to SETRA [6].

Frequency (Hz)	0-1	1-1.7	1.7-2.1	2.1-2.6	2.6-5	>5
Range 1						
Range 2						
Range 3						
Range 4						

where, Range 1= maximum risk of resonance, Range 2= medium risk of resonance, Range 3= low risk of resonance for standard loading situations and Range 4= negligible risk of resonance.

This critical range of natural frequencies is based on empirical investigation of step frequencies. Dynamic actions and vibration behaviour of bridges should be considered at an early stage of design, since this calculation is only an indicator of actual behaviour. Dynamic responses should therefore be measured after finishing the construction [7].

The main objective of this project is the evaluation of the frequencies of vibration of timber footbridges using dynamic tests and the analysis of the influence of different design configurations on these frequencies. The respective natural frequencies of vibration of each design will determine the risk of resonance of each bridge because of the transition of pedestrian according to the normative.

Natural frequencies are highly influenced by the boundary conditions of the structure. To know whether the actual behaviour of support corresponds to theoretical expectations is difficult. So other objective of this project is to study the influence of the supports in bridges using timber beams that simulate different boundary conditions.

Finally the stiffness of timber footbridges will be predicted comparing static and dynamic load test.

2. Methodology

2.1 Procedure for dynamic test in bridges and beams

2.1.1 Instrumentation

Six model PCB 3711B112G accelerometers with a sensibility of 1000 mV/g and a measurement range of ± 2 g will be used to determine the accelerations of the bridges and the beams. For the bridge and beams excitation an impulse force test hammer, model PCB 086D20, was used. To register the data of the impact hammer and the accelerations response during the dynamic test mobile measurement system from IMC with eight channel recording will be used which allowed a real time monitoring of the entire data acquisition process. To measure vibrational response and natural frequency it is necessary to attach accelerometers underneath the bridge girders or the beams and to hit on it with an impact hammer.

2.1.2 Test procedure

Acceleration responses generated by the structures during the dynamic test are recorded on a laptop through the IMC Devices software. The configuration of the program is determined according to the characteristics of the structure, based mainly on the adjustment of characteristic values of the accelerometers and the impact hammer (sensitivity, measuring range, number of samples collected during each part of the excitement, etc.). Use of the software allows us to visualize the behaviour of each individual accelerometer, and to confirm that the structure had been excited correctly, through its faithful monitoring of the impact hammer.

2.1.3 Processing of vibration data and analysis of the FRF

The IMC Famos software is used to confirm that all the excitement impacts on the bridges were applied correctly and that there were no anomalies during the recording of responses. Also, this software enables the Frequency Response Function (FRF) to be determined from the Frequency Time Function (FFT). Processing is carried out using LMS Test.Lab software which synthesizes the FRF's of the tested bridges or beams and dynamic responses were evaluated.

2.2 Dynamic test in beams

In order to analyze the behavior of the supports on timber beams, is carrying out three different dynamic tests on the beams under study: a first test for hanging beams simulating a free-free condition (Figure 1), a second test for simply supported beams on neoprene (Figure 2) and finally a third test for pinned-pinned supported beams. (Figure 3).

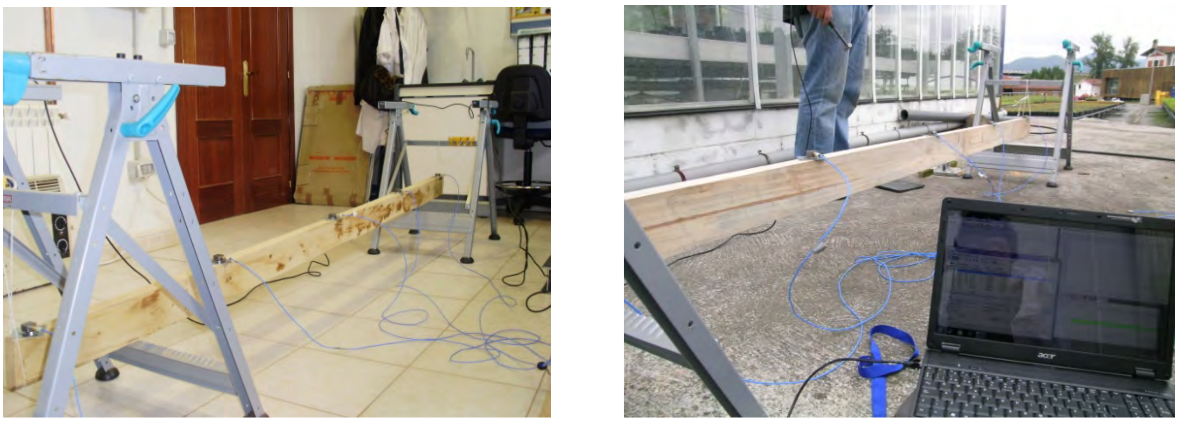

Figure 1. Test for hanging beams (free-free condition).

Figure 2. Test for simply supported beams on neoprene.

Figure 3. Test for pinned-pinned supported beams.

2.3 Dynamic test in bridges

To measure the natural frequency it is necessary to attach accelerometers underneath the bridge girders and to hit on the bridge with an impact hammer. Five accelerometers are placed on the lower edge of the two main girders using magnets and metal plates installed on the bridge.

The tests are conducted in two phases due to the limited number of digital inputs available on the signal acquisition system. In the first phase the accelerometers are placed in positions 1, 2, 3, 4, 5 and 6 of the structure as shown in Figure 4. In the second phase they are placed in the remaining positions: 7, 8, 9 and 10. The points of impact to achieve the excitation of the structure by the impact hammer are 3, 4, 5 and 6.

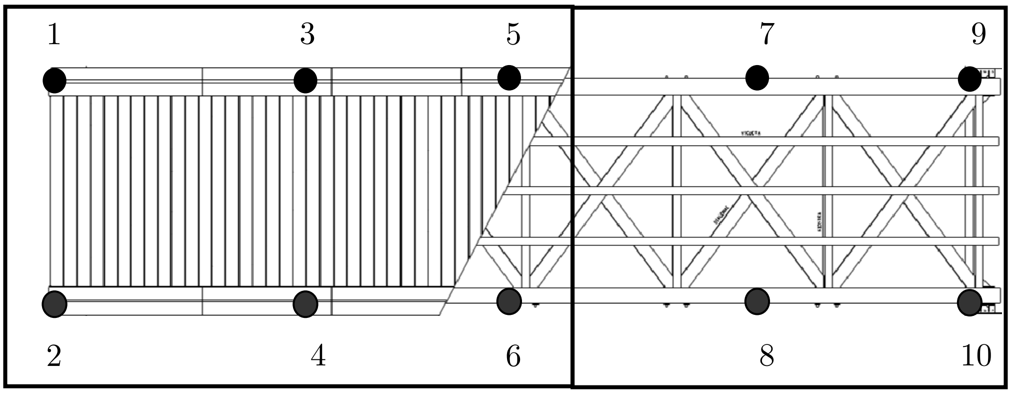

Figure 4. Locations of accelerometers in the two mentioned phases of the test.

This test is carried out in the workshop of the builder of the bridge and then on the place in situ where the structure will be located (Figure 5). Also a methodology based on works developed by Crews et al. [8] on non-destructive evaluation of old timber bridges using dynamic frequency analysis to determine global stiffness is applied in some bridges. This methodology is based on measuring the natural frequencies of the bridges after placing a point charge in the centre of main span. The natural frequency obtained for the bridge unloaded is compared with the new frequency obtained for the bridge loaded, and the flexural stiffness of the structure will be obtained (equation 1):

$$k = \frac{(2\pi)^2 f_1^2 f_2^2}{f_1^2 - f_2^2} \Delta \hat{m}_i \tag{1}$$

Where k is the flexural stiffness of the structure, $\Delta \hat{m}_i$ is the modal mass and f_1^2 and f_2^2 are, respectively, the frequencies of the structure loaded and unloaded.

Figure 5. Dynamic test in two bridges.

3. Results

Table 3 shows the preliminary results for eight bridges without added mass. This value of the first natural frequency of each bridge was compared with theoretical natural frequency calculated and numerical natural frequency obtained from Robot software. Table 3 which also shows the risk of resonance of each one according to the IAP-11 and its range classification is according to SETRA.

Table 3. Preliminary results of first natural frequencies in eight bridges.

	L	w	w_g	h_g	m	H_g	f_{1a}	Resonance risk		f_{1b}	f_{1c}
	[m]	[m]	[mm]	[mm]	[kg]	[%]	[Hz]	IAP-11	SETRA	[Hz]	[Hz]
1	15.6	1.5	185	858	4725	14.50	7.01	No risk	Range 4	5.50	5.58
2	10.6	2	185	561	2442	16.60	8.37	No risk	Range 4	7.21	7.06
3	18.6	2	185	924	6336	14.40	4.88	No risk	Range 3	4.08	4.11
4	9.1	2	185	462	2107	28.90	9.58	No risk	Range 4	7.29	7.45
5	14.6	2	185	726	4191	15.90	5.59	No risk	Range 3	5.02	5.11
6	7.6	1.5	185	462	1601	15.30	13.67	No risk	Range 4	10.94	11.2
7	5.8	1.5	185	330	974	13.70	15.88	No risk	Range 4	12.73	13.9
8	21.6	1.5	190	952	6886	15.10	4.54	Possible risk	Range 3	3.32	3.33

where

L = span of the bridge (m), w = width of the bridge (m), w_g = width of main girder (mm),
h_g = height of main girder (mm), m = mass of bridge (kg), H_g = humidity of beams (%),
f_{1a} = experimental natural frequency of first bending mode,
f_{1b} = theoretical natural frequency of first bending mode calculated by Eq. (1),
f_{1c} = numerical natural frequency of first bending mode calculated by ROBOT,
r = correlation coefficient, range 3 = small risk of resonance, range 4= negligible risk of resonance.

4. Conclusions

Currently are being carried out tests on timber beams to determine the influence of the type of supports and their degree of agreement with theoretical boundary conditions. Furthermore, dynamic tests are still carried out on bridges in order to estimate the influence of different designs in the characteristic frequencies and to know them modal stiffness. Preliminary results of the bridges showed that there was a decreasing relationship between the natural frequency of the first bending mode and the span of the footbridges. A risk of resonance was probable for bridges of longer span when the step of a pedestrian was considered.

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Timber composite structures: literature review and new proposals in the analysis, design and construction.

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Summary

A first research project on timber-concrete composite beams was developed at UPM between 2004 and 2007. The first results are commented, any of them still unpublished.

A new stage in this working line is now beginning.

The annex B and C of the Eurocode-5 [1] includes the termed *Gamma* method for the analysis of timber composite beams and columns, respectively; but the scientific existing knowledge available in the literature is much more complete. An extensive and systematic literature review is currently developed, and it is absolutely needed for updating the professional field.

Work on new analysis tools is also being carried out. Two first proposals have been already implemented, one analytical and other numerical. The use of commercial Finite Element packages (SAP-2000® in our case) offered a very simple a precise analysis method. The possibility of using Elasto-Plastic analysis is being checking using the obtained experimental data.

The international research on prefab solutions is being now very active; but it is still open and it is just one more path for going ahead. The timber composite

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concept can be used in most general way. Researching works around new structural composite types and new connection systems are also being developed, still unpublished.

The implementation of optical measurements methods is also a secondary target of the project.

Two Short Term Scientific Missions (STSM´s) will be carried out inside this COST, with the collaboration of Professor Dr. Ing. Kjell Arne Malo (Norwegian University of Science and Technology, Department of Structural Engineering), and Professor Dr. Ing. Roberto Crocetti (Lund University, Department of Structural Engineering). These STSM´s will impulse this research, and we hope it would be a first step in a standing collaboration.

1. Introduction

The termed *Gamma* method of Eurocode-5 [1] is clearly not enough even at the professional field. It is not able to properly deal with a discrete connections systems except for the case of quasi-regular solutions, and its application out of the simple supported beam offers immediately difficulties [2] [3].

EC-5 has also the drawback of being limited to timber, being e.g. the timber-concrete composite case not explicitly included neither at the concrete nor at the timber Standards.

But the existing scientific literature in this field is huge. Works on the State of Art are now appearing [4], but additional work on the state of art is still needed. And this is a work to be done for the whole scientific community.

By other hand, the problem has been usually faced, especially in the last two decades using a local finite element (FE) modeling; i.e. with a local approach. That makes difficult the transfer of know-how to the professional area, which has contributed in the existing gap between the scientific and the professional ones.

Additionally, there is, in general terms, a lack of parametrical studies, which are crucial to have a real insight of the problem. These studies will be also very beneficial to define the bounds of the influence of connections in the whole response of the structure.

The selection of the connection systems is totally related with the cost. The structural efficiency is one important variable, but not the most important one. Simplicity is the key. The current and logical trend is to reduce the number of joints, which clearly makes no longer reasonable to work with the *Gamma* method [2] [3]. The parametrical studies can offer a quick tool for selecting the connection system, in advance, and establishing a previous structural efficiency.

The international research on at prefab solutions is being now important. The most important works are located in the Nordic Countries (see e.g. [5][6][7]), in New Zealand and Australia (see e.g. [8]), and Germany (see e.g. [9]).

New ideas are emerging, but we think that this is still an open field, and the previous work of global analysis tools and parametrical analysis would guide the search of new possibilities.

Section 2 will summarize the obtained results until now and part of the ongoing works. Section 3 compiles discussion and conclusions.

2. Previous results and ongoing works

2.1 First results

As it was commented, this work relies in a previous national research project. As it was the first work for the authors in this topic, an important effort was just made in the state of art. A document oriented for the professionals was compiled [10].

Around 50 joists have been already tested. Joists were preferred instead beams for the sake of simplicity, and considering that this experimental program had already previous international references.

Coach screws, self-tapping screws, nails and rebars with epoxy resin have been until now used as connectors; most of them placed vertically, but also with and angle of 45 degree. It has been used different distributions of the connectors, not always regular. Two types of section were used (see *Fig. 1*). The lower component was always timber (glulam most of times), but the upper one was concrete and LVL (Kerto in particular) .The use of LVL showed really good results.

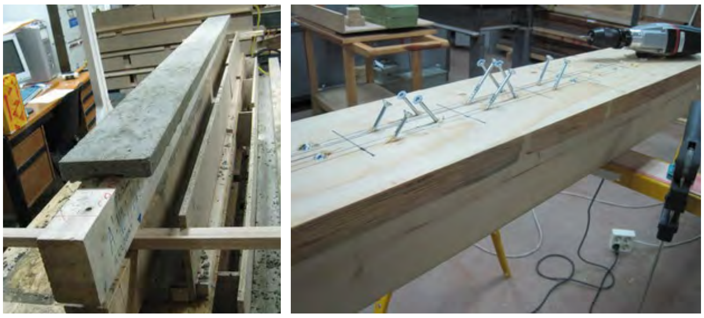

Fig. 1 a) glulam-concrete and b) glulam-LVL composite joist

The results of the experimental program, still unpublished, is being compiled and analyzed in other Ph.D. Thesis [11]. The new proposals for the analysis are being checked with those experimental data.

The theoretical work was mainly focused in the connections systems. A first paper [12] and a Ph.D. Thesis [13] describe these results.

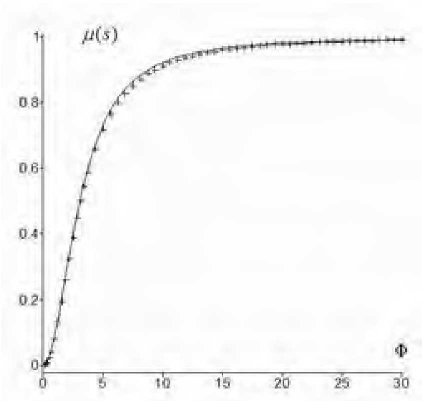

Fig. 2 relative efficiency versus the non-dimensional parameter Φ

The efficiency of the composite beam can be described in terms of just two non-dimensional parameters; and just one, Φ, when the shape of the beam is not a variable. The relationship between the relative efficiency (between 0 and 1) in strength and Φ is non linear (see *Fig. 2*). Φ is function of the square root of the unitary stiffness of the connection system, k_u, (i.e. the quotient between the stiffness of one connection and the distance between connectors).

Fixing all the other variables, that relationship (see *Fig. 2*) offers a deep insight over the influence of the stiffness of the connections. As the relation is non-linear, there is one value of stiffness beyond that a more stiff connection system is needed.

2.2 Literature review

The work on the state of art showed how impressive the scientific and technical knowledge in this field was. It is difficult to understand why the Eurocode-5 contains just the *Gamma* method. Any of the existing work has the drawback of being too complex for using at the professional field. A huge amount of literature has been now compiled. It has been step by step analyzed, with an especial focus in the most simple but precise approach. The work in this topic is being used for one of the next steps: to update and develop new analysis tools.

2.3 Methods of analysis

It is clear that using Finite Elements, with a local discretization, the analysis problem would be always solved. But a total local approach is not practical from the professional side.

We think that a global or a local-global approach is desirable; i.e. avoiding a total local discretization. Two first methods of analysis, one analytical, using the global flexibility matrix, and one numerical, using SAP-2000®, has been already developed and implemented [2][3]. Some results presented in [2] and [3] are now showed in *Fig. 3* and *Fig. 4*. Those data correspond to the short term analysis of a simple supported timber-concrete composite beam of 20m span with a constant continuous load.

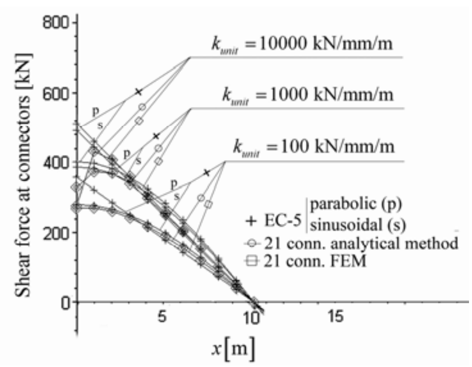

Fig. 3 Shear force at connectors (kN) versus distance from the left support: case 1, h_g = 0 m, s= 1 m ⟶ x_0= 0 m.

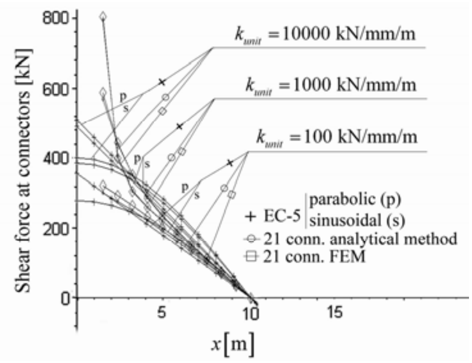

Fig. 4 Shear force at connectors (kN) versus distance from the left support: case 1, h_g = 0 m, s = 0.85 m ⟶ x_0 = 1.5 m.

Fig. 3 plots the shear forces for a separation between connectors of s = 1m, which means a distance from the left support to the fist connector of x_0 = 0 m. The shear force distribution for the lowest value of unitary stiffness, k_{unit} =100 kN/mm/m, presents a typical shape. But as the stiffness is increased, a *shear lag* at the first connector appears. *Fig. 3* also shows that the greater the stiffness, the higher the *shear lag*. This *shear lag* is surprising, and it is not detected by the *Gamma* method of EC-5 for neither the sinusoidal solution nor the parabolic one.

Fig. 4 shows the results for a spacing between connectors of s = 0.85m, which means a distance from the left support to the first connector of x_0 = 1.5 m. *Fig. 4* presents a typical shape for the lowest stiffness, k_{unit} =100 kN/mm/m. However, for the other two higher values an opposite tendency appears in comparison with the first case of s = 1m (*Fig. 3*).

I.e., there is a notable increase in the shear force at the first connector (which will be termed *shear advance*). In parallel with the first case of s = 1m, *Fig. 4* shows that the greater the stiffness, the greater the *shear advance*. This phenomenon cannot also be detected using the *Gamma* method of EC-5.

The numerical approach with SAP-2000® is very precise and simple. Additional work is being now developed expanding that first publication. Similar works have been recently developed in Portugal [14], even though they have being developed independently. Other working lines are being also explored. The research for improving ductility in timber connections is now an active area, as the robustness of the structure strongly relies on that parameter. Quantifying ductility is absolutely needed (see [15]).

Elasto-plastic analysis has been already proposed by some authors (see e.g. [16] [17]), but it is still an area worth of additional research. The obtained experimental data are also being used to determine when Elasto-plastic analysis viable. The use of the plastic range, when can be assured, notably increase the robustness of the structure.

2.4 New construction systems

The authors are working also at the construction field. Two working lines can be differentiated: the connection system and the structural type.

2.4.1 Connection systems

The literature also contains the description of a huge amount of connectors. Many of the systems are protected by patents, which makes complex a collective development. The Standards contains data of just a few cases of the whole existing catalog. It is necessary to clarify how the real advantages and disadvantages of these solutions are. The span is a variable of the problem, a question not always considered. Tests on connections are now made in Europe using the Standard EN 26891:1991 [18]. But this Standard does not precise the set up. The definition of an explicit set up is desirable, as other researchers have already mentioned [19]. Additionally, guidelines for the use of the new optical measurements are needed. A work about those questions has been already published by this research team [20].

2.4.2 Structural types

The new proposals around prefab beams are being now very active (see references at the Introduction). But the ideas presented at the timber composite beam can be implemented for obtaining new structural solutions. Some ideas has been already developed by this research group, even tough are still unpublished. It is the authors opinion that the more relevant advances will be more connected with technology than with science.

3. Discussion, Conclusions and Acknowledgements

The paper described the first results and ongoing works made at the Polytechnic University of Madrid around the timber composite structure, with particular emphasis to the beam. The first stage covered experimental works and theoretical ones. The theoretical results offered the analytical tools for selecting the connection system, and first proposals for new tools in the analysis (as an alternative to the *Gamma* method). The use of commercial Finite Element packages (SAP-2000® in our case) seems a very promising working line.

The experimental results are been still compiled and analyzed in a current Ph.D. Thesis. Main targets for this new stage of research are: to make an extensive literature review, to implement new and general analysis methods for substituting the *Gamma* one; to study the conditions for applying Elasto-plastic analysis, increasing the robustness of the structure; to improve experimental set up´s using optical measurements and to develop of new connection systems and structural types beyond. This new stage in the research was born with a previous project, with title "*Definition of a protocol for the refurbishment of timber-concrete composite floors*", funded by the Spanish Government and developed between 2004 and 2007 under the coordination of Jose Miguel Avila-Jalvo.

We would like also to thank the support of this COST Action FP1004 for approving the STSM of María Bona-Gallego, to be developed in Norway under the supervision of Professor Dr. Ing. Kjell Arne Malo; and the STSM of Edurne Bona-Gallego, to be developed in Sweden under the supervision of Professor Dr. Ing. Roberto Crocetti. We really appreciate the kind support offered by these two Professors. These two STSM will be an impulse in our work, and we also hope a first step for a standing collaboration.

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Timber-Concrete composite floors with prefabricated Fibre reinforced Concrete (FRC) deck

Tiziano Sartori[1], Luca Costa, Roberto Crocetti[2]

Summary

The presented investigation concerns the study of novel types of timber-concrete composite floors, manufactured with so called “dry connections”. Two full-scale floor elements, each with a different shear connection system, were experimentally investigated. The two shear connector systems used in the investigation were: i) shear anchor-key of furfurylated wood and ii) inclined steel tubes. In both cases, the connectors were incorporated in the prefabricated concrete slab, before casting of concrete. The slab was then connected to the underlying timber beams by means of self-tapping screws. The top slab for both specimens was made of fibre reinforced concrete (FRC). Both dynamic and quasi-static bending tests were performed. The spans of the two floors were different, which resulted in different depths of the timber beam. It was found that both the proposed connection systems performed more than satisfactory, showing a very high degree of composite action even at load levels close to the failure load. Moreover, it appeared that the assembling of the prefabricated fibre reinforced concrete slab with the underlying beams occurred in an extremely easy fashion.

1. Introduction

Timber-concrete composite structural systems have been extensively used during several decades both in floors and in bridges. Most of the studies performed to date have focused on composite systems where “wet” ordinary concrete was cast on top of timber beams with preassembled different shear connectors. Even though such systems have proven to perform very well from the point of view of statics and dynamics, in-situ concrete casting has some clear disadvantages, e.g:

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- the introduction of a “wet” component (concrete) on a typically “dry” structure (timber)
- the time needed for the concrete to cure, which adds to the time required on-site before the next scheduled action can be taken
- low stiffness and high creep while the concrete cures
- the high cost of cast-in-situ concrete slabs, mainly due to the cost of transporting fresh concrete, the use of props, the use of formworks and the labor.

The investigation presented herein focus on the use of composite structure with high prefabrication level and high performance. For such a purpose, completely dry connection systems were investigated, i.e. systems where the prefabricated concrete slab is connected to the timber sub-structure only by means of self-tapping screws. Moreover, the use of very efficient materials – such as modified wood (furfurylated) and fibre reinforced concrete (FRC) – is investigated.

2. Materials and Method

In order to investigate the behaviour of the prefabricated composite system, two full-scale tests were carried out at the laboratory of Structural Engineering, Lund University. The spans were different for the two tested floors, namely 6 m and 8 m respectively, which resulted in two different cross sections for the timber beams, namely 90x360 mm^2 and 90x450 mm^2. The timber used for the manufacturing of the floors was spruce glulam (GL30c). The moisture content of the beams was approximately 12%. For the production of $1m^3$ of FRC, 45 kg of steel fibres and 375 kg of cement, which gave a mean value of compression strength $f_c \approx 55$ MPa for the concrete. In the following, the two tested systems will be referred to: *F45* and *T12*. The shape of the specimens is shown in Fig.1.

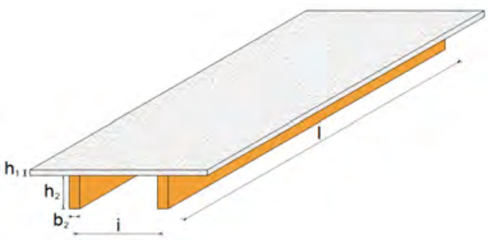

Fig. 8: Geometry of the tested specimens

During the bending tests, the load was applied by an actuator in a displacement controlled manner. The load was distributed on four lines in order to induce stresses and deformations in the floor similar to those induced by a uniformly distributed load.

The total load applied to the specimen, the mid-span deflection, and the relative slip between slab and beam at the supports was continuously measured during testing.

2.1 Manufacturing of F45 floor system

The main dimensions of F45 floor system are reported in Table 1.

Table 1: Geometry of the F 45 composite system dimensions in [mm]. See also Fig 1.

Span (l)	*Slab width*	*Slab thickness* (h_1)	*Beam width* (b_2)	*Beam depth* (h_2)	*Beam spacing* (i)
6000	1600	50	90	360	800

For the manufacturing of the F45 floor system, wooden shear anchor-keys were applied in the prefabricated slab before concrete was cast. Self-tapping double threaded screws with dimension 6.5x220 were driven in the anchor-keys, perpendicularly to the direction of the applied load, before concrete casting, see Fig.2. The main functions of such screws are to allow for a proper anchorage of the shear anchor-key to the concrete slab and also to reduce the risk for premature splitting of the anchor-key. The anchorage between concrete slab and underlying timber beams was made by self-tapping screws with dimensions 7x180 driven through the shear anchor-key to the timber sub-structure with an inclination of 45°, see Fig.2. The ultimate tensile strength of the screws was approximately $f_u =$ 1250 MPa.

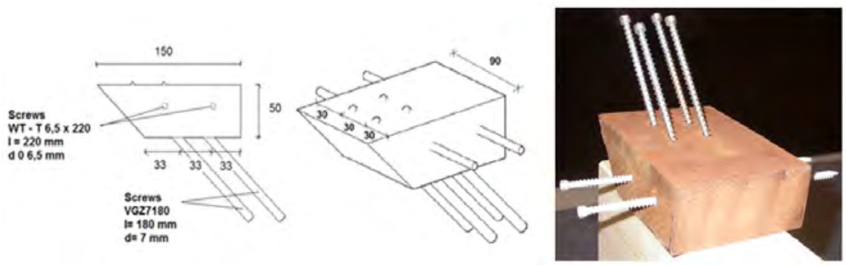

Fig. 2: Shear connectors in F45 specimens

The wood used for the anchor-key was maple impregnated with furfuryl alcohol. Furfurylation improves shape stability and compression strength and make it more suitable to be used with the concrete. The spacing between the wood anchor-keys was 250 mm, for a total of 24 anchor-keys on each beam.

2.2 Manufacturing of T12 floor system

The main dimensions of T12 floor system are reported in Table 2.

Table 2: Geometry of the T12 composite system dimensions in [mm]. See also Fig 1.

Span (l)	*Slab width*	*Slab thickness* (h_1)	*Beam width* (b_2)	*Beam depth* (h_2)	*Beam spacing* (i)
8000	1600	50	90	450	800

For the manufacturing of the T12 floor system, special steel tubes were applied in the formwork before the prefabricated concrete slab was cast.

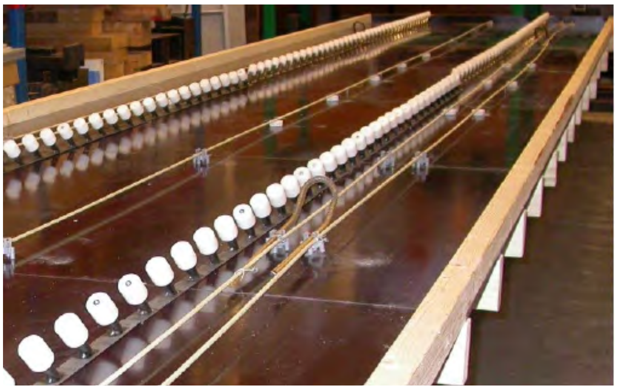

Fig. 3: Shear connectors in T12 specimens (note the white plastic plugs inserted on the top of the tubes to prevent concrete from entering in the tubes during casting)

In order to achieve a better stiffness, the tubes had an inclination of 45° to the longitudinal axis of the beam. Approximately one month after the concrete was cast, the prefabricated slab was placed on the top of timber members. Successively, self-tapping screws with dimensions d = 11mm and l = 250mm were driven in the steel tubes and tightened to the timber sub-structure. The spacing between the steel tubes was 100 mm.

3. Results and Analysis

Deformations of the shear connectors generate horizontal movement, i.e. slip at the interface between concrete and timber. Such a behavior is referred to as "partial composite action" and, as the slip increases it reduces the efficiency of the cross section. The efficiency of a shear connection for a composite beam can be estimated using the following equation:

$$\eta = \frac{EJ_{real} - EJ_0}{EJ_\infty - EJ_0} \tag{1}$$

where η is the efficiency, EJ_∞ is the bending stiffness of the beam with full composite action, EJ_0 is the bending stiffness of the beam with no composite action and EJ_{real} is the actual bending stiffness of the beam.

3.1 Behavior of the F 45 floor system

The main results concerning F45 floor system are shown in Fig. 4. The curve on the left shows the relationship between the "equivalent uniformly distributed load" q (i.e. the total load applied divided by the slab area) and the deflection f at mid-span. As it is observed, the behaviour was linear up to a load level of approximately 35 kN/m^2, which is well above the design load for common floor structures. The decrease in stiffening observed after this load level was reached, may be the result of a severe crushing which occurred at one of the beam support, due to large compression perpendicular to the grain.

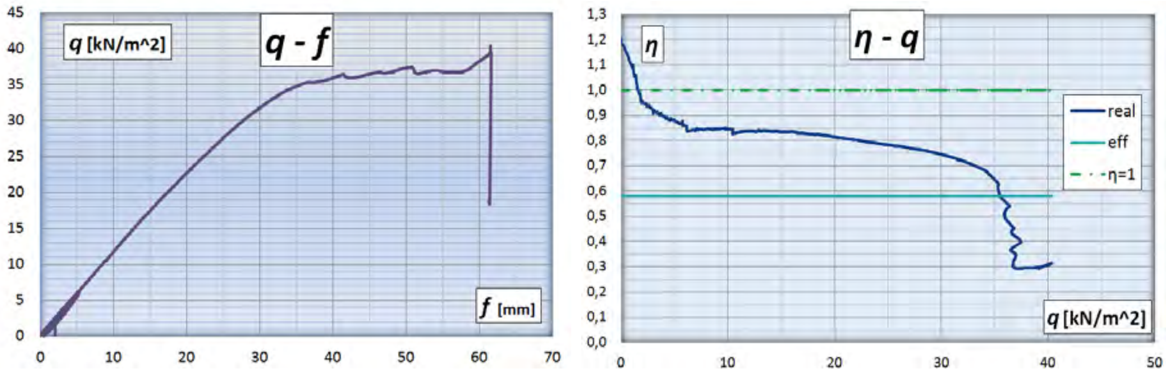

Fig. 4: Left: Load-deflection behavior and Right: efficiency-load curve

The curve on the right shows the variation of the efficiency η with increasing load levels. At load levels comparable to those at the serviceability limit state (i.e. 1-3 kN/m^2) the efficiency is high, close to 1.0. The "real efficiency" (dark blue) was always higher than the "effective efficiency" (light blue) as evaluated according to the γ-model, proposed in EC 5 (Annex B). The collapse of the system occurred at $q \approx 42$ kN/m^2, with the propagation in one of the two beams of two large cracks in the direction parallel to the grain. The lower crack was located at the interface between the second and the third lamella in the bottom part of the timber beam; the upper crack along a line running through the points of the screws used in the shear connections For the *F45* system, the first natural frequency determined experimentally was $f \approx 14$ Hz, which is in agreement with the value of the first eigenfrequency determined analytically.

3.2 Behavior of the T 12 floor system

The main results concerning T12 floor system are shown in Fig. 5. The curve on the left shows the relationship between the "equivalent uniformly distributed load" q (i.e. the total load applied divided by the slab area) and the deflection f at mid-span.

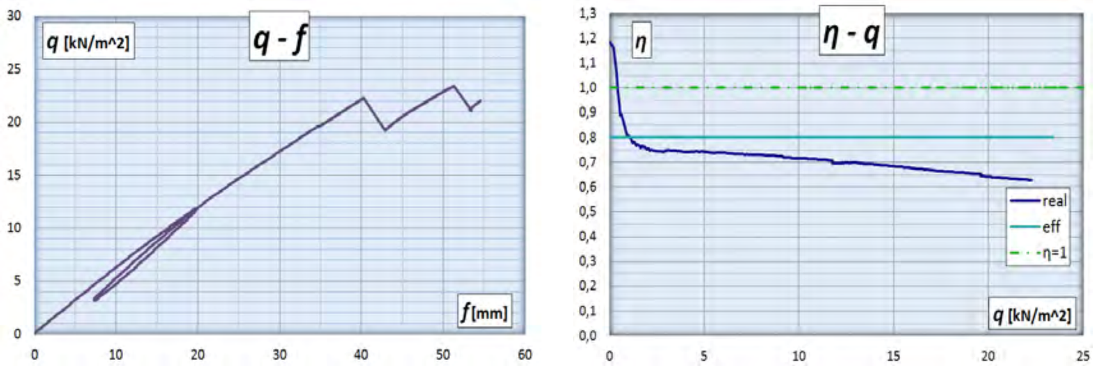

Fig. 5: Left: Load-deflection behavior and Right: efficiency-load curve

As it can be observed, the behaviour was linear up to a load level of approximately 22 kN/m^2, which is well above the design load for common floor structures. The curve on the right shows the variation of the efficiency η with increasing load levels. At load levels comparable to those at the serviceability limit state (i.e. 1-3 kN/m^2) the efficiency is high, close to 1.0. The collapse of the system at $q \approx 24$ kN/m^2 occurred due to bending failure at a finger joint in one beam and at a knot in the other beam. Both the finger joint and the knot were located close to mid-span. For the *T12* system, the first natural frequency determined experimentally was $f \approx 11$ Hz, which also is in agreement with the value of the first eigenfrequency determined analytically.

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Vibration of Timber Structures with Dowel-Type Connections

Thomas Reynolds[1]

Summary

The ability to derive dynamic properties of structures for use in their serviceability design has the potential to reduce the incidence of complaint by building users, and the necessity of remedial work. This is particularly so in the case of lightweight, flexible structures such as those found in well-designed, cost-effective timber construction, where dynamic serviceability criteria are more likely to govern the design than in conventional structural materials. Understanding the behaviour of connections is critical to being able to predict dynamic response. This project seeks to develop a method for estimating dynamic stiffness and damping in timber structures with dowel-type connections. A test method is proposed to estimate the dynamic properties of a connector, and a method is put forward to combine the response of each dowel and connection to predict the modal properties of a whole structural system.

1. Introduction

1.1 Dowel-type Connections

Dowel-type connections are widely used in timber structures because of their convenience in construction and their long history of successful use. This type of connection can be flexible in comparison with the structural members it connects, as force is transmitted over the relatively small contact area between the connector and the timber. The connections can, therefore, play a significant role in determining the stiffness of the structure as a whole.

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1.2 Vibration

When vibration of the structure is considered, the effect of the connections on stiffness is important, as, along with the distribution of mass, the distribution of stiffness determines its natural frequencies and mode shapes. Another property of the structure also becomes important in vibration: its damping. This represents the energy dissipation in the structure as it vibrates. In structural analysis, damping is generally estimated based on the global properties of the structure, and on empirical evidence from structures previously completed in that form. This project takes another approach, by assessing the energy dissipation in the components of the structure, and combining the energy dissipation in each mode of vibration to generate the damping ratio for the structure in that mode.

The significant contribution of the connections, in a structure with dowel-type connections, to its damping and stiffness is recognized in the field of seismic engineering. Researchers have analyzed and modeled the response of individual joints to cycles of load, and used the results to generate models of whole structural systems [1, 2]. This project applies a related approach to the contribution of connections to the way a structural system behaves under the smaller forces and displacements associated with their vibration in service conditions.

2. Experimental Work and Connection Modeling

An assessment of the damping in a structure requires knowledge of the proportion of the energy which is dissipated with each cycle of vibration. Experimental work and modeling has been carried out to assess the conserved and dissipated energy in connections as they vibrate. The modeling uses the assumption that the energy is dissipated by friction between the connector and the timber. Whether other mechanisms contribute significantly to the energy dissipation must be assessed using the experimental results.

2.1 Dynamic Embedment Tests

For individual connectors, damping and dynamic stiffness are evaluated using a test method analogous to the static test methods in British and European standard BS EN 383 [3], and American standard ASTM D5764 [4]. This is so the embedment of an individual rigid dowel can be assessed and used to predict the behaviour of a whole connection. These tests isolate the stiffness and energy dissipation which result from the embedment of the dowel, held rigidly straight by the loading piece, into the timber. The loading scheme, shown in Fig. 3, has been chosen to be representative of the along wind vibration of a structure due to turbulent wind load, in that it consists of the combination of an oscillating force and a constant mean force, and the magnitude of the oscillating force is sufficiently small that there is no reversal of the load. The fact that there is no reversal of the load makes it possible to conduct the tests using a half-hole specimen.

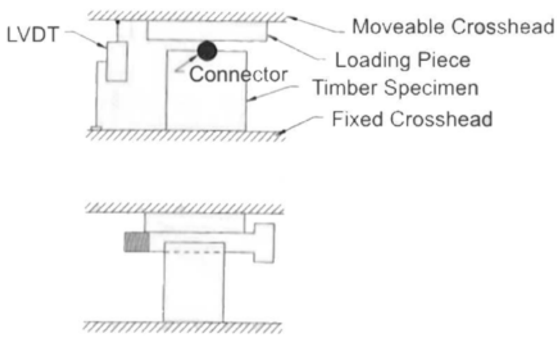

Fig. 2 Embedment test setup according to ASTM D5764

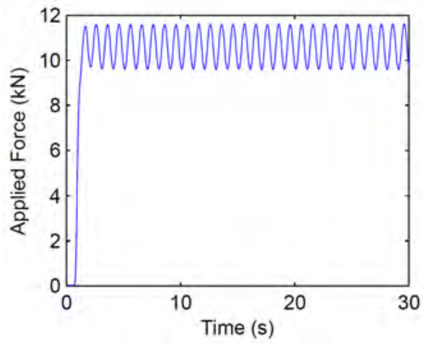

Fig. 3 Typical loading scheme for dynamic embedment tests

Damping is measured by analysis of the hysteretic force-displacement diagram for the dowel under cycles of load. The area inside the hysteretic loop on the force-displacement diagram, as shown in Fig. 5, represents the energy dissipated in that cycle.

The process of the dowel embedding into the timber is modeled in two dimensions as the loading of a plate by a rigid pin. This problem has been addressed for composite materials in aircraft design through the use of complex stress functions, as first proposed by Lekhnitskii [5]. The method was then applied to timber connections by Echavarria et al. [6]. Since this model can incorporate friction between the dowel and the timber, it has the potential to allow estimation of frictional energy dissipation and stiffness in dowel-type connections.

The calculated displacement field in the test specimen is shown in Fig. 4. It is based on the geometry of the test shown in Fig. 2 and an applied load of 10kN on a 20mm diameter dowel in Douglas fir. The calculation uses a complex stress function proposed by Zhang and Ueng [7] for application to a general composite material.

Knowledge of the deformations in the timber can be used to predict the deformation of the connection. It also enables assessment of the slip between the surface of the dowel and the timber around the dowel which in turn can be used to calculate energy dissipation by friction. The fact that the stress and deformation are represented by an analytical formula means that analytical formulae can be derived for calculating values such as the dowel deflection or the peak stress. Such formulae could be used in design codes.

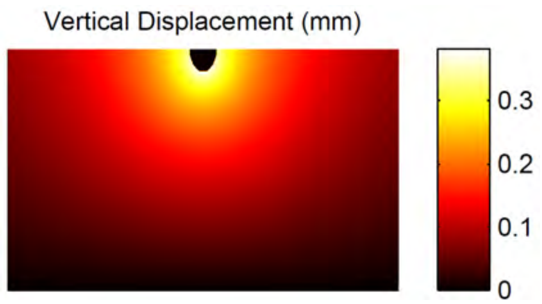

Fig. 4 Displacement field around a dowel in timber loaded perpendicular to the grain

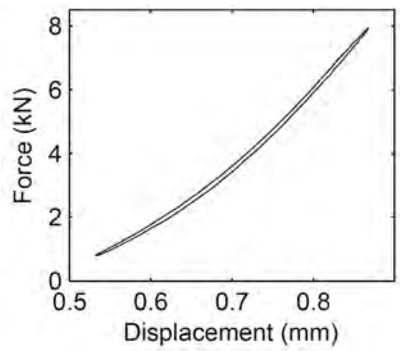

Fig. 5 Force displacement diagram for Douglas fir with 20mm dowel for a single cycle of load

2.2 Connection Tests

The experimental part of the project will go on to test real connections, where the deformation of the dowel contributes to their stiffness and damping. A beam-on-elastic-foundation model will be used for prediction of the response of each dowel, where the properties of the foundation can be determined by the rigid-dowel method described above. Additional energy dissipation is now generated by the friction of the dowel with the parent material as it pulls through the hole.

The parameters derived from the analysis and testing of connections will be used in a global structural model, for determination of modal properties. The parameters will, in general, vary with time and loading and therefore enable a prediction of the dynamic properties of the structure at various times in its design life.

3. Structural Modeling

The tests on dowels and connections described above identify the proportion of the energy put into a system or component which is dissipated under a cycle of load. A structural analysis procedure is being developed which calculates the distribution of elastic energy in a structural system in each of its modes of vibration, so that the dissipated energy in each part of the system can be assessed, and a global damping ratio for each mode of vibration can be identified.

The results of an example calculation are shown in Fig. 6. On the basis of this calculation, and using the results of the connection tests, the amount of energy dissipated in each member and connection could be found and summed to find the total energy dissipated in the structure, allowing calculation of the damping ratio for this mode of vibration.

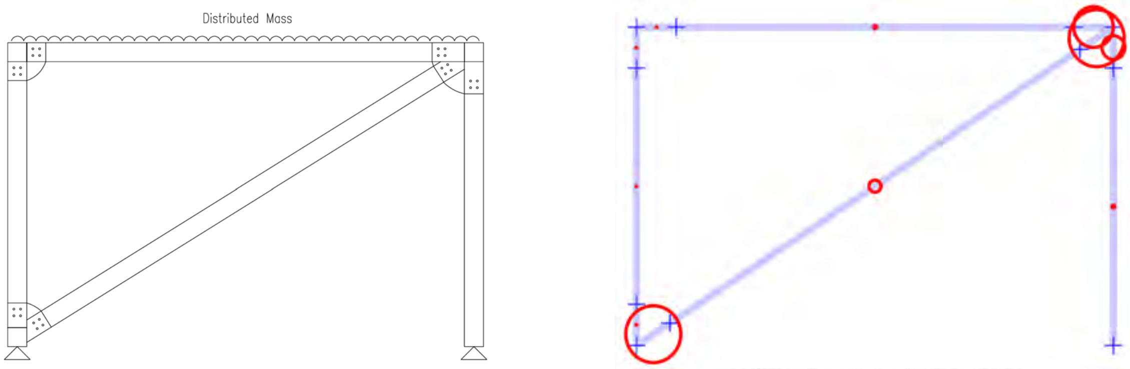

Fig. 6 Distribution of elastic stored energy in the fundamental mode of vibration of a simple structure. The size of the circles indicates the magnitude of the work in that component.

4. Conclusion

This project investigates the vibration of timber structures with dowel-type connections. Its particular focus is vibration under service conditions, where there is no yield of the connector material and the response is near-elastic. Such low amplitude vibrations may be caused by dynamic loads such as wind and footfall. The response of the structure as a whole is considered as a combination of the responses of its component parts. The stiffness and damping of individual connectors is assessed, and used to develop a model for a whole frame, so that accurate modal properties for the frame in each of its modes of vibration can be derived.

The research will proceed to propose a method to evaluate the damping in a frame structure in a particular mode of vibration, based on the properties of its members and connections, and to look at how the dynamic stiffness of the structure changes with applied load and time under service conditions.

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Advanced Methods for Design, Strengthening and Evaluation of Glued Laminated Timber

Lenka Melzerová[1], Petr Kuklík[2]

Summary

This article summarizes our experience with testing and FEM models of beams from glued laminated timber. These experiences provide the basis for our project. This article is focused on the twenty beams from the glued laminated timber with the real structure dimensions. All beams were tested in bending up to their destruction. Different parameters were monitored during bending tests including relation between displacement of beam and its loading. In our previously articles were demonstrated that modulus of elasticity in fibres directions (E_x) are more important for displacements. Non-destructive testing of modulus of elasticity is a key element. Precisely according to experiments are made 2D isotropic FEM models. The agreement between experiment and calculation is excellent.

1. Introduction

Displacement is more important parameter for building structure and for our twenty beams is necessary draw a comparison between their displacements. Uniform maximal loading level for all twenty beams was detected, because for each beam came damage in different loading level. This uniform loading level is 24 kN for each from two forces in real bending test and 60 kN/m in 2D FEM model, because length of loading is 0,4 m.

In our previously articles were demonstrated that modulus of elasticity in fibres directions (E_x) is for beam displacement significant, but on the distribution of finger joints is displacement practically independent. For our experiments is

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necessary the knowledge of distribution of modulus of elasticity in fibre direction as precisely as possible. Measurements of E_x were realized for twenty beams in each segment of each lamella on four places. Final file of 1448 values of E_x is for statistical research sufficient. The rightness of measurement was verified independently via strain gauges and displacements sensors on the several places of each beam during loading tests. The relation between stress and strain from the stain gauges during the loading tests is linear. Now is a possible make FEM models of tested beams.

2. FEM models according to real beams

The first set of FEM models is made according to real tested beams. The distribution of joints in beams is observed as precisely as possible. Individual modulus of elasticity in fibres direction is put in each segment of beam. In our previously works were tested different FEM models. Firstly was tested 3D orthotropic and secondly 2D isotropic. Results from both types of models were practically identical (for this type of structure and loading). For timber structure are 3D orthotropic models very appropriate for details and different type of lading than is stress in fibre direction. Presented results are from 2D isotropic FEM models. Compare of measured and computed values of beams displacements are demonstrated to Fig. 1 and 2.

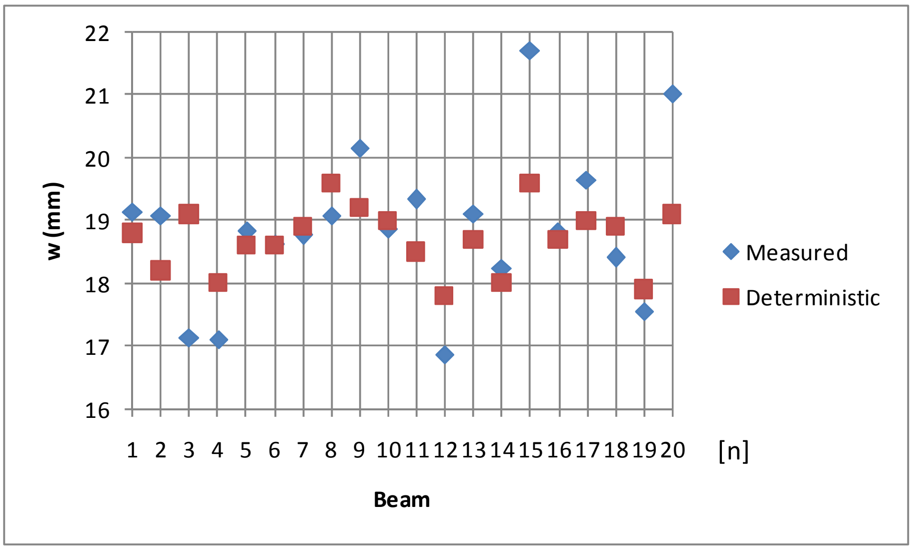

Fig.1 Comparison of measured and numerically derived deflection using deterministic FEM model applied to 20 beams each loaded by 60 kN/m

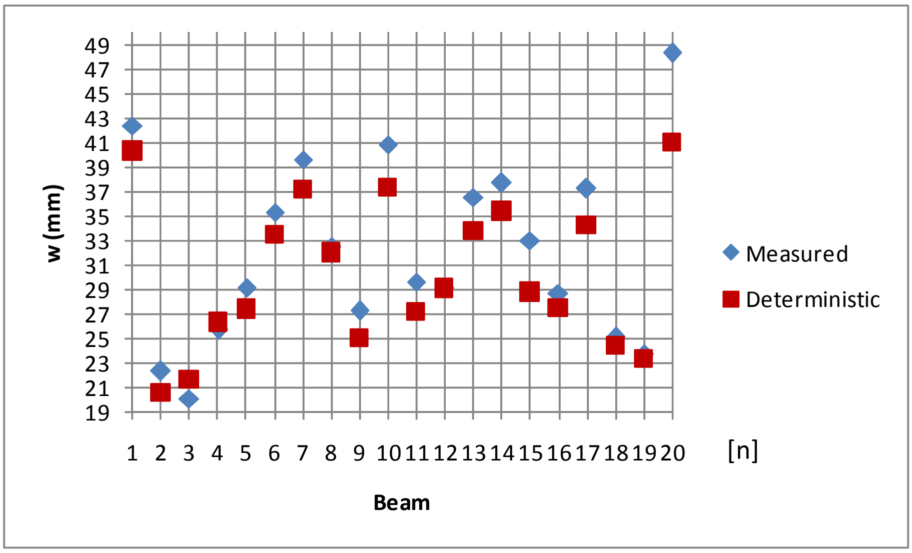

Fig. 2 Comparison of measured and numerically derived deflection using deterministic FEM model applied to 20 beams loaded by their maximal loading

On Fig. 1 is demonstrated compare of beams displacements for the same loading for each beam. Compare of displacements for maximal loading is on Fig. 2. The correspondence measured and computed values are excellent [2]. Different input files of E_x are compared in Fig. 3.

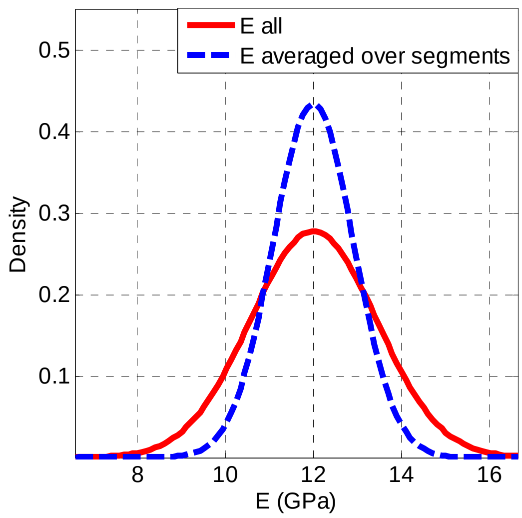

Fig. 3 Comparison of probability density functions of Young's modulus E derived from all 1448 measured data and from values averaged over individual segments

Low curve of probabilistic density function is for file of all 1448 values of E_x. Upper curve of probabilistic density function is for file in each segment constants values of E_x. Mean in all causes is the same but variances are different [1]. On the Fig. 4 is demonstrated precisely distribution of lamellas and finger joints in one from twenty beams. Each beam has 8 lamellas with 4 cm thickness. But finger joints have in each beam individual distribution. Loading level is here comparative.

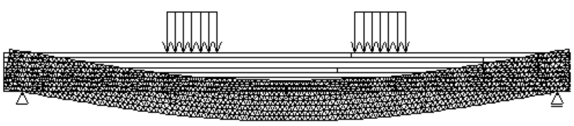

Fig. 4 Illustrative example of FEM model

3. Conclusions and Acknowledgements

For beams from the glued laminated timber tested in bending are better 2D isotropic models than 3D orthotropic. Correspondence between displacements in FEM models and measured displacements is excellent. Normal distribution is responsible for all files of E_x . Use of lognormal distribution is possible too. During production of beams from glued laminated timber is better distributed timber in beams according to values of E_x because these beams are more economical. More important is distribution of E_x in beams for large structures. The following work will focus on detailed non-destructive detection of modulus of elasticity distribution in beams from glued laminated timber as inputs to the FEM models. Each segment will be made larger number of non-destructive tests. This number will be determined via detailed statistical analysis. This outcome has been achieved with the financial support of the Ministry of Education, Youth and Sports of the Czech Republic, project No. LD12023.

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Bonded-in rods connections: modeling of mechanical behavior

Julie Lartigau[1], Jean-Luc Coureau[2], Stéphane Morel[3], Philippe Galimard[4], Emmanuel Maurin[5]

Summary

Structural bonding technology has proven to be an economically and attractive connection method in timber engineering. Within old or historical wooden structures, local reinforcement of weak zones is frequently provided by bonded-in rods connections. Despite previous research programs, some design rules, predicting pull out or push out strengths, are available but a common criterion is not proposed yet. The combination of experimental and numerical investigations on small samples leads to a better knowledge of mechanical and fracture behavior of such connections, by assessing stiffness, pull out strength and fracture energy. The main results aim at supplying information relative to the safety prediction of bonded-in rods connections.

1. Introduction

Bonded-in rod connections are one of structural bonding process widely used for the repair or the reinforcement of wooden structural members. Using structural adhesive, such connections offer aesthetic benefits (inconspicuous joints and rods hidden in the members' cross sections) and preserve a large part of original timber. Several research projects [1, 2] were achieved to improve the knowledge of this method, exhibiting experimentally the influence of various material or geometric parameters: kind of adhesive, thickness of adhesive, type of rod, glued-in lengths, species of wood … As a result of these studies, some design rules [3, 4, 5],

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predicting the axial strength, are available but a general approach is still lacking. Besides, few numerical investigations on such connections exist, that narrows the understanding on mechanical and fracture behavior of each component. The present study combines experiments with finite element (FE) computations. Numerical analysis reveals important information on mechanical behavior of bonded-in rod connections, difficult to evaluate with experiments only. For instance, the stress distribution at wood/adhesive interface and the evolution of stiffness according to a crack length can be estimated. Within this knowledge, an inverse method using fracture mechanics is applied.

2. Experiments on bonded-in rod connections

2.1 Test set-up

45 test samples (cube shaped, edge of 50 mm) are made with spruce (Picea Abies L.), reaching an average density of 395 kg/m^3. The moisture content of samples is around 12%, at the time of bonding. Threaded steel bars of grade 8.8 (yield strength of 640 MPa and minimum tensile strength of 800 MPa) are used (diameter of the bars: 8 mm). Rods are introduced in the direction of the fiber of the wood and are bonded with a two-component epoxy adhesive (thickness of adhesive: 2 mm and glued-in length: 50 mm). Epoxy adhesive is chosen for these connections because it reveals an increasing strength in comparison to other kinds of adhesives, such as polyurethane or phenol resorcinol [6]. Samples are tested on "pull out" configuration, under displacement control (0,5 mm/min), so that failure is obtained in 3 minutes in order to minimize visco-elastic effects. Test machine recorded axial load applied to rod and three LVDT (Linear Variable Differential Transformer) precisely measured displacements between wood and rod. Two LVDT are placed on each specimen, to provide global displacement of the connection (wood + glue joint + rod displacements) according to axial load. The third LVDT is placed on the rod in the end of the bonding, to

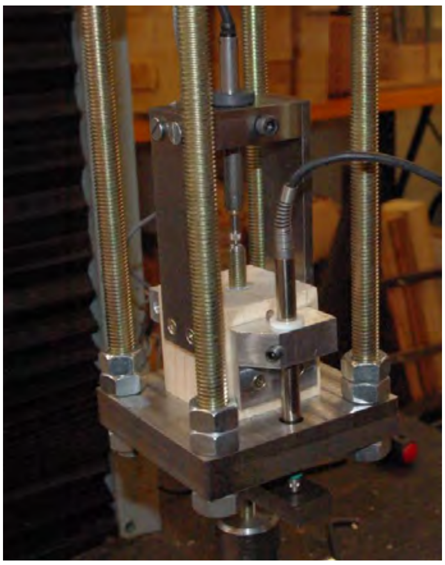

Fig. 1 LVDT on sample

evaluate more accurately the rod displacement at the end of the anchorage. Displacements of the rod are estimated from a reference point located on the lateral faces of the wood samples (fig 1).

2.2 Results

Failures at wood/adhesive interface are observed on all samples: wood surrounding bond is extracted with rod and epoxy cylinder. So, shear stress at wood/adhesive interface (failure zone) is calculated, by dividing the maximum load by the shear area. Measurements device by LVDT allowed analyzing each axial load-displacement curve. They presented the same trend and three distinct parts can be shown:

- An initial elastic field, from which the initial stiffness of the connection can be estimated
- A non linear zone, in which the progressive decrease of the stiffness appears and probably indicates the beginning of damage/crack propagation at wood/adhesive interface
- The failure (at wood/adhesive interface)

Average and 95 percentile values of stiffness and pull out shear stress are given in the following table.

Table 1 Experimental results

	Stiffness (N/mm)	**Shear stress (MPa)**
Average value	58 775	6,3
95 percentile value	70 175	8,9

Stiffness is an important parameter required for FE computations, especially to calibrate the model in the elastic domain.

3. Numerical computations

3.1 Modeling features

The experimental "pull out" configuration is also reproduced on a FE modeling (Castem® 2009 software). Specimens' geometry, boundary and loading conditions revealed an axis of symmetry through the middle of the rod. Thus, a 2D axisymmetric model is used, with a revolution axis corresponding with the rod axis. An elastic model is firstly used to calibrate the initial stiffness of the connection. Mechanical properties of each material of the connection are supposed to be homogeneous in the cross section and are listed in the table below.

Table 2 Materials characteristics

Wood	**Rod**	**Adhesive**
Spruce	Steel	Epoxy
E_L = 11 430 MPa – E_R = 905 MPa – E_T = 560 MPa	E = 210 000 MPa	E = 4 000 MPa
ν_{LR} = 0,39 – ν_{RT} = 0,49 – ν_{LT} = 0,42	ν = 0,3	ν = 0,3
G_{LR} = 780 MPa		

E means modulus of elasticity, ν indicates Poisson's ratio and G is the shear modulus
L, R and T are the three orthotropic directions
Wood properties are estimated for hardwoods with an average density of 395 kg/m3 [7]

The elastic model allows evaluating the initial stiffness of the connection. Numerical value of stiffness is upper (36% higher) than mean experimental value.

3.2 Influence of mechanical properties

An experimental design is set up to evaluate the influence of the relevant mechanical properties on stiffness: E_L, G_{LR}, E_{adh}, ν_{adh} and E_{steel}. Three values, or levels (marked –, 0 and +), are affected to each mechanical property and are chosen according to experimental material values.

Tab. 3 Levels chosen

	E_L (MPa)	**G_{LR} (MPa)**	**E_{adh} (MPa)**	**ν_{adh}**	**E_{steel} (MPa)**
Level –	10 290	700	2 000	0,2	200 000
Level 0	11 430	780	3 000	0,3	210 000
Level +	12 570	860	4 000	0,4	220 000

This method consists in varying levels together in each simulation, which allows decreasing the number of numerical tests. Stiffness is computed on Castem®, by varying values of mechanical properties, in accordance with an experimental design (Taguchi method). Results prove that numerical stiffness is mainly governed by the modulus of elasticity of the adhesive (74,7%). Experimentally, it may be adherence defects on interfaces (bonding without pressure), that are not symbolized in the model and these imperfections can explain differences between experimental and numerical values of stiffness.

3.3 Crack propagation at wood/adhesive interface

The distribution of stresses along the anchorage length is assessed at wood/adhesive interface (failure zone). This evaluation exhibits significant tensile stresses transversally at the beginning of the bonding, in comparison with shear stresses (10 times higher). The outstanding consequence is that the crack propagation mainly occurs in mode I rather than in mode II.

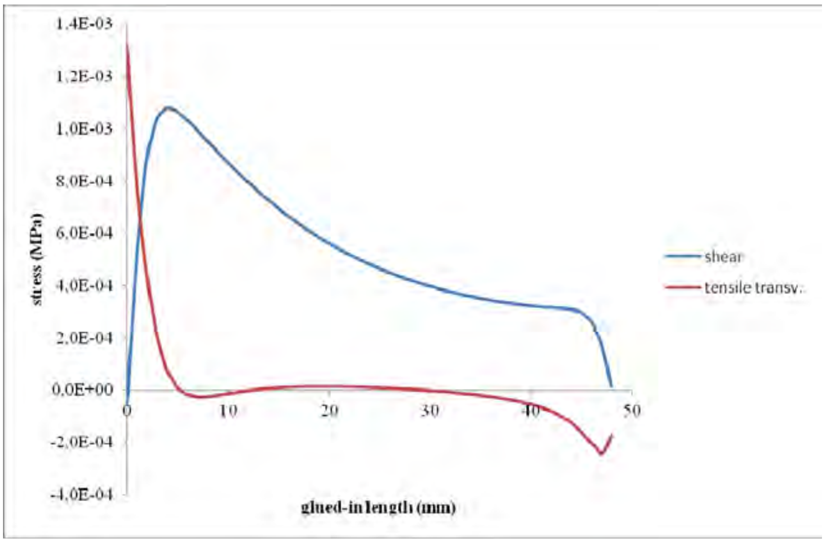

Fig. 2 Numerical assessment of stresses at wood/adhesive interface

Thus, the initial approach used, to evaluate strength of bonded-in rod connections, is focused on fracture mechanics in mode I. In this way, the beginning of nonlinearity in load-displacement curves (decrease of stiffness) is wholly imputed to the formation of a crack at wood/adhesive interface. Hence, this approach requires the evolution of compliance (inverse of stiffness, i.e. $C = \delta/P$) according to the crack length (a) and allows the determination of the elastic energy release rate in mode I ($G_I(a)$), using equation (1):

$$G_I(a) = \frac{P^2}{2}\frac{\partial C}{\partial A} = \frac{P^2}{2b}\frac{\partial C}{\partial a} \tag{1}$$

Crack propagation is expected when $G_I(a) = G_{Ic}$, where G_{Ic} corresponds to the failure energy of wood in mode I. In a first approximation, fracture energy of spruce is chosen in agreement with [8] and is equal to 150 J/m^2, and leads to good estimate of the peak load of bonded-in rod connections despite the unstable crack propagation.

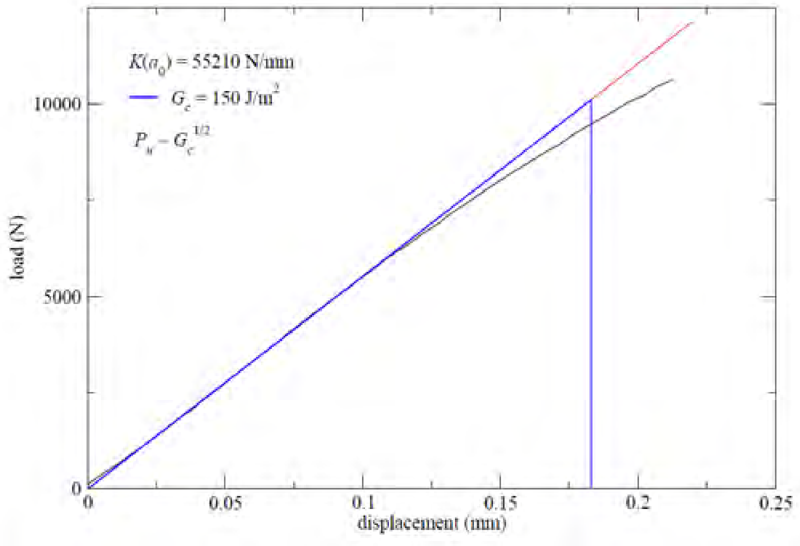

Fig. 3 Load-Displacement (LD) curves: experimental and fracture mechanics approach

Experimental LD curve

Assessment of stiffness by a linear regression

Reconstitution of LD curve by fracture mechanics approach in mode I

Brittle fracture behavior of bonded-in rod connections is obtained numerically (see fig 3): fracture mechanics in mode I supplies a first approximation. Failure of samples at wood/adhesive interface and numerical analysis of opening stresses in glued joint reveal that transversal tensile strength of wood generates the fracture behavior of bonded-in rod connections.

4. Conclusion

Behavior of bonded-in rod connections (experimental LD curve) is investigated with a numerical inverse method, using FE modeling and fracture mechanics. In a first approximation, only the fracture energy of spruce in mode I is taken into account, which allows predicting the peak load of a tested sample. Fracture behavior of bonded-in rod connections will be considered through a new modeling, taking into account the contribution of mode II (shear mode) and using a fracture criterion in mixed mode (mode I + mode II).

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Contactless Measurement with Close-Range Photogrammetry (CRP)

Karl Rautenstrauch[1], Wolfram Haedicke, Martin Kaestner[2]

Summary

In many fields of wood engineering the photogrammetric measuring system that has been developed can lead to new insights, because the determination of strains can be determined independent of their direction and their location in the whole area of interest. Currently the highest accuracies in measuring of strain distribution are possible with the 2D-photogrammetry method with telecentric measuring lenses, circular measuring marks and a mark localization with the subpixel-method. With the measuring system it is possible to capture the crack geometry to specify fracture mechanics parameters. This was done in recent research projects for notched structures, beams with openings and connections with a load axis perpendicular to the grain [1][2][3]. With the help of CRP it was also possible to identify material parameters for a constitutive material model for wood which can be used in FEM analyses [1][4]. The results of CRP measurements are very helpful for the calibration and verification of FE-Models. By inverse FE-simulations on the basis of the photogrammetric data, the complete analysis and graphics capacities of commercial finite element software can be used for processing and visualization purposes. This is very useful for the understanding of the behavior of components under load. In the future the application potentials of CRP should be expanded through the development and application of appropriate error filters.

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1. Introduction

From the beginning of the scientific analysis of component tests strain measurement was an important tool, because only through knowledge of the strains, the stresses can be determined. The measuring methods for strain gauging have been constantly developed. Initially, only fine-mechanical sensors were available. Now, in the age of electronic data processing, electronic measurement methods are state of the art. In civil engineering strains are typically determined by the use of glued strain gauges. Disadvantage of these methods is that the measurement can only be made locally and usually only in one direction. Therefore it is very difficult to measure multi-axial strain states. Another drawback is that these measurement techniques influenced the test object due to the direct application. For the estimation of strains, regardless of the direction and in larger areas without affecting the test object, in recent years various systems were developed and were used in the investigation of wood specific problems. In this context, especially optical methods such as the Close-Range Photogrammetry (CRP) and the electronic speckle pattern interferometry (ESPI) should be mentioned.

2. Application of CRP on wood specific problems

2.1 What is Close-Range Photogrammetry

In opposite to the so called Aerial Photogrammetry which is typically used for topographic topics in close-range photogrammetry the camera is close to the subject and is typically hand held or on a tripod. CRP has its applications in a wide variety of measurement tasks for example architectural and archaeological plans, volume calculations, forensic science and high precision measurement in industrial manufacturing or research.

2.2 Possibilities of the developed system

An specifically developed procedure, based on digital close range photogrammetry and image processing, enables to measure the progression of deformations, cracks and deteriorations during the loading and unloading of specimens. The significance of a test can be substantially increased by applying measuring marks or imaging the specimen's surface. The contactless determination of the coordinates is done by a photo sequence over the total duration of the test followed by automated evaluation with a computer program. As the evaluation is done in the post process, a frequency can be achieved in the test of up to 5 Hz. The gathered displacements of the coordinates of the measuring marks or other geometrical marks are easily transferable for a following processing in a numeric simulation by a Finite-Element-program. The achieved results having accuracy in micrometer range lead to new insights for timber constructions.

2.3 2d- versus 3d-photogrammetry

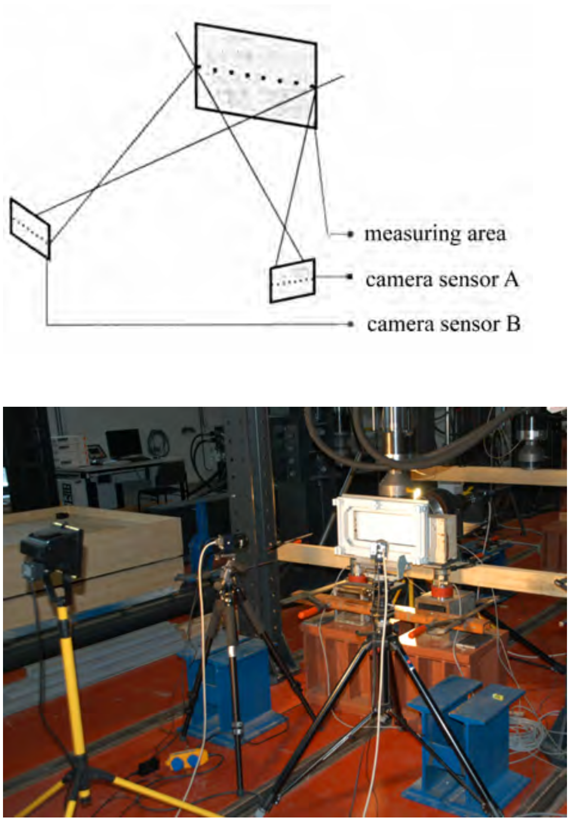

Fig. 1 Example for a 3d-measuring

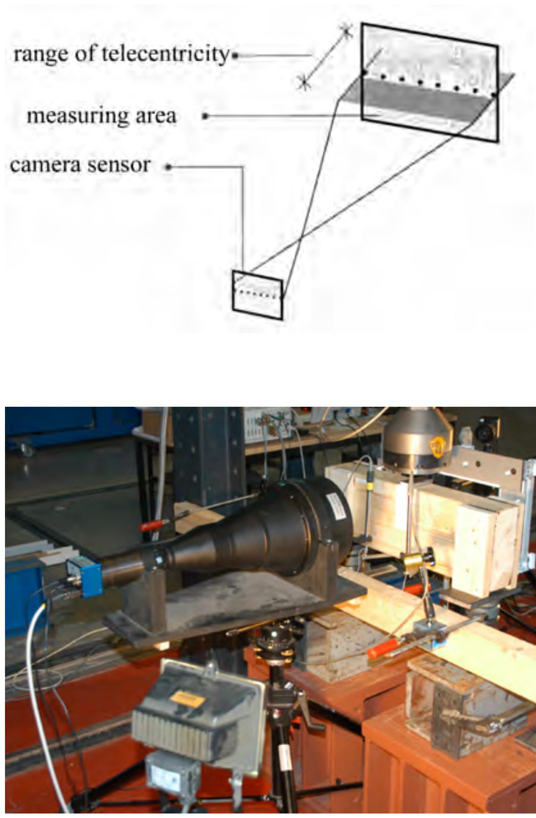

Fig. 2 Example for a 2d-measuring

In principle CRP is common as 2d or 3d systems. However, for most applications 3d systems are used. The main advantage of the 3d-systems is the quasi unlimited measuring area and the obtaining of three-dimensional object information. For this at least 2 images which have to be taken from different directions are needed. For moving objects this means in reverse, the images have to be taken at the same time. So, at least 2 cameras are needed. The more cameras respectively observation perspectives are used, the higher is the accuracy. If only relatively small areas with flat surfaces and negligible movements perpendicular to the component surface should be examined, a 2d-system, consisting of only one camera with a telecentric measuring lens, can be used. This alternative photogrammetric measuring system has a very high and even higher accuracy than the 3d-system. All further descriptions are restricted to the 2d-system, although of course, measurements were made with a 3d-system.

2.4 Used hard- and software

The basis of the measuring system are high-resolution digital monochrome cameras with telecentric measuring objectives. The cameras have a resolution of 4008 x 2672 pixels with a maximum picture rate of 5 images/second. The technology of the camera is based on a CCD-sensor as established for cameras in industrial image processing. A CCD-sensor consists of a geometrical grid of photosensitive cells. The picture information is represented by the number of electrons in the cells. The CCD- matrix camera sends a digital picture, which is transferred through the framegrabber to the PC. The image information can be taken out for processing and further analysis routines by the help of various software. In case of 2d-measurements at the department of the authors the program OSPREY is used for the determination of the coordinates of the marks. The marks are detected by the subpixel method whereby the high accuracy is possible.

The selection of a suitable measuring objective is important for the measurement. The applied telecentric measuring objectives are two TZL3005/0.33 with a magnification of 1:3. These objectives have a parallel optical path and enable a measuring field of approximately 70 mm x 108 mm. The specimens, based on pure 2D-measurement, can move slightly depth wise without creating a distortion on the measuring results.

3. Some examples of application

3.1 Examination of transverse tensile stressed areas of notched structures

Notches represent an unstable (stepwise) change of cross section which leads to stress concentration singularities and can no longer be represented by elastic theory. The stress must actually decrease to the edge of the notch. Otherwise a failure would occur under smallest loads. For design of, e.g. reinforcements, the estimation of the size of transverse tensile stressed area and the tension resulting from this area are needed. With the photogrammetric measuring technique progressions of transverse strain in crack growth direction could be determined for the accomplished test series. Figure 3 shows the applied measuring grid for photogrammetric measurement and the definitions of the specimen.

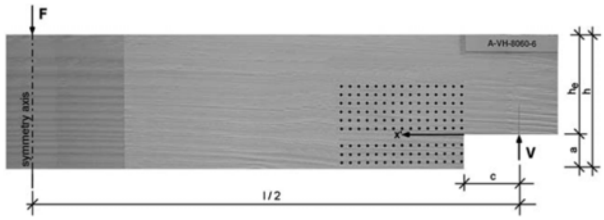

Fig. 3 Test specimen with measuring area [3]

For two specimen with different notch ratios Figures 4 and 5 contain the determined transverse strain progression for the comparison with the strains from the modeling with the program ANSYS®.

By determination of real strain distributions of the specimen a increasing of transverse tensile stressed area was detected. With increase of notch ratio *a*, defined as $a=h_e/h$, the difference between strain distributions determined by FE-simulations and by tests also increase. [3]

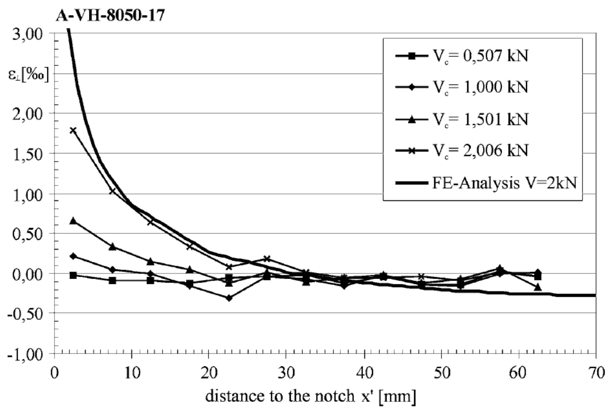

Fig. 4 Transverse strain progression between the internal two point rows for α = 0.625—solid wood

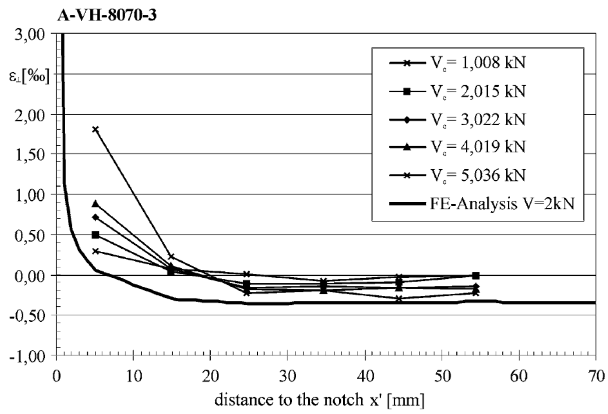

Fig. 5 Transverse strain progression between the internal two point rows for α = 0.75—solid wood [3]

3.2 Glued in steel rods

The load capacity of a glued connection is mainly affected by the geometry and the difference of stiffness between the materials. The geometry, basically the anchorage length, influences the stress distribution at the end of the bond line. At least there is a singularity which induces peaks in the stress distribution at each end respectively in zones of material changes. The slenderness of the connection increases with an increasing mounting length. Therefore the inhomogeneity of the stress-distribution rises. To determine the allocation of strain distribution CRP was used at different specimens during the tests. In order to use the CRP special specimens have to be produced because normally the area of interest inside the test body is hidden. With this special test setting and the use of CRP the different effects which affected the load capacity could be investigated. Next to the effect of the inhomogeneity of the stress distribution is another effect is the deflection of the force lines. Even if the rod is exactly glued in the central axis of the timber bar, the force has to be bypassed over the bond line. The eccentricity induces transverse forces to the grain of wood. These forces can exceed the capacity of tension force of timber perpendicular to the grain with increasing thickness of bond line. This effect becomes relevant with bond-failure at the end grain. Figure 6 shows, that the stress is almost linearly distributed along the bond line. After the failure of the bond at end-grain, there appear significant peaks in the end of anchorage zone, shown in Figure 7. [5]

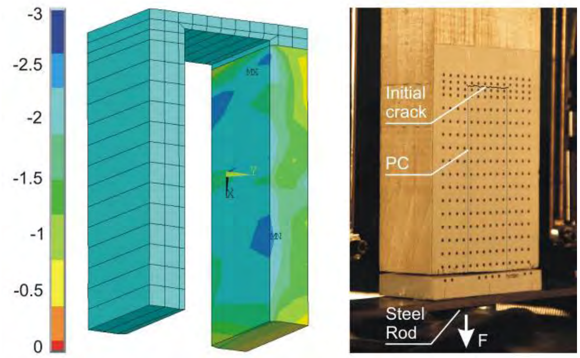

Fig. 6 Left: shear-stress distribution in the adjacent wood before bond-failure at the end-grain gluing in N/mm², inverse FE-simulation based on close-range photogrammetry (CRP); Right: initial crack at the end-grain

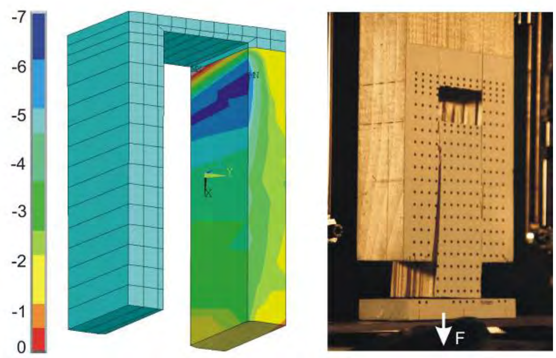

Fig. 7 Left: shear-stress distribution in the adjacent wood after tension-failure between polymer concrete (PC) and timber at the end-grain gluing in N/mm², inverse FE-simulation; Right: final longitudinal shear-failure along the wood

4. Planned improvements

Currently the highest accuracies in measuring of strain distribution are possible with the 2D-photogrammetry method with telecentric measuring lenses, circular measuring marks and a mark localization with the subpixel-method. The measurement error depends on the external conditions (vibrations e.g), on the used hardware (resolution of the camera, imaging properties of the lens etc.), on the applied marks (sharp edges, quality of roundness, diameter etc.) and on the mark localization method. When using the 2d-photogrammetric measuring system, which was developed at the author's department, the localization of measuring points under optimal conditions has a margin of error of approx. ± 1-1.5 μm [1]. In the stress analysis of very stiff materials or large volume specimens, where only small strains occur, even small measurement errors lead to results for certain stress components which do not represent the real situation. In order to make meaningful statements about the stress distribution also in these cases, it is necessary to find methods to eliminate the error. For such problems, the statistical method of compensation for the response surface method is established. The method should be used as a post-processing procedure and works with the 2d displacement-data which were determined by the mark localization method for every measuring mark and every time-step. By using standard mathematics software (Maple), the following function should be applied to the displacement-field of each time-step.

$$f\left(x,y\right) = p_0 + p_1 \cdot x + p_2 \cdot y + p_3 \cdot x \cdot y \; + \\ \sum_{i=2}^{n} \left(\begin{array}{l} p_{(4+(i-2)\cdot 5)} \cdot x^i + \; p_{(5+(i-2)\cdot 5)} \cdot y^i + \; p_{(6+(i-2)\cdot 5)} \cdot \; x^i \cdot y^i \; + \\ p_{(7+(i-2)\cdot 5)} \cdot x^i \cdot y \; + \; p_{(8+(i-2)\cdot 5)} \cdot x \cdot y^i \end{array} \right) \tag{1}$$

It follows the determination of the parameters p_n by the use of a nonlinear statistic fit algorithm. After that the average deviations to the measuring values are checked. If it exceeds a tolerance value the function is increased by one degree by the program logic. This process is repeated until the tolerance limit is complied. The result is a function which represents the smoothed displacement-field. This function can be used for further interpretations.

5. Conclusions

In many fields of wood engineering the photogrammetric measuring system that has been developed can lead to new insights. The test series that have already been completed provide many interesting results, which considerably contributed to new findings in timber engineering and new opportunities in the field of experimental stress analysis and experimental technique.

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Linear members made of cross-laminated timber (CLT)

Marcus Flaig[1]

Summary

As a cross-laminated product, CLT has a number of advantages over glued-laminated timber (glulam). In particular the tensile strength perpendicular to the beam axis is considerably higher and therefore CLT has a much lower sensitivity to cracks causing numerous damages to glulam components in the past. The basic idea of this research project is hence to develop linear CLT components like beams able to replace glulam members where large tensile stresses perpendicular to the grain occur, e.g. beams with notches or holes, members with connections loaded perpendicular to the grain and tapered or double tapered beams.

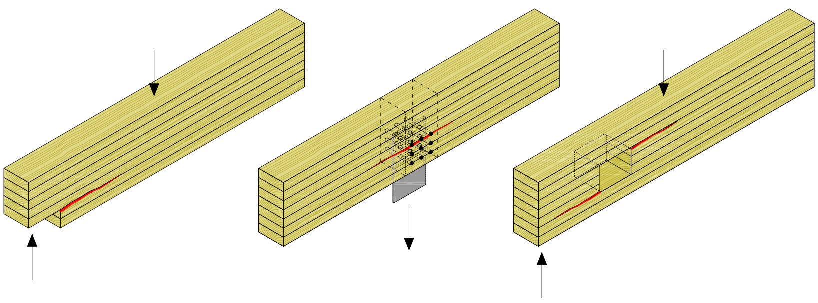

Fig. 1: Components made of glued-laminated timber are prone to cracking when exposed to tensile stresses perpendicular to the grain

In glulam components tensile stresses perpendicular to the grain often result in uneconomically large cross-sections or require cost-intensive reinforcement measures. In CLT components the cross-layers, as an integral element of the product, will provide sufficient reinforcement if they are suitably arranged and dimensioned. Since cross-layers are present over the entire length of CLT components, beams made of cross-laminated timber are also less sensitive to cyclic climatic stresses. Thus they are considered more robust than glued-laminated timber components.

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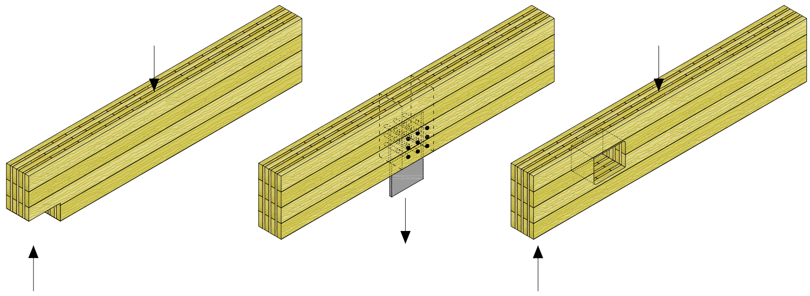

Fig. 2: Cross-laminated timber beams are less susceptible to cracking since cross-layers oriented orthogonally to the beam axis take effect of reinforcement

In longitudinal direction the load-bearing capacity and the stiffness of CLT-beams are rather poor compared to glulam beams with the same dimensions. In order to make linear CLT components more competitive these characteristics have to be improved. This can be achieved by reducing the proportion of cross-layers to a required minimum and by exploiting homogenisation effects resulting from the parallel alignment of multiple lamellae at the edge of a cross section.

1. State of knowledge

1.1 Bending strength of CLT-beams

In current technical documentations for CLT-products the bending strength of edgewise loaded CLT-members is calculated by means of simplified design rules as e.g. given in [1]. Usually the homogenisation due to the parallel alignment of multiple boards is taken into consideration only by the use of conservative approaches since no systematic studies into increasing strength in parallel systems with multiple boards have been conducted yet.

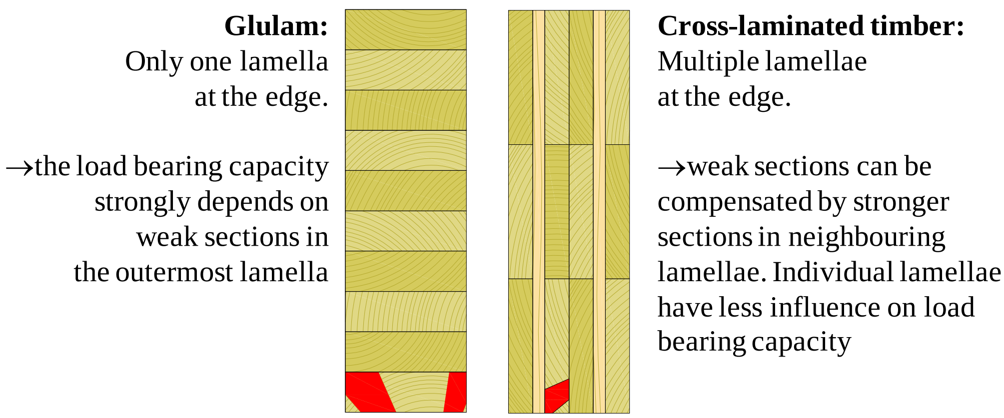

Fig. 3: Homogenisation due to parallel alignment of boards

The very high strength values in the direction of longitudinal layers as stipulated in technical approvals for multiple-layer boards do however indicate a high and as yet unexploited potential. However, the extremely high economic costs of determining the homogenisation effect by experiments, while also taking into consideration board quality and lay-up, does not appear feasible.

1.2 Load-bearing capacity of CLT-beams with notches or holes and tapered CLT-beams

Current technical documentations do not provide rules for the design of CLT-beams with notches or holes and tapered CLT-beams since only few tests or in the case of tapered beams even no tests have been conducted yet. However existing test results indicate very high load-bearing capacities for CLT-beams with notches or holes [2].

2. Approach of the ongoing research project

2.1 Simulation of bending strength and stiffness

A possible alternative for the determination of strength and stiffness values by tests is to calculate these properties using suitable computational models which in addition provide the possibility of quantifying homogenisation and size effects.

For the simulation of glulam beams a computational model has been developed at the Chair of Timber Construction and Structural Engineering at KIT (Karlsruhe Institute of Technology) as early as 25 years ago [3]. Since both glulam and CLT are products built up from finger-jointed softwood lamellae considerable parts of this model can be used for the simulation of CLT-beams as well.

Within the computational model Monte Carlo simulations are used to generate strength and stiffness properties of individual board sections on the basis of experimentally obtained frequency distributions of various wood properties and regression equations describing the relationship between these properties and the characteristic strength values to be generated.

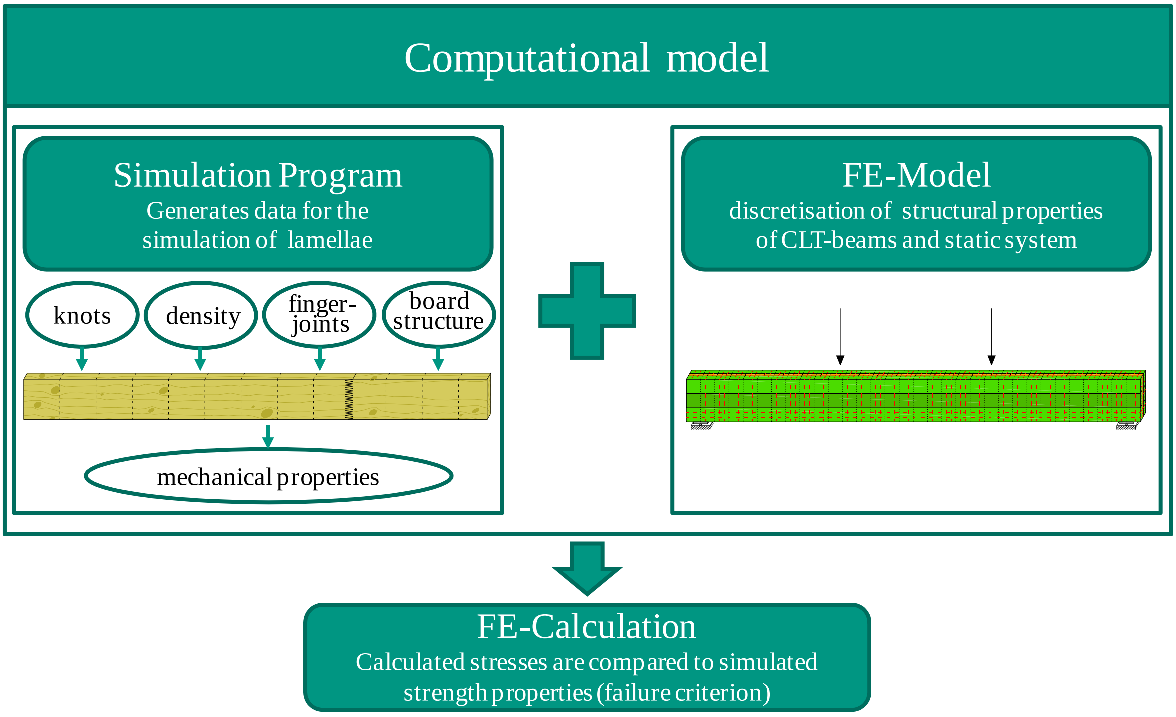

Fig. 4: Scheme of the computational model used for the simulation of bending strength of cross-laminated timber members

For the calculation of the strength and stiffness properties of CLT-beams the existing data given by the glulam model have to be prepared and supplemented with a couple of characteristics that are required in addition. For a number of variables, such as the bending strength of softwood, published frequency distributions and regression equations are available [4]. Due to a lack of relevant data, frequency distributions of other material properties, such as the edgewise bending strength of finger-joints and the rolling shear strength, have to be determined by tests.

2.2 CLT-beams with notches or holes and tapered CLT-beams

To determine the load-bearing capacity of CLT-beams which are subject to tensile stresses perpendicular to the grain tests have been carried out. In particular, test series involving load components perpendicular to the grain in connections, beams with notches, and beams with holes and tapered beams have been conducted. The test results shall serve as a basis for deriving design approaches for the beam types under investigation.

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Working Group 3

Modeling the mechanical performance of enhanced wood-based systems

In this area, scientific activities focus on increasing and consolidating the current knowledge of mechanical and structural behaviour of timber elements and systems - with enhanced performance - by use of advanced numerical modelling and analysis.

This scientific area includes:

- Identification of properties to be enhanced;
- Material properties needed in numerical models;
- Design and performance models of enhanced timber structures;
- Cracks parallel to the grain related to moisture content variations and different longitudinal shrinkage.

Advanced interface interaction in timber engineering joints

Kay-Uwe Schober[1], Michael Drass[2], Wieland Becker[3]

Summary

Experimental and numerical investigations have been done on timber-composite joints in truss structures with dowel-type fasteners embedded in high-performance ceramic fillers to study the embedding effects and the advanced composite action around the timber interface for design subjected to axial loading. To control block failure and ensure the maximum withdrawal resistance, different embedding materials have been used.

1. Introduction

In the design process the strength of the structure will be determined by the strength of the connections. The joint stiffness will greatly influence the displacement behavior and member sizes are often determined by the number and physical characteristics of the type of connector being used rather than by the strength requirements of the member material.

Different types of joints with mechanical fasteners for high-loaded truss structures have been developed in the past years. In most cases the full structural capacity of these joints cannot be used. Only 60-70% of the applied loads will be transmitted through the joint into the supports or other structural parts due to the low embedding strength as design criteria and reduced load-carrying capacity when loaded under an angle to the grain. To avoid this problem several investigations have been done on axially loaded truss structures transferring the forces into a composite joint with dowel-type fasteners embedded in special ceramic fillers to obtain the maximum withdrawal resistance around the timber interface.

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2. Composite friction model

Friction forces between the joint members as well as the higher withdrawal resistance of the fasteners due to friction interface interaction are mostly ignored in structural design. To account this additional load-carrying capacity for design, a cohesive friction model approach has been used to calibrate the numerical model with the experimental results [1].

To obtain the adhesive strength of composite joints embedded in timber the study of crack initiation and propagation is necessary when the joint geometry and connection surfaces are quite complex. This can be done by the use of special interfacial decohesion elements in finite-element modeling, placed between composites material layers or in the structure where cracking or joint separation can occur. These elements directly introduce fracture mechanism by adopting softening relationships between tractions and the separations, which in turn introduce a critical fracture energy that is also the energy required to break apart the interface surfaces by combining a stress based and a fracture mechanics based formulation.

For a single-mode delamination, the model result in an elastic relationship between the relative displacement and the traction in the timber-composite filler interface, as long as the stored specific elastic energy Ψ is lower than the critical energy release rate G_c, and in a sudden loss of adhesion when $\Psi = G_c$. Full delamination occurs and the crack propagates as soon as the equation is satisfied which involves the ratio between the dissipated energy for each mode and the critical energy release rate characteristics of the mode itself. For embedded dowel-type fasteners the interaction of shear and normal stresses has to be considered (mode II). NEEDLEMAN considered cohesive zone materials particularly attractive when interfacial strength are relatively weak compared with the adjoining material [2], as in the case of high-performance ceramic filler and timber. The proposed exponential interface law (Fig. 1) offers a continuously differentiable stress-crack opening-behavior and accounts shear, as well tension stiffness.

The decohesion response was specified in terms of a surface potential $\Phi(\delta)$ relating the interface tractions and the relative tangential and normal displacements δ_n and δ_t across the interface [3].

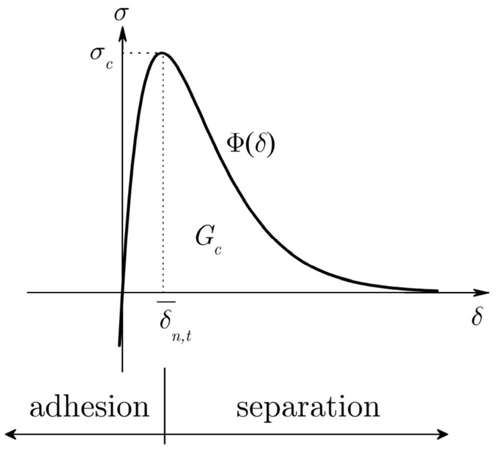


Fig. 1: Damage law for fracture mode I and II

A cohesive zone may be placed anywhere along element interfaces where fracture is expected to occur to take these effects into account. Furthermore, using decohesion elements, both onset and propagation of delamination can be simulated without previous knowledge of crack location and crack propagation direction.

3. Experimental and analytical investigations

Analytical investigations have been done first to estimate the pull-out strength of timber joints with glued-in steel rods. Here normative approaches and approaches based on research results were analyzed. The achievements of the analytical and experimental investigations will be compared in chapter 5. At this point it should be noted, that generally the analyzed approaches refer to steel rod glued with epoxy resin in timber, where the hole diameters are marginal larger than the steel rod diameter.

The experimental investigations serve for model comparison as well as obtaining the embedding stiffness of the casting compounds. Here a single steel rod was put into a drill hole embedded in different casting compounds. These different types of casting compounds used are shown in Tab. 1.

Tab. 1: Casting compounds

Type	**Commercial product**
Epoxy resin	WEVO EP 32 S
Mineral-bound mortar	BVD-Vergussmörtel
Polymer concrete (Epoxy PC)	Compono® 100 S
UHPC	Nanodur® Compound 5941

The purposes of the extensive experimental investigations were the characterization of the composite behavior and the determination of the load-carrying capacity for glued in rods in different composite compounds. The mean objectives are shown in the following table.

Tab. 2: Research objectives

	Objectives	Relevance
1)	Investigation of the different composite behavior	Explicit statement about the composite loading Good comparison of the casting compounds
2)	To enforce timber failure	Simplified design possibility
3)	To obtain the pull-out strength	Formation of a complex assemblage joint due to arbitrarily formwork possibilities

To fulfill these objectives the specimens were clamped in a special appliance (Fig. 2) to induce the fracture only in the adhesive bond line or pure timber failure due to shear overstress, where bending and shear in the interface can be neglected.

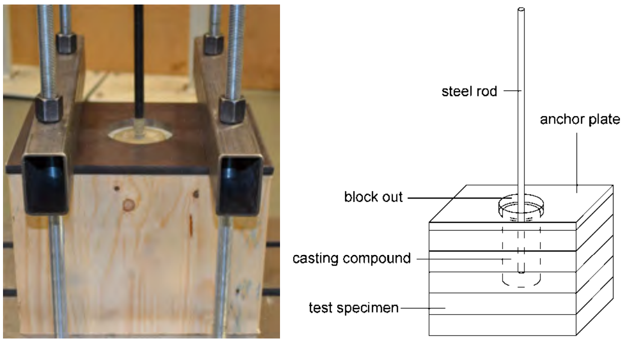


Fig. 2: Appliance of the experimental investigations

4. Numerical Investigations

The modeling of the composite structure has been done using ANSYS® by applying solid and cohesive elements in a 3D-model for steel rods glued in timber with epoxy PC. The FE-analysis has been verified with the test results.

The model consist of the following four components (Fig. 3):

1 timber specimen (cube of wood)
2 casting compound
3 steel rod
4 cohesive-elements

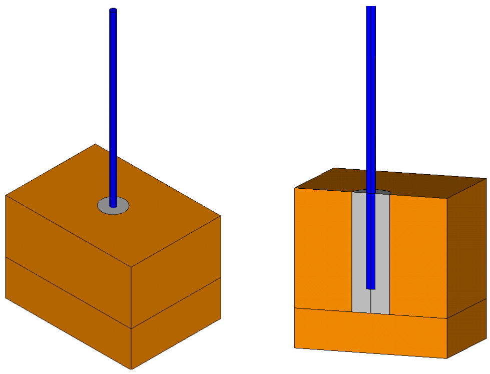


Fig. 3: 3D solid model of the composite structure

The cohesive elements have been placed along the adherent between steel and casting compound as well as casting compound and timber. The material constants for the numerical investigations are shown in Tab. 3.

Tab. 3: Mechanical properties for numerical investigation

Modulus		**Timber**	**Epoxy PC**	**Threaded rod M12 10.9**	**Structural steel BSt 500**
E_L	[MPa]	11.990	19.800	210.000	205.000
E_R	[MPa]	390	isotropic	isotropic	isotropic
E_T	[MPa]	820	isotropic	isotropic	isotropic
G_{RT}	[MPa]	42	isotropic	isotropic	isotropic
G_{LT}	[MPa]	720	isotropic	isotropic	isotropic
G_{LR}	[MPa]	620	isotropic	isotropic	isotropic
ν_{TR}		0,599	0,3	0,3	0,3
ν_{LT}		0,035	isotropic	isotropic	isotropic
ν_{LR}		0,055	isotropic	isotropic	isotropic

Interface-Element

Timber-casting compound	$\sigma_c = 2,5$ [MPa]	$\delta_n = 0,4$ [mm]	$\delta_t = 0,4$ [mm]

The material properties of timber apply to homogenous orthotropic material behavior. The casting compound epoxy PC and the threaded rod were assumed as multi-linear elastic. Referring to the results of the lab tests the compound between steel rod and adhesive layer can be assumed as rigid and shear-resistant.

5. Results

The comparison of experimental results related to the maximum tensile force are shown in Tab. 4 and classified by the drill hole diameter, load direction and steel grade. Tab. 5 show the obtained test results (Tab. 4) divided by the analytical results from different approaches.

Tab. 4: Pull-out force (mean value) in [N]

Glued-in rod	M 12 10.9		BSt 500		M 12 10.9		BSt 500	
Diameter [mm]	∅ 50	∅ 50	∅ 50	∅ 50	∅ 75	∅ 75	∅ 75	∅ 75
Loading to grain	‖	⊥	‖	⊥	‖	⊥	‖	⊥
WEVO	21.512	27.748	23.336	31.322	11.550	11.298	10.238	10.550
Bertsche	19.558	26.142	20.362	23.234	14.752	17.292	15.976	16.316
Epoxy PC	78.550	94.308	69.104	73.420	95.110	96.555	72.916	73.606
UHPC	4.690	5.540	6.704	3.720	9.802	16.075	10.484	9.710

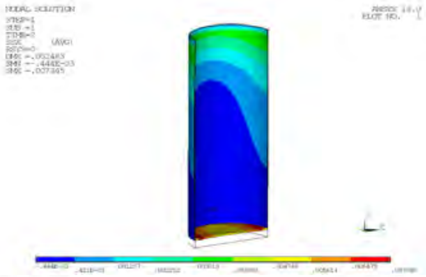
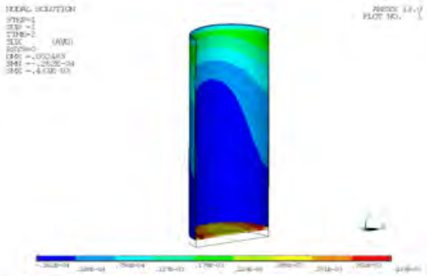
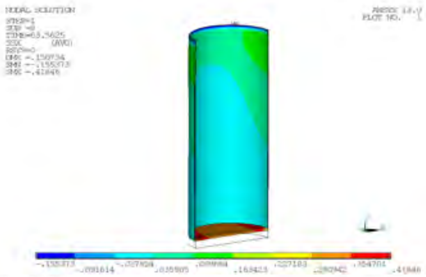
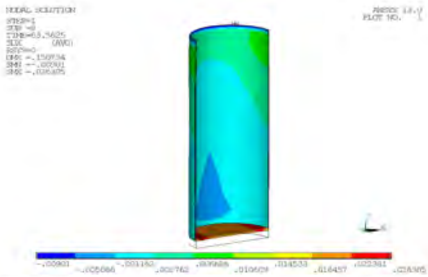
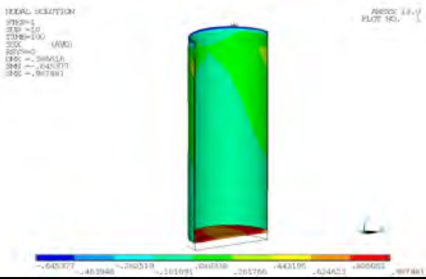
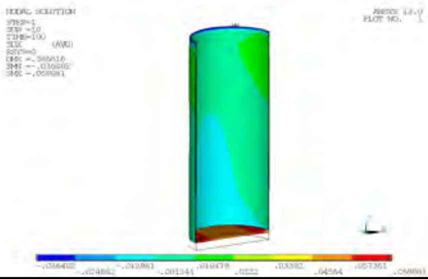
Tab. 5: Different approaches compared with the experimental results

Glued-in rod	M 12 10.9		BSt 500		M 12 10.9		BSt 500	
Diameter [mm]	∅ 50	∅ 50	∅ 50	∅ 50	∅ 75	∅ 75	∅ 75	∅ 75
Loading to grain	∥	⊥	∥	⊥	∥	⊥	∥	⊥
Comparison among each other								
WEVO	4,6	5,0	3,5	8,4	1,2	1,0	1,0	1,1
Bertsche	4,2	4,7	3,0	6,2	1,5	1,5	1,6	1,7
Epoxy PC	16,7	17,0	10,3	19,7	9,7	8,5	7,1	7,6
UHPC	1,0	1,0	1,0	1,0	1,0	1,4	1,0	1,0
Comparison with GIROD								
WEVO	0,7	0,9	0,8	1,0	0,4	0,4	0,3	0,4
Bertsche	0,7	0,9	0,7	0,8	0,5	0,6	0,5	0,5
Epoxy PC	2,6	3,1	2,3	2,4	3,2	3,2	2,4	2,4
UHPC	0,2	0,2	0,2	0,1	0,3	0,5	0,3	0,3
Comparison with Riberholt (1988)								
WEVO	0,3	0,3	0,3	0,4	0,1	0,1	0,1	0,1
Bertsche	0,2	0,3	0,2	0,3	0,1	0,1	0,1	0,1
Epoxy PC	0,9	1,1	0,8	0,9	0,8	0,8	0,6	0,6
UHPC	0,1	0,1	0,1	0,0	0,1	0,1	0,1	0,1

Referring to Tab. 4 the excellent composite loading capacity between epoxy PC and timber is conspicuous. In the test series Bertsche, WEVO and UHPC fracture occurred by adhesion failure; in the test series with epoxy PC as bonding material by pure cohesive failure in the timber and results in a much higher pull-out strengths. The comparison of analytical und experimental investigations show that only epoxy PC has the potential to achieve higher pull-out strengths. This was also confirmed when using the GIROD-approach, which is auspicious because of its actuality and the influence of lots of parameters as well as the implementation of parameters describing the fracture-mechanical behavior. However, by closer examination of the result comparison with Riberholt [4], it is conspicuous that the obtained axial pull-out resistance is more less than the results using Riberholt's approach. This can be explained by the assumed linear increase of the pull-out resistance by growing drill hole diameter.

The numerical model show good agreement with the lab results and addresses structural nonlinearities with debonding where the interface separation occurs first on the basement of the drill hole with subsequent stress redistribution to the lateral bonding area. A closer look to the gradual separation gives Tab. 6.

Tab. 6: Stress / Strain behavior of bond line in different load steps

F [kN]	**Stress in x-direction [MPa]**	**Strain in x-direction [mm]**
2,0		
63,5		
100,0		

6. Outlook

The deformation behavior of the specimen showed good agreement with the obtained numerical data. Referring to Tab. 2 it could be confirmed that only epoxy PC complies with all objectives. Further research is needed to obtain the long-term behavior of the composite structure and an analytical calculation model to make this efficient assembly accessible for practical design engineers.

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High-performance composite joints for spatial round wood truss structures

Wieland Becker[1], Jan Weber[2], Kay-Uwe Schober[3]

Summary

The anisotropic characteristics of wood still challenge the work on engineered timber constructions, especially for joints of spatial structures. Glued solutions for wood-wood or wood-steel connections with nearly 100% efficiency are known, but spatial structures are usually connected by expensive steel knots.

Naturally dried logwood has outstanding mechanical values and an excellent energy balance, but it is not considered in the current technical timber constructions. Our project presents a process which creates composite joints with force- and form optimized design and simple fastening methods. Occurring forces can be resisted, highly stressed sections are brought into the joints. They are optimized by finite element software and the computer aided manufacturing process allows efficient, individual solutions just as designs, where the joints are mass-produced.

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1. State of knowledge: Current situation and use of round wood truss structures

Although the high mechanical values of round wood is documented in EC 5 and the production process has very low energy requirements, round wood is usually not used and considered in contemporary timber constructions. In rare cases where round-diameters are used, profiles are made of glued laminated timber lamellas, which are crafted in cylindrical form to girders or columns. Industrial use of logs or round wood timber has the disadvantage, that the cutting of girder joints or girder connections cannot be manufactured by joinery machines. An automatic cutting process in different angles is currently not given. Manual cutting processes are not precise and reduce the application of round wood constructions up to simple constructions in rural environment. Different statical and mechanical requirements concerning compression, tension or bending forces cannot be corresponded without large deformation behavior of the complete bearing system. Fig. 1 show a conventional solution for round wood trusses which are manually fitted. Since the Olympic Games of Munich in 1972 it is well known in engineering of steel constructions, that framework jointing elements can be manufactured as steel casting elements. First applications where used for steel grids and membrane structures for tensional use.

Fig. 1 Round wood truss joint

Between 1990 and 2010 a lot of wide spanned roofs and bridges in steel with casting elements where realized. Fig. 2 show a detail and Fig. 3 the casting-joints of the Stuttgart Airport roof construction, which was built between 1981 and 1991 [1].

Fig. 2 Casting joint of Stuttgart Airport

Fig. 3 Stuttgart Airport roof construction

2. Objectives: Composite joints with force- and form optimized design

According to the described solutions in steel a new-type of high performed composite joints (HPTJ) for round wood-truss structures was created. The design of timber composite joints allows solutions in high performance concrete or high performance materials based on epoxy-resin. The mechanical values for standard products can be assumed for compression strength of 135-150 MPa, for flexural strength of 35-45 MPa. Different load-bearing capacities as compression, tension or shear force can be resisted by contact pressure or glued bars [2]. If required, a pressure ring between the contact surfaces with ductile behavior will resist also the small bending moments in the joints. In Fig. 4, a 3D-joint for spatial structures as a standard detail for mainly compression strength is shown, Fig. 5 show the function of usual load cases.

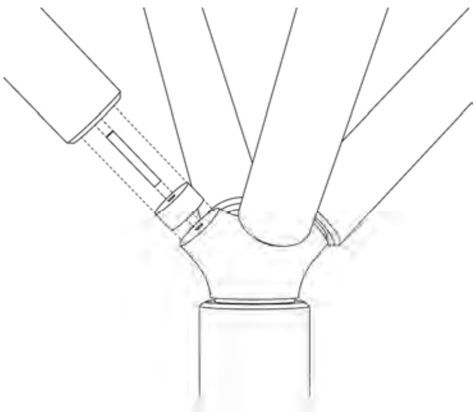

Fig. 4 3-D joint for spatial structures

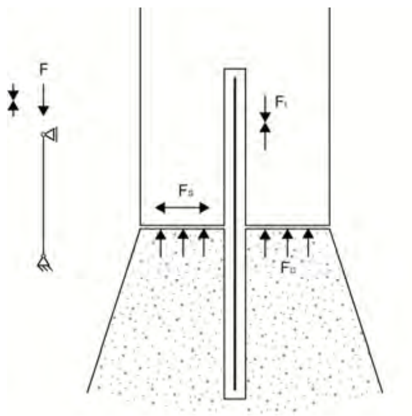

Fig. 5 function of usual load cases

Fig. 6 show a joint solution for outdoor use and also under bending loads. The ductile behavior of the joint is given with a pressure ring, used depending on the expected bending load. Fig. 7 show the mechanical model of this proposal. With regard to Fig. 6 the fastener can be fixed at the round wood diameter with a commercially available screw product.

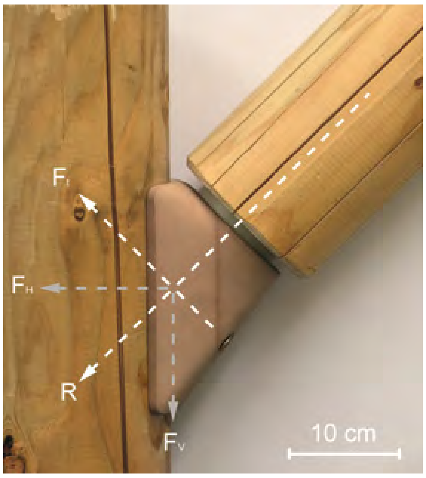

Fig. 6 Joint solution for bending load

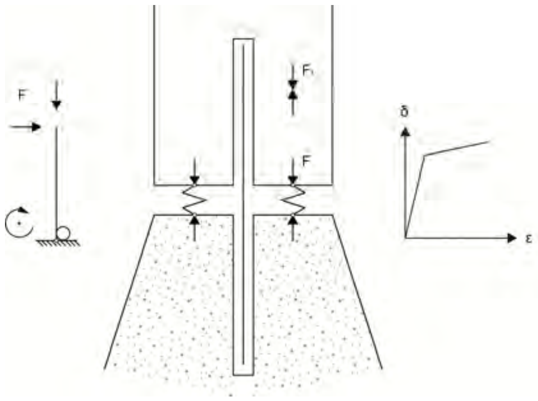

Fig. 7 Mechanical model with ductile behavior

Fig. 8 Concrete Joint

Fig. 9 Concrete Joint connected to timber

3. Work packages

The research focuses three main aspects:

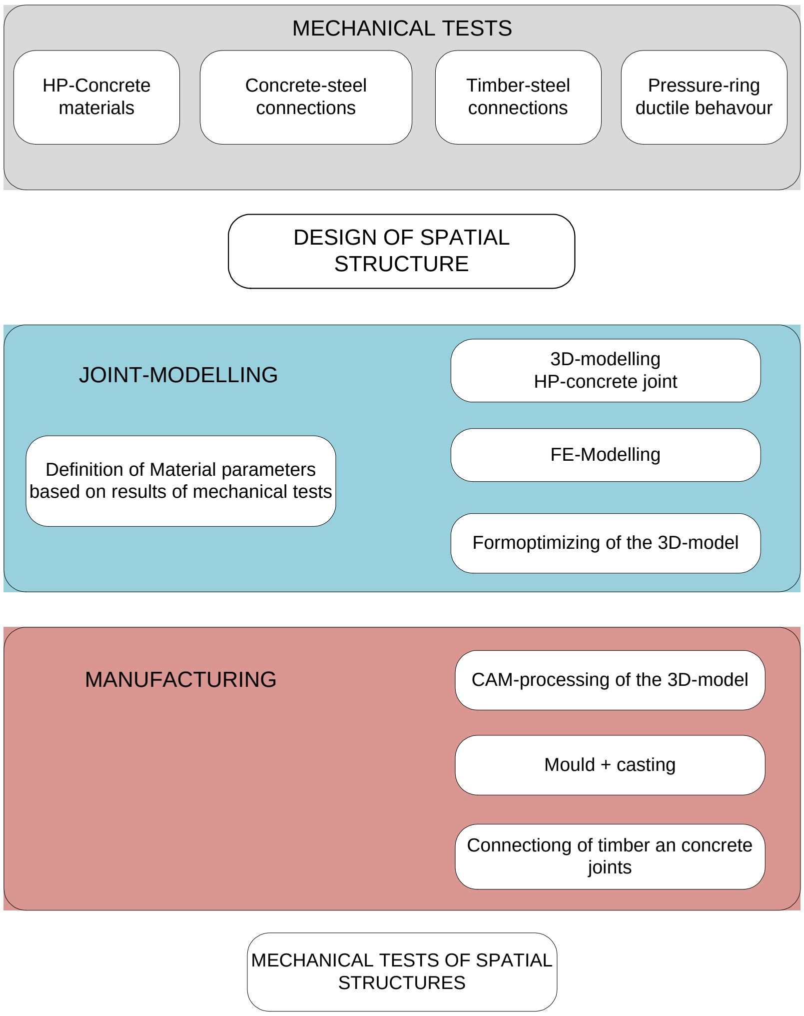

Fig. 10 Work packages

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Earthquake resistance of multi-storey timber buildings made of cross-laminated timber panels

Igor Gavric[1], Massimo Fragiacomo[2], Ario Ceccotti[3]

Summary

In a frame of a current PhD research project, started in 2010 with collaboration of University of Trieste and CNR-IVALSA research institute, an extensive experimental programme on typical X-Lam connections and wall panels has been conducted. The outcomes of these tests are being used to calibrate advanced component FE models for non-linear static and dynamic numerical analyses of X-Lam walls and buildings, as well as to evaluate the mechanical properties and calculate the ductility ratio, energy dissipation, impairment of strength, and behavior factor q, which are all needed in seismic design and are currently not provided by codes of practice such as the Eurocode 8. The overstrength factor, which is of great importance in capacity based design, was also evaluated for the different types of connection tested. With the newly developed analytical models for seismic behavior of X-Lam connections, walls and buildings, and the numerical parametric analyses carried out, a proposal for seismic design of X-Lam building system will be provided.

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1. Introduction

This paper discusses a continuation of SOFIE research project, whose aim is to develop seismic resistant multi-storey timber building system with use of prefabricated cross-laminated panels. SOFIE project started in 2005 with performance of racking tests on wall panels with different layouts of connections and openings [1] and pseudo-dynamic tests on a full scale one-storey building, continued with shaking table tests on a 3-storey building in 2006 [2] and on a 7-storey building in 2007, the latter one conducted at E-Defense facility in Miki, Japan. Experimental tests provided excellent outcomes, as the buildings were able to survive a series of strong recorded earthquakes, such as Kobe earthquake (1995), virtually undamaged, while at the same time demonstrating significant energy dissipation. However, further research in this field is still needed in order to better define the seismic behavior of typical X-Lam connections (1-D models), the behavior of single wall panels or series of adjacent wall panels (2-D models) and the behavior of entire X-Lam buildings (3-D models). An extended experimental programme on typical X-Lam connections was performed at IVALSA Trees and Timber Institute. In addition, cyclic tests were carried out on full-scale single and coupled cross-lam wall panels with different configurations and mechanical connectors subjected to lateral force.

Fig. 1 7-story SOFIE building

In this paper, some of the results and analyses from experimental tests are presented. The derivation of a simplified analytical model is discussed, together with an introduction on an advanced FE model that will be used to investigate the seismic performance of X-Lam buildings.

2. Experimental tests programme

In order to obtain statistically reliable values to be used for analytical and numerical analyses, at least one monotonic and six cyclic tests were performed for each of twenty different configurations of typical X-Lam connections, with an addition of three to five tests for each configuration with changed connection layout. More than 200 tests were performed in total, following the EN12512 standard. Shear and pull-out monotonic and cyclic tests were carried out on hold-downs and steel angle connectors used to anchor the wall panels to foundation and to connect wall panels to floor panels in upper stories. In-plane shear tests were also performed on mechanical screwed connections between adjacent X-Lam panels, using different types of vertical joints [3]. In addition, cyclic tests were carried out also on orthogonally connected panels (wall-wall and wall-floor) subjected to shear and withdrawal load.

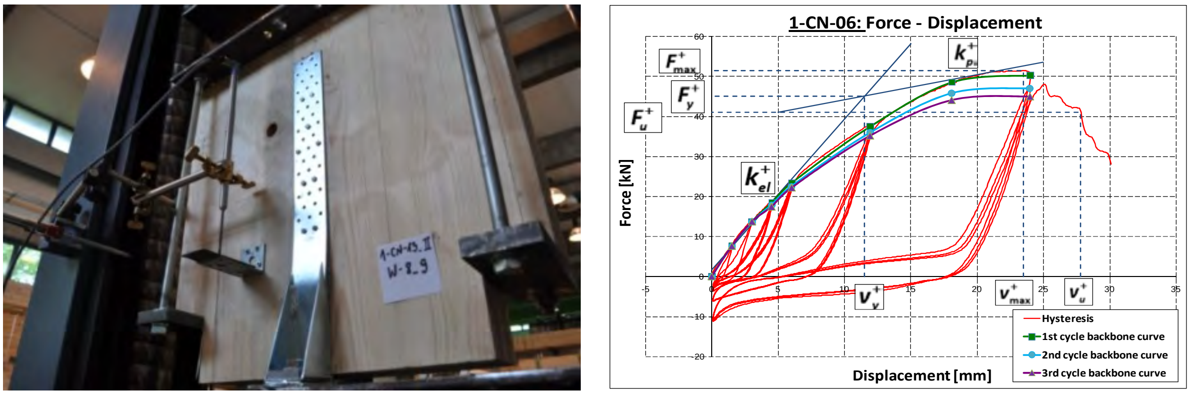

Fig. 2 Experimental setup of hold-down loaded in tension (left) and hysteresis loops of a hold-down loaded in tension (right)

In addition to monotonic and cyclic test programme conducted in CNR-IVALSA in 2005 [1], a series of cyclic tests were also carried out on full-scale single and coupled X-Lam wall panels with different configurations and mechanical connectors subjected to lateral force. Influence of vertical load, geometry of the panels, types of used fasteners, types of connections and types of connectors was studied. The aim of additional experimental tests on wall panels is to find out the differences in seismic behaviour between long single walls and a series of adjacent wall panels with vertically screwed joints. Analysis of seismic performance was done, with detailed investigation of energy dissipation properties and damping capacity of X-Lam timber panels.

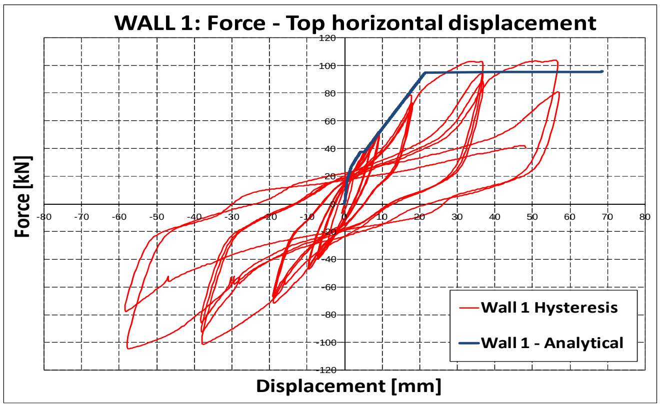

Fig. 3 Experimental setup of a single wall panel subjected to a horizontal load (left) and hysteresis loops of top horizontal displacement and analytical prediction of the backbone curve (right)

3. Analytical methods

Experimental values obtained from typical X-Lam connections will be used to calibrate analytical calculation methods of connection behaviour such as elastic stiffness, shear resistance, slip modulus etc. based on the formulas from current standard EC5 and on the extension of the Johansen equations from glulam to crosslam connections as proposed by Uibel and Blass [5]. If necessary, some adjustment or correction factors to the analytical formulas will be proposed in order to make sure the analytical design formulas are conservative. Overstrength factor, needed for capacity design of cross-laminated timber structures will be evaluated.

X-Lam wall panels are very rigid in comparison to the anchoring connections, so the most of the flexibility is concentrated in the connections. The vertical forces due to rocking of the panel are generally assumed to be resisted only by the hold-downs which are normally placed at the corners. The base shear forces due to horizontal slip of the panel are usually assumed to be taken by the angle brackets, and the vertical shear forces between adjacent wall panels are taken by the in-plane screwed joints. A new analytical method was developed which takes into account all the stiffness and strength components of hold-downs and angle brackets also in the weaker direction.

All walls in one storey in CLT construction contribute to the lateral and gravity resistance, thus providing a system effect. The effect of the perpendicular walls on the seismic performance of CLT walls will be studied. On a base of analytical models a comparison of different approaches will be discussed and the simplified design process will be proposed.

4. Numerical analyses

An advanced numerical model to describe accurately the hysteretic behaviour of typical X-Lam connections was developed at University of Trieste [4], and presented in detail in another paper [6]. The FE model with non-linear hysteretic springs schematizing the connector behaviour as obtained from the experimental tests will be calibrated on the results of the single and coupled wall tests first, and then on the outcomes of experimental full-scale tests carried out at IVALSA. A parametric study will be carried out using the FE model mentioned above to extend the results of the experimental tests to different configuration of technical interest, and to derive the behaviour factors *q*.

5. Conclusions

Several configurations of typical X-Lam connections and wall panels were investigated by means of cyclic tests, carried out according to EN 12512 standard. Wall configurations included different anchoring systems, single and coupled walls, and different types of screws to connect the panels. First part of analytical and numerical analyzes in comparison with experimental results has confirmed that the layout and design of the joints is critical for the overall behaviour of the X-Lam structural system. The axial stiffness and resistance of angle brackets are quite important and should not be neglected in the equilibrium of the wall under lateral forces [3].

6. Acknowledgments

The authors would like to acknowledge the contribution of laboratory staff from CNR-IVALSA for their help in the experimental tests. The PhD research project presented in this paper is sponsored by CNR-IVALSA, and University of Trieste, Italy. Special thanks also go to Dr. Marjan Popovski for hosting the author at FPInnovations research institute (Vancouver, Canada).

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A numerical model for hysteretic behavior of timber connections

Giovanni Rinaldin[1], C. Amadio[2], Massimo Fragiacomo[3]

Summary

In recent years a strong interest in wooden buildings has grown, especially in earthquake prone areas, where the main issue is to estimate in a conservative way the dissipative capacity of buildings. The aim of this project is to develop an effective model to characterize seismic behavior of metal connections in timber buildings. This is made possible by introducing several types of hysteresis loops in a non-linear finite element that can be used in static and dynamic analyses. Every hysteresis law requires a lot of parameters that can be obtained from experimental tests on a single connector. Several hysteretic behaviors are studied and parameterized; this will allow the user to change parameters needed for characterize the cycles. Finally, an automated calibration program is provided.

1. Introduction

Metal connectors are widely used in timber buildings: nails, bolts, dowels, angle brackets, hold-down and tie-down. In earthquake-prone areas, the dissipative capacity of this kind of buildings is demanded to the connections between the wooden elements, which act like a damper for the whole structures thanks to the slip-type behavior of the connections themselves.

The aim of this project is to develop a model that can be used to obtain an estimation of dissipated energy provided by metal connections, through a non-linear characterization of their hysteresis law. This is done with the software ABAQUS, a general finite element software package, where a non-linear spring is

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implemented. The spring works in plane between 2 nodes in the model and its hysteresis behavior can be set by the user through several parameters. This kind of parameterization allows the user to implement different types of connectors, from dowels to hold-downs. The wooden panels are modeled as linear-elastic.

2. Constitutive model of the springs

2.1 Model kinematic

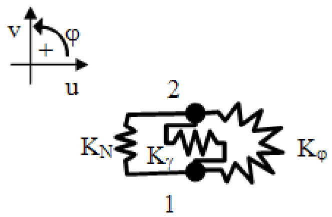


Fig. 1 Schematic of spring

The non-linear spring connects two coincident points in the undeformed state, hence it has zero length. In the most general case, every spring returns to the solver the three forces that develop in its plane and the corresponding three stiffnesses.

Only planar springs with three degrees of freedom have been considered in this study for the sake of simplicity as this is the most important case. However, the theory can be easily generalized to the case of a spatial spring with six degrees of freedom.

By referring to Figure 1, the congruence equations used for the spring are:

$$\varphi=\varphi_2-\varphi_1,\ \gamma=u_2-u_1,\ \varepsilon=v_2-v_1 \tag{1}$$

and the constitutive equations are:

$$M=M(\varphi),\ \ T=T(\gamma),\ \ N=N(\varepsilon) \tag{2}$$

with:

N = axial force;
T = shear force;
M = bending moment;
u = axial displacement;
γ = shear displacement (slip);
φ = relative rotation between the two connected nodes.

For every step of analysis, the spring returns stiffness and force values to the solver. Non-linear static and dynamic analyses of complicated structures can be easily performed. Finally, the spring rotational stiffness is always linear and it has been set to zero in all the analyses carried out in this paper.

2.1.1 Shear and axial hysteresis models

The hysteresis models under shear and axial forces are depicted in Figures 2a and 2b. The model for shear is symmetric, whereas the model for axial force is non-symmetric due to the contact in compression between the timber members.

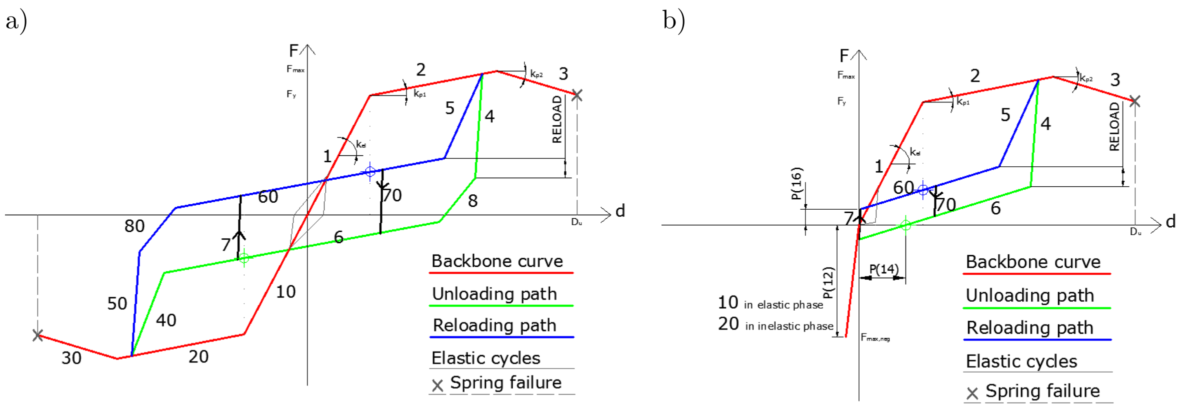


Fig. 2 Piecewise linear law of shear (a) and axial (b) component

2.1.2 Strength and stiffness degradation

Pinching effect, softening behaviour, stiffness and strength degradations have all been implemented in the model as they are important features of timber connections. A degradation of stiffness proportional to the maximum displacement attained during the load history has been assumed for the last unloading branches #5 and #50 (after the pinching effect) for both spring models. The strength degradation depends on the energy dissipated and on the maximum displacement attained during the load history. Due to the complexity of evaluating the dependence of the strength degradation on both these quantities, three calibration parameters have been introduced: a linear, an exponential, and a logarithmical one.

2.2 Calibration and analysis

The calibration is done by following the steps listed herein after:

- the yielding and peak force were extracted from the experimental results carried out on connections separately tested in shear and tension;
- the elastic stiffness was estimated once a good fit of the yielding displacement was obtained;
- the hardening stiffness of the plastic branch was chosen on the basis of the backbone curve;
- the other parameters, such as the stiffness and the strength degradation factors, were evaluated in an iterative way until a goof fit between the experimental curve and the model was obtained.

Figure 3 displays the calibration of the shear (left) and axial (right) non-linear springs on the experimental tests carried out on angle bracket and hold-down, respectively.

a) b)

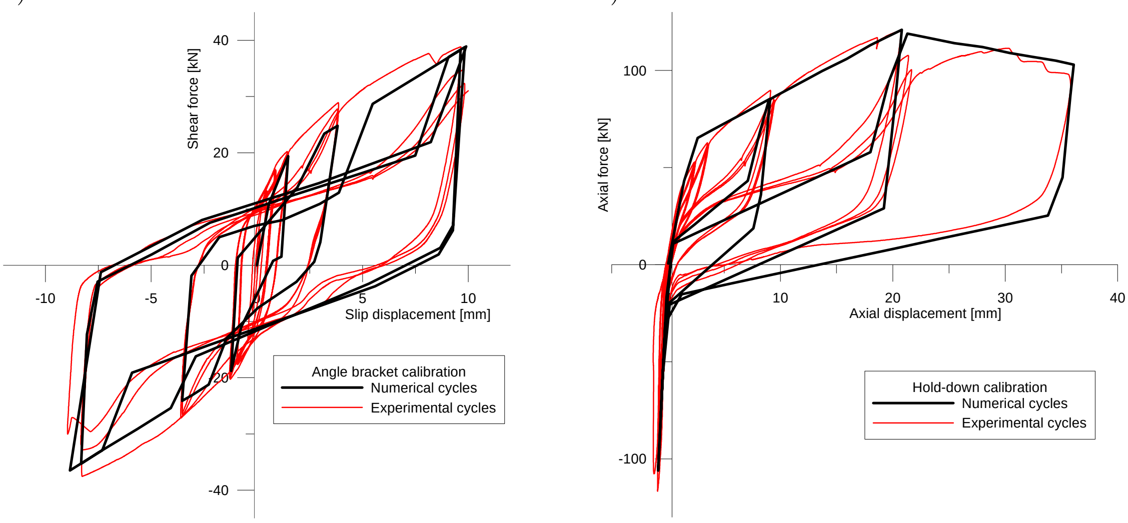
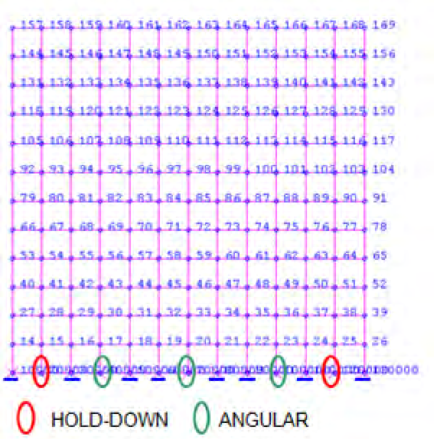


Fig. 3 Calibrations in shear (a) and axial (b) DOFs

A further and more definitive calibration will be possible only after a complete cyclic test programme on each individual component (hold-down in tension, angle bracket in shear, and panel-to-panel screwed connection in shear) will be completed. Once calibrated, springs are ready to be used in numerical simulations. Fig. 4a displays the mesh of a single X-lam panel connected to the foundation. The timber panel was modelled using 4 nodes elastic and orthotropic shell elements. A composite shell section formed by 5 layers of linear elastic wood material was used for the panel, so all the dissipation was assumed to occur in the connectors. These experimental results will also allow calibration of the strength degradation, which was ignored in this first comparison due to the lack of experimental data.

a)



b)

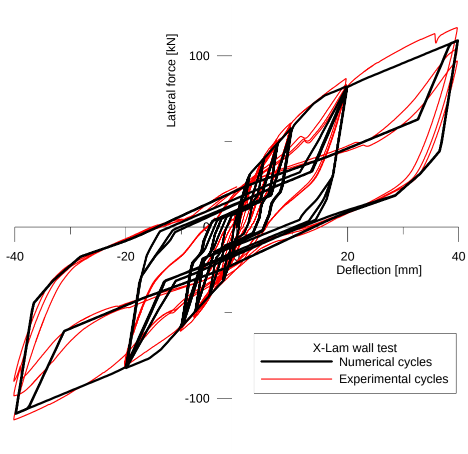


Fig. 4 Mesh and springs used for a X-lam wall model (a), and cyclic experimental-numerical comparison (b)

3. Conclusions

A component approach for non-linear dynamic analysis of structures made from X-lam solid timber walls was presented in this paper. This model was derived from results of cyclic tests available in literature for single X-lam panels and steel connectors such as hold-downs, angle brackets, and screwed connections between adjacent walls. A spring for every type of connectors has been modelled in a phenomenological way. Based on experimental evidence, all energy dissipation was assumed to take place in the steel connectors, whilst the timber panel was regarded as linear elastic. Two different hysteretic loops characterized by a tri-linear backbone curve with significant pinching effect were implemented in ABAQUS software package using external user-subroutines. Allowance for strength and stiffness degradation, based on the dissipated energy and maximum displacement attained, was also made. The proposed hysteretic model is very robust and unlike other software packages does not suffer from convergence problem. As such, it can be effectively used to model the seismic performance of cross-laminated buildings and subassemblies. The cyclic behaviour of the steel connectors allows a correct estimation of the dissipated energy and, consequently, a reliable prediction of the seismic capacity of the whole timber building.

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Density and stiffness-strength variations within Lithuanian-grown Scots pine tree stems

Antanas Baltrušaitis[1], Vilija Pranckevičienė[2]

Summary

The main aim of research will be to investigate spatial physical and mechanical properties in various age and diameter pine tree stems. Standard felling age and over-matured (up to 300 years) trees wood properties will be used as references. Research will be completed using two concepts. In the first models, density and stiffness-strength variation will be observed by the pith-to-bark and longitudinal direction estimating distance from pith to center of each specimen in millimeters. In the second approach, number of tree rings estimated in order to evaluate cambium age, eccentricity, and then density variation in radial pith-to-bark direction.

To explain testing procedures below is presented pilot research of density variation within 300 years old pine tree stem. The same approach used for ongoing testing of stiffness and strength. Specimens' properties taken from butt, middle, and top logs modeled in spatially polar coordinate system. Variation of results explained by numerical models and visualization in black and grey scale graphical models.

1. Introduction

The density of wood is impact of complex and long processes, based on genetic and environmental conditions that influence on variation of quantity of juvenile, matured and over matured wood in the tree. Significant variation of density begins from 25-60 year [1]. The wood density depends from not only moisture content, species, growing conditions, part in the stem etc. Exist opinions, that the density

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of the same species of tree stem in some cases can differ by 10–20%. In most species of wood the biggest density is in the bottom of the stem and towards to top the density decreases by about 1.5% per stem height meter. At Lithuanian conditions when coniferous species reach approximately one hundred years age the average density develop tendency to diminish [2].

Formation of matured wood begins only when tree attain specific age. Therefore, quantity of maturity wood is related to the age but also there is pronounced accent of negative correlation with tree growing speed. Formation of mature wood is such phase-mode changes, when tissue with live cells experience transition to the substance with nearly no metabolism [3].

Heartwood of pine forms after 30-35 years of growing. The widest sapwood of pine is in the bottom of stem; from bottom to the top it becomes narrower but sapwood percentage in cross-section area enlarges. Variation of density toward bark is direct result of the cambium maturing. Over-matured trees often expose observable density reduction close to the bark when measured at DBH and at 60%-70% relative stem height [4].

In the cross-section in radial direction from bark towards pith, the differences in density can be from 5% until 20% or more. From anatomic aspect, juvenile wood may be described by stepwise variation of dimensions and changes in form, structure and cells distribution in annual rings respectively. No precise number how much annual rings consist in that wood exists but most authors indicate first twenty annual rings. Normally some quantities of annual rings surrounding heartwood and having worst mechanical properties are indicated.

In some cases over-matured wood have linear density decrease toward bark at 50% of relative stem height and higher. One of the explanations is that extractive materials in pine mature wood twice exceeding that in the sapwood. The highest quantity of extractive materials in the mature cross-section part is in the butt log, where extractive materials distribute almost equally throughout all section length [4].

As can be find in some literature sources decreasing of basic density from bark towards heartwood in radial direction is most visible in fast-growing trees. Conversely, no such features tendencies observed in the slowly and at dense sites growing trees. It can be find narrower rings near the heartwood and at the same time higher densities compared to the sapwood rings together with the ring widening and density decreasing towards the sapwood [5].

Radial density variation in the stem usually is determined in to ways. When cambium age is evaluated, annual rings from pith towards bark is specified. This in turn impose that specific coordinates are not related to the same calendar year because upwards from bottom annual rings are formatted at different periods. Another way is to calculate annual rings from bark toward wood pith. It means that at different stem heights annual rings were formatted the same year [6].

2. Materials and methods

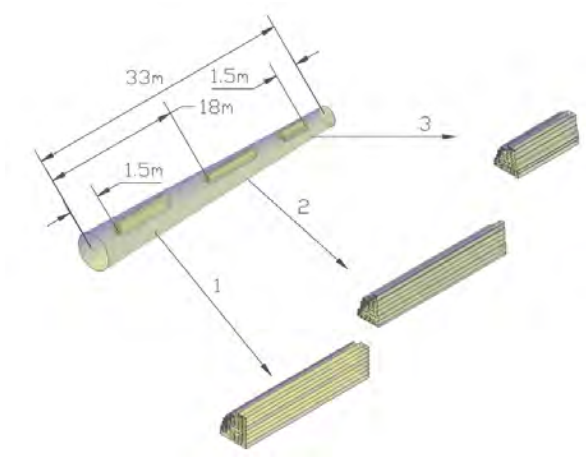

Fig. 1 Pine stem cutting scheme: 1 – bottom part samples, 2 – middle part samples, 3 – top end part samples

Wood samples cut from over-matured pine (*Pinus silvestris*) and natural matured stem bottom, middle and top parts used for testing (Fig.1). Pine tree sections selected at different stem heights. Logs were cut from stem and from each log one-quarter parts were cut. Bottom part dimensions were (2240×700×610) mm, middle part dimensions were (2290×500×420) mm and top end part dimensions were (1149×410×400) mm respectively.

Samples were split into smaller parts from outside (sapwood) towards heartwood direction gradually (Fig.1). Specimens were cut systematic diminishing their sizes starting from sapwood. Finally, 65×60 mm cross-section wood specimens were received. For investigation of variation of density in radial direction, two methods were used: calculating quantity of annual rings (cambium age) and measuring distance from specimen center towards the pith.

One –quarter section density variation was modeled considering sawing kerf and at each splitting stage samples were measured with the accuracy (±1 mm). In theoretical model either distance from pith to the specimen center (mm), was fixed drawing lines in polar coordinate system and cambium age was calculated.

Cambium age was fixed at the center of every specimen. Graphical model of the samples, finite specimen dimensions and physical groups of samples are presented in Fig 2.

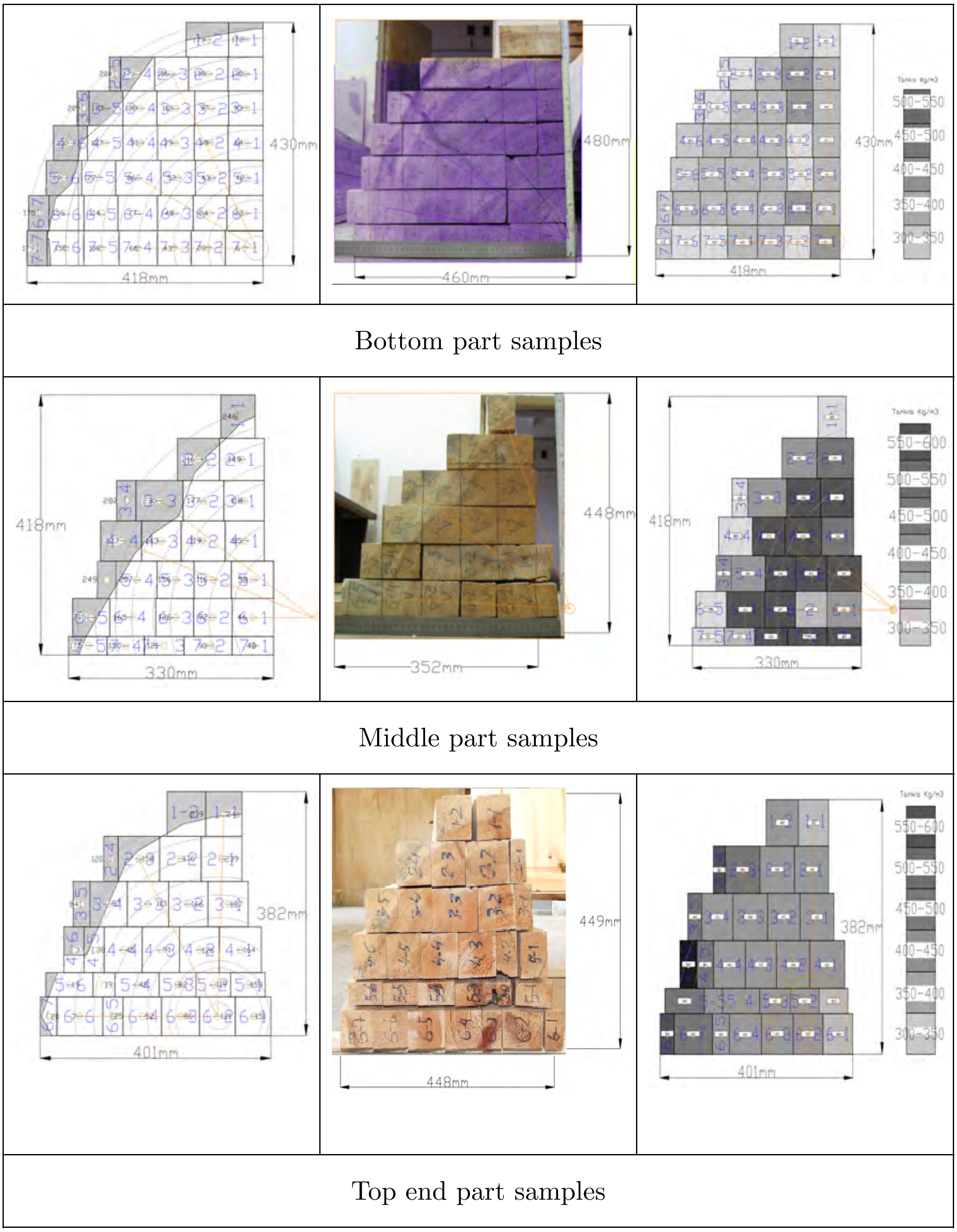

Fig.2. Schemes of the quarter-section samples

From theoretical model we can see, that average cambium age is about 250 year. The basic density of the samples is marked in grey-color pallet. Same approach will be used for modeling MOE-MOR. The tendency o increasing of the density towards bark was observed only in top end quarter-section sample group. Another sample groups show lower variation of the density (but-end quarter-section sample group), or considerably higher density in the heartwood central part, what contradict with the usual result findings in the literature [5, 8]. The ratio of early wood and latewood within annual ring depends on wood species and factors influencing width of annual rings (tree age, climate, growing conditions).

In the literature, the most commonly used method is cutting disks from trunk and after that splitting into the small-scale specimens [8]. The main task of our experiments will be to ascertain structural specimen sizes to estimate density variation and real influence on stiffness-strength wood properties alongside the tree system.

3. Conclusions

- The density values of middle part of tree stem part are biggest from all investigated quarter-sections and their specimens groups. The density of the over matured pinewood is lower near the bark, except of values of top end quarter-section specimens group.
- Decreasing of the density from the butt of the stem towards top end is not uniformly coherent: in the middle section of the stem density markedly increases and then again diminishes towards top-end.
- Distance from the pith to the specimen's center explains density variations better than heartwood specimen cambium age.
- Best-fitted modeling of the density variations received integrating cambium age, distance from pith and average of annual rings per cm at specimen centre.

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Timber - Concrete - Steel Composite Slab System

L. Calado, J. Proenca[1], R. Goncalves[2], K. Gramatikov[3]

Summary

The behavior of timber buildings when subjected to earthquake can be improved by introducing a slab concrete, which increases the in-plane rigidity and keeps the shape of the building. The experimental tests performed as a part of comprehensive experimental PROHITECH project activities at Instituto Superior Tecnico, Lisbon, concern a system which allows the connection between ancient timber beams and concrete slab using special device. The device is conceived allowing reversibility of the system. Evaluation of the structural behavior of the composite slab system was estimated based on performed push-out tests on parts of timber beam strengthened by steel device, without and with concrete slab. Six full scale beams were tested for studying the connection between timber and concrete under bending loading.

1. Introduction

The experimental campaign concerns an innovative connection system for composite timber-steel-concrete floors as shown on Fig.1. This connection system is purposely conceived with the twofold aim to realize local strengthening of ancient beams and to allow the stiffening of the existent floors by means of introducing a collaborating concrete slab. The device comprises two separate parts assembled by bolts, two connectors welded to the main part, steel stiffeners and, when necessary, rubber strips.

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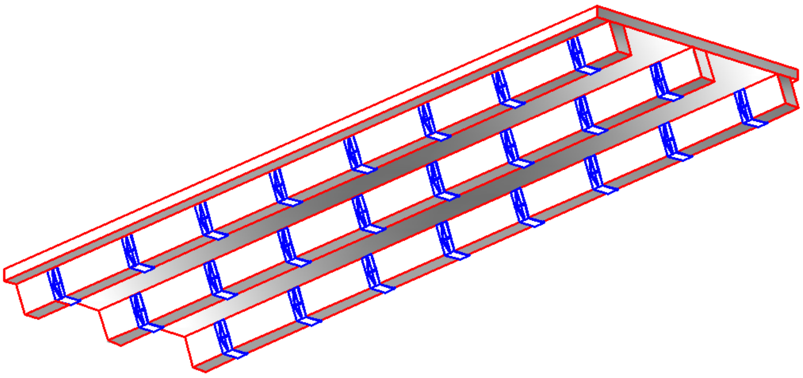

Fig.1 Composite timber-steel-concrete slab system

In order to analyze this system, push-out tests without concrete slab, push-out tests with concrete slab and beam tests with concrete slab were performed. performed.

2. Performed experimental activities

2.1 Push-out tests on specimens without concrete slab

The basic shape of the device has been shown on Fig.2

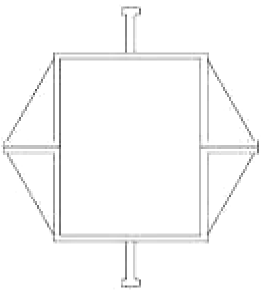

Fig. 2 Steel device

Several full scale types of devices were tested in order to determine the resistance of the specimen. The differences between the devices were related to the type of device surface, number of bolts connecting the two parts of the device, the angle of stiffeners, the pre-stress applied to the bolts and loading protocols (monotonic or cyclic). (Fig. 3, Fig. 4)

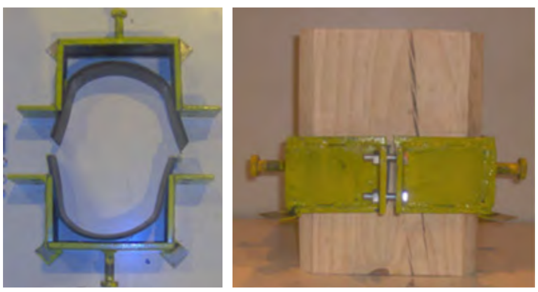

Fig. 3 Device without stiffeners

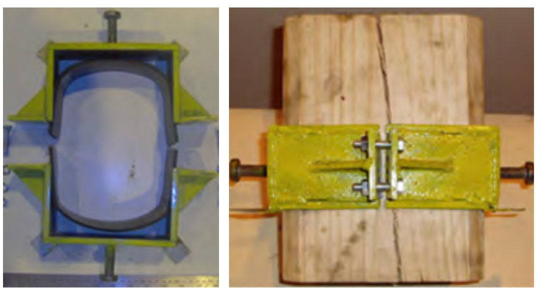

Fig. 4 Device with stiffeners

Ten types of connectors, without or with rubber were tested under monotonic loading, while 2 types were subjected to cyclic loading. The loading machine used was INSTRON mod.1343, with maximum compression and tension force of 250 KN, with a capability of controlling the applied force or displacement, as well as of applying cyclic loads. Transducers for measuring relative displacements between the timber and the device were used, too. These results allowed determination of the average of relative displacement and its graphical presentation for both, monotonic and cyclic testing. (Fig. 5, Fig. 6)

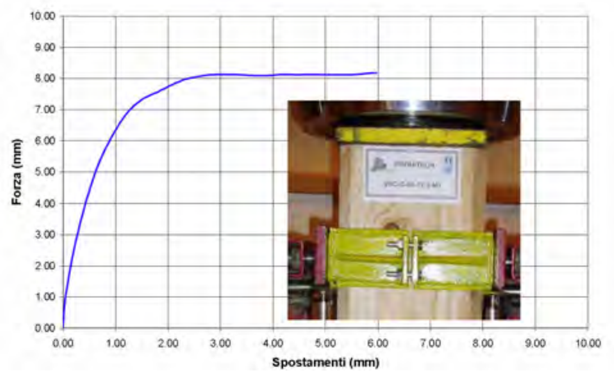

Fig. 5 Monotonic load test

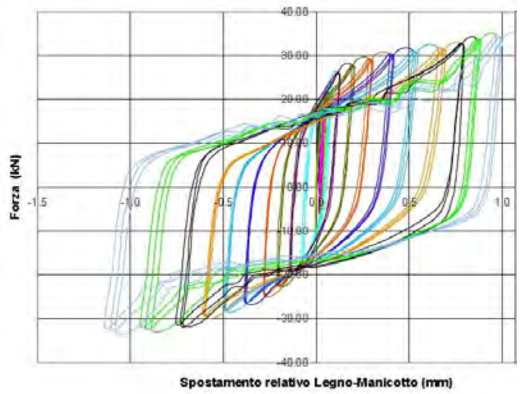

Fig. 6 Cyclic load test

2.2 Push-out tests on specimens with concrete slab

The push-out tests allow studying the connection between steel devices and timber or concrete. To provide this, full scale specimens were assembled.

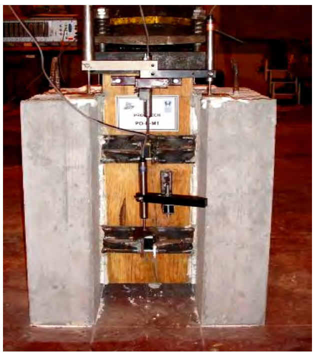

Fig. 7 Assembled specimen

Each specimen (Fig. 7) comprised a small timber beam, two concrete reinforcement slabs and two devices connecting timber and concrete. A total number of 52 tests was performed. Small timber beam measures 0.20 x 0.16 x 0.60 m, and the concrete slab measure 0.14 x 0.40 x 0.60 m. Used materials were the same as at previous test, with concrete class C30/37. Devices were both, with, or without inner side rubber.

Besides testing on specimens at dry condition, some of tests were performed considering a presence of water, by specimens remained in chamber for 4 days and watered during 1 minute each 5 minutes. Equipment used during these tests is the same with all other performed experiments. Results from tests will be shown both with beam test results.

2.3 Beam tests

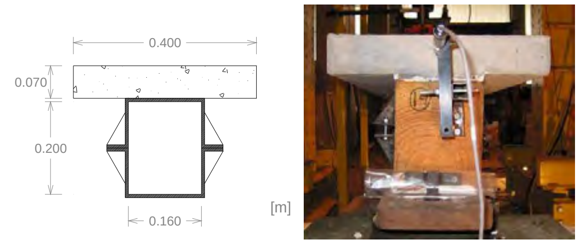

Fig. 8 Cross section of the beams

The beam tests allowed studying the connection between timber and concrete using the steel devices. To accomplish this, 6 full scale specimens were assembled. Each specimen comprised a timber beam (0.20x0.16x4.40 m), a concrete reinforced slab (0.07x0.40x4.40 m, class C30/37) and eight devices.(Fig. 8, Fig. 9, Fig. 10) The distance between the eight devices can be constant or variable. The steel devices used had rubber or a steel rough surface between the timber and the device. Test of material properties were also performed before beam tests.

Fig. 9 Devices with constant spacing

Fig. 10 Devices with variable spacing

The experimental results were obtained from each test and each transducer. Transducers were measuring relative displacements between timber and concrete, timber and devices, as well as deflection of the beams. Here will be shown just some of characteristic load – displacement graphs from performed testing.

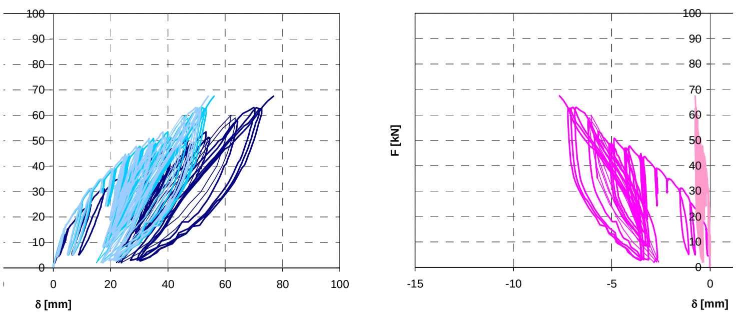

Fig. 11 Load vs. displacement for different transducers

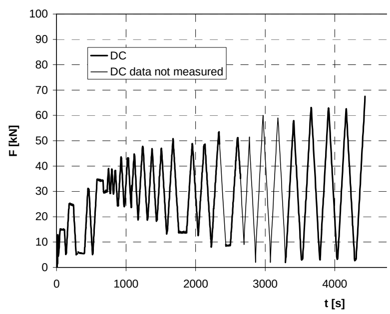

Fig. 12 Load vs. time

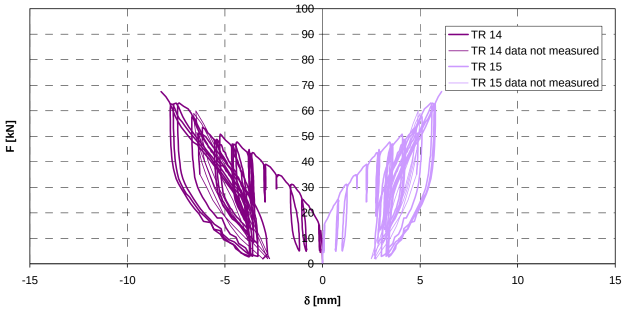

Fig. 13 Load vs. displacement for different transducers

Fig. 14 Collapse of the beam

The experimental results were also compared in order to evaluate the role of the device inner surface, the different spacing between the devices and use of bolts connecting steel and timber.

3. Discussion of the results and further work

To compare all test results, the envelope of each force – mid span deflection was sketched, from which several variables were derived, such as: stiffness in linear and non linear range, equivalent yield load, equivalent yield displacement, comparison of ultimate load and ultimate displacement, allowing comparison among the tests. Accumulated dissipated energy at failure was obtained, too.

Further experimental activities related to the long term behaviour of timber-steel-concrete system under constant bending load have been predicted at the Civil Engineering faculty in Skopje, MK.

4. Acknowledgement

The results presented in this summary paper have been obtained from the performed experimental activities of FP6 INCO Project PROHITECH, financed by European Commission. The Leader of the performed project experimental activities was Prof. Kiril Gramatikov.

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Wood and wood products characterisation from full-field measurements

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Summary

In this project, novel mechanical test methods are proposed for the characterisation of mechanical properties of wood and wood products. The focus has been given to pine maritime at both macro and meso (growth ring) scales, and MDF panels with recycled fibres. The approach couples inverse identification methods (*e.g.* the virtual fields method) with full-field measurements provided by suitable optical methods (*e.g.* the digital image correlation and the grid method). An overview of the ongoing work includes: (*i*) multi-parameter identification from single test configurations aiming to overcome cost and time demands of standard test methods; (*ii*) assessing wood quality by spatial variability of elastic properties within and among trees.

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1. Introduction

The parameters governing constitutive equations of materials are determined experimentally by means of suitable mechanical tests. In the field of solid mechanics, this issue is presented as an inverse problem where the material parameters are to be determined from the knowledge of geometry, boundary conditions and strains (or displacements). Conventionally, this identification is achieved by carrying out mechanical tests in which specimen geometry and loading system are designed to generate homogeneous or simple strain/stress states across the gauge region. The idea behind this assumption is useful for theoretical analyses because a closed-form solution can be deduced, relating the unknown material parameters to the load and strain measurements (statically determined tests). However, the practical implementation of these tests can be difficult, especially for anisotropic and heterogeneous materials like wood. The recent development of full-field optical techniques has enabled a new glance on the mechanical tests for material characterisation [1]. The basic idea driving this new approach is that a single specimen can be loaded in order that several parameters are involved in the mechanical response, yielding heterogeneous and complex strain fields (statically undetermined tests). By means of a suitable identification strategy all the active parameters can be determined afterwards.

In this project this identification approach is applied to the mechanical characterisation of wood and wood products based on full-field measurements.

2. Characterisation of MDF panels by deflectometry

In the last decades, developments on both manufacturing process and wood adhesives have led to new engineering wood products (*e.g.*, veneer, particle or fibre based products). These materials have been designed to fulfil specific structural requirements and have been emerging in different applications. An important class of these wood products is the Medium Density Fibreboard (MDF). Conventionally, MDF panels are produced from wood-based raw material by transforming wood chips into fibres (refining system) and gluing them with a resin binder by applying high temperature and pressure. However, alternative sources of lignocellulose fibres in the MDF manufacturing process have been explored, namely by using agricultural and forestry waste, in a policy of better preservation and management of the available natural resources. These MDF products require specific mechanical characterisation. Standard test methods for mechanical characterisation of wood products only specify the determination of the modulus of elasticity and modulus of rupture by three-point bending tests. In order to overcome this limitation, a plate bending test is proposed for direct identification of the whole set of bending stiffness components of MDF panels [2]. The test method consists in gripping the plate at several points at the edges and applying a point load at a given location on the plate (Fig. 1a). The approach couples the virtual field method with full-field measurements provided by the deflectometry technique (Fig. 1b). An experimental procedure was validated for transferring a reflective coating to the MDF panels required in this technique. An extension of this approach to solid wood is under current evaluation.

(a) (b)

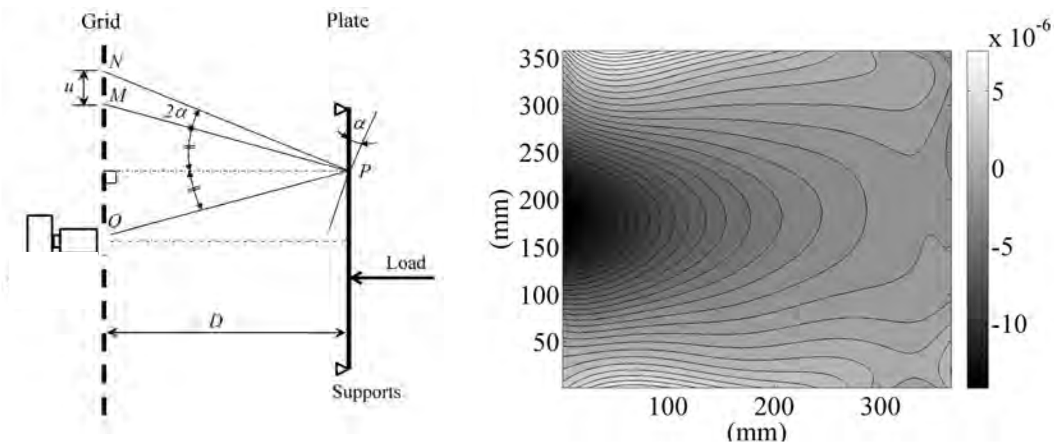

Fig. 1 (a) Heterogeneous plate bending test; (b) $K_{11}(x_1,x_2)$ curvature field (unit:mm^{-1}).

3. Assessing wood quality from gradient properties within the tree

Maritime pine accounts for about 30% of the forestland in Portugal, corresponding to a volume of raw material of about 86×10^6 m^3. This species has been used as firewood, pulpwood, pallet wood, and for boat building, construction, fencing and furniture making. However, despite its relative abundance and variety of applications, this important natural resource has not been efficiently used because there are still wood quality assessment problems. Namely, there is a lack of forestry programs in the majority of plantations which mostly belongs to smallholders and absence of quality grading parameters. Wood quality is defined in terms of attributes that make it valuable for a given end use. Density has been considered a suitable parameter to define wood quality. However for some end uses it does not represent the quality grades. For instance, for structural applications, it is more adequate to have a grade of wood quality based in terms of strength and stiffness. In this work, both density and stiffness will be analysed as indices for assessing wood quality of maritime pine for structural timber. The transverse elastic properties of both earlywood and latewood and their spatial variability are determined among five trees, aged from 60 to 68 year-old, harvested in Portugal central region [3-4]. X-ray micro-densitometry measurements are carried out to assess the local density of wood, as well as the respective dimensions and fractions of the earlywood and the latewood layers within the growth rings. Tensile tests at the growth ring scale are carried out. Specimens with nominal dimensions of 50(R)x5(T)x2(L) (mm) are tested on an Instron 5848 Microtester machine under a displacement rate of 0.2 mm/min. These tests are coupled with digital image correlation for assessing strain fields across the region of interest (Fig. 2). Images are recorded by means of an 8-bit Baumer Optronic FWX20 camera (1624x1236 pixels, pixel size of 4.4 µm) coupled with a telecentric lens TC 2309. With this study, it is intended to develop methodologies for quantifying the variability of maritime pine wood within the stem. Assessing this spatial information can be of major importance for wood modelling and end-user applications.

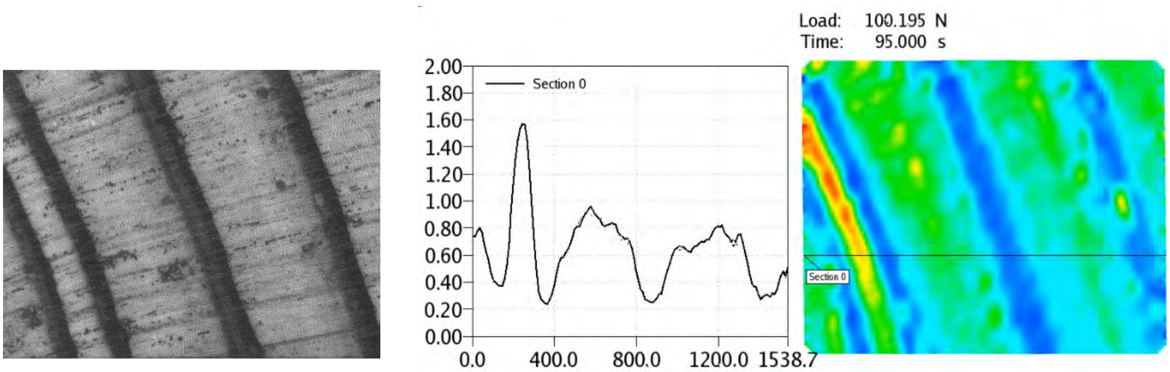

Fig. 2 Strain field along the radial direction ar the growth ring scale.

4. Determining stiffness components of clear wood from stereovision

Wood is a biological composite material formed by trees. It can be analysed at several scales of observation from timber down to chemical constituents. The mechanisms of deformation in wood can be quite complex involving, for instance, anisotropic, viscoelastic and hygroscopic phenomena. Moreover, the intra and inter variability of wood is reflected on the material parameters governing relevant constitutive equations. Therefore, the investigation of the wood mechanical behaviour raises several difficulties from both modelling and experimental points of view. In most practical applications and with some simplification hypothesis, invoking low levels of stress, short periods of time and minor variations of moisture content and temperature, wood can be modelled as a linear elastic anisotropic material. Besides, at the macro scale (0.1-1 m) wood is usually assumed as a continuum and homogeneous medium. The complete characterisation of the linear elastic orthotropic behaviour of clear wood requires the determination of nine independent stiffness components. Conventionally, this set of material parameters are determined experimentally by carrying out several test methods, in which both loading and specimen geometry are usually oriented along the material directions. Moreover, these tests are based on the assumption of simple and homogeneous stress/strain states across the elementary representative volume of the material at the scale of observation. This approach represents a great effort from an experimental point of view because only a few (*i.e.*, one or two) stiffness components are obtained per test configuration. Besides, the complete stiffness matrix will be characterised from different test and specimen configurations, enhancing variability. In order to overcome these limitations, a single off-axes compression test method for clear wood was recently proposed by Majano-Majano et al [5]. This task aims further improvements of the test method with regard to the identifiability of the whole set of orthotropic stiffness components [6].

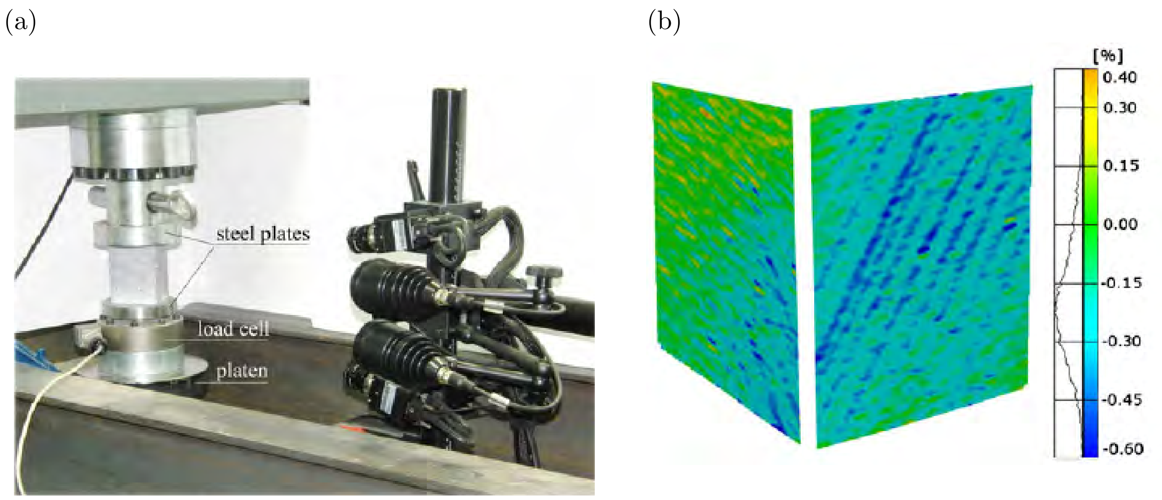

Fig. 3 (a) experimental set-up (b) results of ε_y in two faces of the specimen.

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Ductile and brittle quasi-static mechanical behaviour of dowel-type wooden joints

Nuno Dourado[1], Abílio de Jesus[2], José Morais[3], Marcelo Moura[4], Cristovão Santos[5], José Xavier[6], Stéphane Morel, Jean-Luc Coureau[7]

Summary

This project aims at investigating the quasi-static mechanical behaviour of dowel-type timber joints. In particular, the brittle and ductile behaviour is to be investigated by means of advanced finite element modelling and experimental validation. An approach based on inelastic constitutive modelling (e.g. plasticity models) will be adopted to describe the ductile behaviour, as well as a cohesive damage modelling strategy to replicate brittle failure modes. Several dowel-type joints will be investigated, including moment-carrying joints.

1. Introduction

Resistance and durability of timber structures is mainly dependent on the design of joints, which are assumed to be the weakest points in these structures. It has been recognized that approximately 80% of all failures in timber structures initiate at connections [1]. In fact, the loss of perfect continuity in structures, which is caused by the presence of joints, will result in a reduction on the global strength, leading to an increase of the dimensions of wood members. The singularity of timber joints is mainly attributed to the highly orthotropic nature of wood with dissimilar tension and compression behaviours.

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Dowel-type connections are the main fastening technique used worldwide in timber structures. Dowel-type wood connections have been modelled based on the analytical Johansen's model [2], which is the basis of current design code procedures (e.g. Eurocode 5 [3]). Effectively, the Johansen's model only predicts ductile failure loads (plastic bending of the dowel or crushing of the wood beneath the dowel); brittle failure observed in single and multi-fastener connections (e.g., shear splitting, block shear, plug shear, normal to grain cracking) are not foreseen by this model. This limitation represents a major drawback, since brittle failures usually lead to catastrophic collapses. To avoid these brittle failure modes, code procedures, based on empirical basis, propose minimum spacing between fasteners as well as minimum dowel-member end or dowel-member side distances [3].

It is recognized that advanced numerical models are necessary to model conveniently the whole load-displacement curves of the joints, including both brittle and ductile failure modes. These models have to account for local contact stresses between fasteners and wood members, interaction effects between multiple fasteners, the three-dimensional nature of the problem, among other parameters. Therefore, three-dimensional finite element modelling appears as an appropriate choice, as long as non-linear stress-strain constitutive modelling is provided together with damage modelling. The authors of this proposal have recently followed this approach. Thus, an overview of the on-going research and future targets is given in the next sections.

2. Experimental program of quasi-static tests of doweled wood joints

Quasi-static tests have been performed in single and multiple dowel joints, in order to better understand the mechanical behaviour of this type of connections, including the characterization of typical failure modes occurring in wood (*Pinus pinaster* Ait.). In the following a list of performed tests is presented: i) embedding tests (perpendicular and parallel to grain) according to the EN383 and ASTM D5764 standard [4]; ii) double-shear single dowel wood connections loaded along grain direction [5]; iii) single dowel double shear T-joint [6]; iv) multiple dowel moment-carrying joints [7] (see Figure 1 for specimens illustrations). Depending on load direction with respect to the grain direction, brittle failure modes have been verified which are, in some cases, preceded by some ductile behaviour. Therefore, a unified modelling approach capable of modelling both ductile and brittle behaviours is required.

Some work has also been performed on the proposal of strengthening solutions aiming both the enhancement of strength and ductility of dowel-type joints. In fact, the application of a new technique based on glued metallic inserts has shown to be effective on the increase of strength. The use of local CFRP reinforcements proved to be effective on eliminating early brittle failures (quasi-brittle) [6].

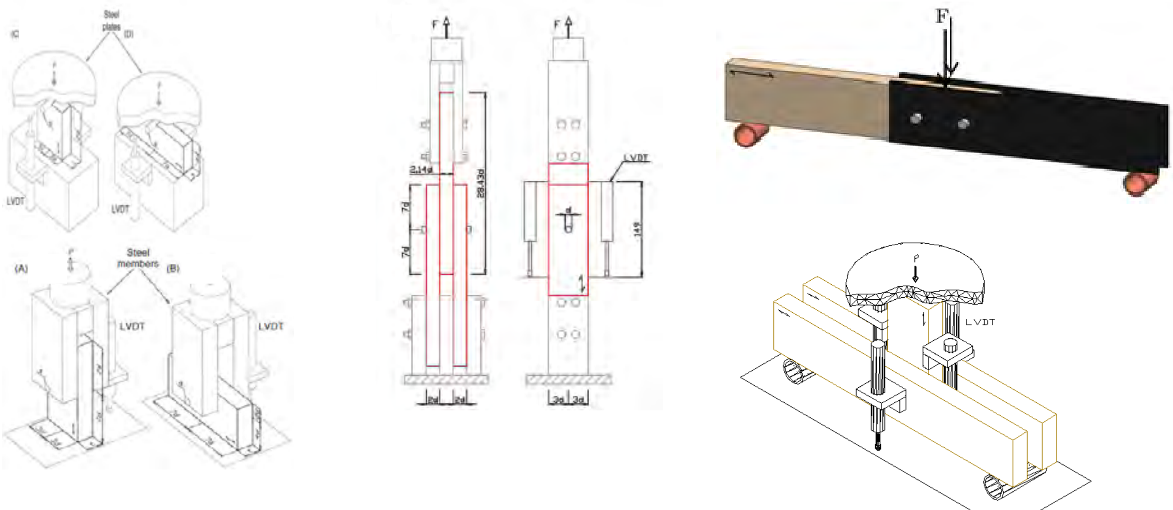

Fig. 1 Experimental program on doweled joints.

3. Modelling brittle to ductile behaviour of dowel type joints

Three dimensional finite element models have been proposed by the authors to simulate the load -displacement behaviour of several doweled joints object of experimental analysis (see Figure 2). These models take into account the contact between dowels and joint members. Orthotropic behaviour of wood was assumed using experimental data derived for the same wood species used in the joints. So far, the initial stiffness of the joints has been modelled with success considering elastic behaviour of wood members [6]. Concerning the modelling of ductile behaviour of dowel joints, plasticity models have been tested, in particular the generalized Hill's plasticity model which allows the definition of distinct tensile and compressive behaviours for wood. While the tensile behaviour may be assumed as quasi-linear until failure, the compressive behaviour is characterized by significant ductility. This task is an on-going research issue since it requires a convenient non-linear characterization of *Pinus pinaster* Ait., which is in progress.

Regarding the brittle failure modes, a modelling approach based on cohesive zone modelling was selected and preliminary works on doweled joints are in progress (see illustration of Figure 3).

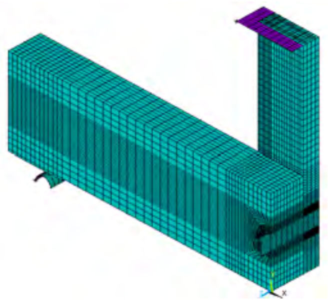

Fig. 2 Finite element model of a single dowel T-joint.

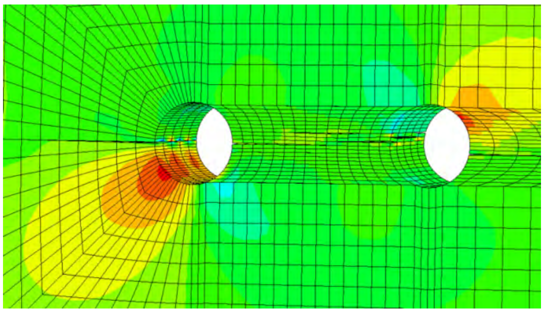

Fig. 3 Cracking at ligament between dowels.

In this model, interface finite elements were placed at locations where cracks are expected to initiate and propagate. Multiple layers of interface finite elements were defined in wood regions that are likely to undergo damage initiation and (crack) propagation. This approach undoubtedly beneficiates of the significant experience of the research group on fracture characterization of *Pinus pinaster* Ait., based on both experimental and numerical (cohesive damage modelling) work [8-13]. The experimental validation of cohesive zone modelling, the verification of rupture modes and the influence of the material heterogeneity on the joint behaviour are also envisaged using the Digital Image Correlation technique [14]. Since the experimental work performed on doweled joints showed ductility prior to brittle failure for some configurations, it is expected in future work to use a non-linear constitutive model for wood (e.g. "plasticity models"), together with cohesive zone modelling. This model will allow characterizing the ductile behaviour, as well as the brittle failure of wood due to shear cracking in the RL fracture system, and perpendicular to grain cracking (pure or mixed (I+II) mode cracking). Furthermore, the experimental praxis has shown that brittle failure of doweled joints is strongly influenced by local heterogeneities of wood at the vicinity of the dowels. In particular, the scatter of results (i.e., stiffness and strength) is much more influenced by these local heterogeneities than by any other factor, which ought to be taken into account in the cohesive zone modelling. Therefore, the position of interface finite elements in the numerical model should be made as to mimic local wood heterogeneities, by introducing local dimensions of early and late wood. The numerical modelling should comprise the natural scatter in fracture properties that exist in these wood domains.

4. Conclusions

Due to recognized limitations of actual design procedures for dowel-type joints, in particular concerning the brittle behaviour modelling, an advanced numerical tool has been proposed based on non-linear constitutive modelling of wood with cohesive damage interfaces. This approach has been applied to dowel wooden joints, with the preliminary results showing its capacity to accurately describe the available experimental results. Future works will consolidate the on-going research and look for the unified modelling of both ductile and brittle failure modes, as well as, investigate the influence of local heterogeneities on mechanical behaviour of doweled wooden joints.

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Influence of delamination and fissures on bending strength

Florindo Gaspar[1], Helena Cruz[2]

Summary

This project aimed to assess natural (outdoor exposure) and artificial ageing of glued laminated timber beams, study the delamination progress, the use/improvement of non-destructive techniques for the *in-situ* detection of ageing or of failure of the glue lines and their influence on glulam beams mechanical properties.

It also investigated the optimization of glulam production made from preservative treated pine (*Pinus pinaster* Ait) aiming to minimize delamination.

In order to gain a more objective and wide knowledge about the importance of delamination relative to its type and extension, a numerical study was developed. Finite element modeling (FEM) was therefore used to evaluate the delamination influence (near the surface on the vertical faces, or ends) on the mechanic performance of straight glued laminated timber beams. The FEM was validated by comparing stresses and deformations obtained with the model and with the application of the beam theory, showing satisfactory results.

Further numerical modeling will also be developed and validated by experimental tests on full scale beams with delamination.

Not only the outcome of this study helps understanding the importance of the addressed delamination problem, but it may also contribute to discuss the importance of drying fissures, both in glued laminated timber and solid timber members.

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1. Motivation

Degradation of bonded joints may not be detected with a simple visual inspection, not only because of its often microscopic nature, but also because visual access to the degraded area may not be possible. However, besides visual inspection, no other reliable methods are yet available to assess their structural integrity in service, stressing the importance to better understand the glue lines degradation process: why they occur, how to detect them and how to quantify the associated loss of strength.

In the scope of a PhD program the glue lines' degradation due to ageing was investigated, as well as some methods to evaluate their performance in glued laminated timber in service. Several types of glulam were exposed, for different periods of time, to natural outdoor environment, as well as to artificial weathering.

Besides the visual appearance of the aged beams, the glue lines' degradation due to ageing was assessed through the extraction and testing of several types of core and block shear specimens, and delamination tests. The use of Near Infrared (NIR) Spectroscopy was also investigated. Tested material included glued laminated timber beams of maritime pine, with and without preservative treatment, produced in different ways, enabling the optimization of the glulam production method.

Being the most visible result of glue line degradation, delamination at the glue lines is one key factor taken into account when assessing glued laminated timber members in service. Not only delamination enable water intake in exterior structures fully exposed to weather and thus progressive damage due to moisture induced dimensional variations, but they may denote insufficient strength or durability of the bond joints regarding the intended service class.

Glued laminated timber structures in service often show considerable delamination, particularly if glulam is exposed directly to atmospheric agents, raising distrust. However, delamination influence on strength and stiffness will depend on their length, depth and exact location in the glulam member, as well as on the member size, shape and stress distribution.

In order to gain a more objective and systematic insight on the importance of delamination relative to their type and extension, a numerical study was developed. This numerical work focused the case of straight beams previously tested by the authors for strength and stiffness.

The details on the experiments conducted and results obtained so far can be found in [1,2,3,4,5].

2. Project overview

2.1 Natural and artificial ageing of glue lines

The experimental work involved glued laminated timber beams of maritime pine (*Pinus pinaster* Ait), some of which were glued after a copper azole deep preservative treatment. Maritime pine was glued in laboratorial environment with a phenol-resorcinol-formaldehyde (PRF) adhesive to produce 6 lamella beams with 0.115m × 0.2m × 1.5m. Untreated pine timber beams were cured at either 20ºC, 30ºC or 45ºC. Treated pine timber beams were cured at 45ºC. Commercial beams of spruce timber glued with PRF and MUF adhesives were also studied.

Natural and artificial weathering were applied to beams of all types. For natural weathering, the beams were sit on a fully exposed rig placed on a flat roof in Leiria (Portugal). A transparent protection coating was applied to all surfaces, as recommended in practice, to prevent premature degradation. Weathering started in April 2007 and a few beams were sampled after each 3 months, during 30 months of exposure.

Artificial weathering was conducted in a climatic chamber Fitoclima, imposing 3 ageing cycles, each one composed of a humid cold (95% rh, 15ºC) period during 4 weeks followed by a dry hot (30% rh, 45ºC) period during 4 weeks. In this case some beams were aged with the same transparent protective coating, the rest without coating.

2.2 In situ assessment of glue line ageing

After their weathering period, the beams were conditioned (65% rh, 20ºC) till moisture content equilibrium before testing. Non-exposed "control" beams were also tested. The effect of the ageing process on delamination and shear strength was subsequently evaluated.

For each beam, two types of shear specimens were extracted from the beams: block type specimens (rectangular right-angled prismatic form with shear area of 50×50 mm^2, EN 392); and core specimens (25mm diameter cylinders) extracted perpendicular to the glue lines. The tools and process for the extraction and test of the cores perpendicular to the glue lines are described elsewhere [2]. These allowed a small amount of material being removed, and testing several glue lines with each core. For a small amount of beams, other block shear specimens (20×20 mm^2 and 35×35 mm^2) and core specimens (35mm diameter cylinders) extracted parallel to the glue lines were also tested to assess the influence of test specimens' shape and size.

Delamination of the beams after natural weathering was determined separately on the North and South facing sides. Delamination tests were also carried out on specimens extracted from all beams following EN 391, method A.

Besides, phenol-resorcinol-formaldehyde (PRF) glue lines of untreated and copper azole (CA) treated laminated timber samples of maritime pine were analysed by Near Infra-red (NIR) Spectroscopy. The objective was to identify spectral changes of the adhesive related to ageing, preservative treatment or curing temperature that could be further used to evaluate glue lines.

The NIR spectra of glue lines were collected, after the shear test on the exposed adhesive surface, including all combinations of treatment levels, curing temperatures (20ºC, 30ºC, 40ºC, and 45ºC) and ageing. Spectra of the hardener and of the adhesive films were also obtained.

2.3 Modelling the influence of delamination

A 3-D finite element model of a straight glulam beam was developed using Abaqus/CAE. The modeled beam was 0.10 m wide x 0.24 m high (h) (6 lamellas) x 4.40 m long, simply supported over a 4.32 m span, with two loads symmetrically applied relatively to the middle of the beam. Tridimensional solid elements of 20 nodes and 3 degrees of freedom per node (displacement in *x, y* and *z* directions) were adopted. Timber lamellas were modeled with elements of 0.04 m x 0.04 m (the lamellas' thickness) x 0.033 m, in the *x*, *y* and *z* directions, respectively. The adhesive elements were 0.01 m x 0.01 m (in the glue line plane) x $0.1\text{x}10^{-3}$m thick (*x, y* and *z* directions, respectively). Delamination would therefore be modeled by removing some adhesive elements. Displacements were fully restrained at one support and allowed only in the x direction at the other one.

The following limit states were considered: deformation (DLS), bending strength (BLS) and shear strength (SLS), according to EN 1995-1-1. The maximum load for each limit state was determined assuming glulam class GL24h (EN 1194). For the deformation and bending limit states the maximum loads found, applied at 1.44m (6 x h) from the beam ends, were 5kN and 7.5kN. For shear limit state, the maximum loads determined were 18.4kN applied at a distance of 2 x h from the supports.

Since a 3-D FEM was considered, the timber was modeled as orthotropic material, to better reproduce the real performance of the beam. Preliminary simulations showed that the values adopted for the mechanical properties of glue line elements would not significantly affect the predicted stresses and deformations. Therefore the mechanical properties of timber were also adopted for the glue lines. Timber was modeled as linear elastic.

Delamination influence was checked as a function of its depth, considering both symmetric delamination (modes A,s to E,s) and non-symmetric delamination (modes B,ns to D,ns). In symmetric modes, delamination varied from 10 to 40mm deep (at each face) or up to 40mm at one face, plus 50mm at the other face, in the case of 90mm delamination. In non-symmetric modes, delamination was assumed on one face only, varying from 20 to 90mm.

The following delamination modes were considered:

- Modes A and B – delamination along the whole beam length: either just on the middle glued glue line (mode A) or in all glue lines (mode B);
- Modes C and D – delamination in all glue lines: either just in a central zone 3.44m long (mode C) or near the beam ends in 0.48m length (mode D);
- Mode E – delamination on both ends of the beam and in all glue lines. Delamination length varied from 80 to 480mm near each end, affecting the whole beam width.

The study of the delamination influence on deformations and stresses was done for the loads of the three limit states referred above: deformation, bending strength and shear strength.

3. Conclusions

The results highlight the different performances of glulam made with various preservative treatments and cure temperatures, in terms of the measured delamination and shear strength after the ageing process. They also show that shear testing of both drilled cores may be a promising tool in the assessment of glulam structures on site.

Modelling show that when delamination is non-symmetric regarding the member's cross section, it can cause the member's lateral instability, thus increasing its stresses and deformations. Delamination is not a problem when it occurs in members or member areas with low shear stresses, particularly when it is symmetric and does not reach the whole width of the beam. The stresses corresponding to the bending or deformation limit-states are close to the elastic limit only for very important delamination. Moreover, delamination depth higher than 60% of the cross section width seems to be a turn point beyond which the structural integrity may be at risk.

Ongoing work includes testing full scale glulam beams with delamination. Further work should cover the efficacy of delamination repair/strengthening methods and the influence of drying fissures on solid timber members in the scope of the assessment of old structures.

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Simplified cross-laminated timber wall modelling for linear-elastic analysis

Iztok Sustersic[1], Bruno Dujic[2]

Summary

The paper discusses simplified modelling of cross-laminated (crosslam, Xlam) timber walls suitable for linear elastic seismic design. We present equations for the appropriate horizontal in-plane stiffness of walls, taking into account the influence of the basic timber panels (including openings), top and bottom connections of walls, connection between adjacent wall panels, vertical load and friction. The proposed procedure is suitable for modelling a crosslam building with substitution frames or diagonals. A case study of a simple 2-paneled wall setup is presented using the latter.

1. Introduction

The reference method of Eurocode 8 [1] for determining the seismic effects on a building is the modal response spectrum analysis, using a linear-elastic model of the structure and the design spectrum. A simpler lateral force method is also allowed, however it can only be used for buildings regular in plan and elevation and conditionally for buildings regular only in plan. Unfortunately for the designers a vast majority of today's buildings does not comply with such criteria. Consequently the linear modal analysis (or any type of the nonlinear methods) must be used.

One of the most important parameters when using the modal analysis is the horizontal stiffness of a building. Stiffness and mass determine the structure's vibration periods and hence the influence of an earthquake's frequency content on a structure's response. If a low-rise (only ground floor) building's vibration periods

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are overestimated (too long) the resulting seismic forces can yield too conservative. If the periods are underestimated (too short), the results yield on the non-conservative side. The situation is just the opposite for higher buildings where the overestimated periods yield on the non-conservative side and vice versa. Hence great care must be taken in assigning the wall's correct stiffness. In the case of crosslam the stiffness is predominantly dependent of the shear angular bracket and hold-down behaviour. However, openings in panels greatly reduce their stiffness. The influence of vertical joints between adjacent panels also needs to be taken into account.

The evolution of the proposed finite element model (if using a substitute diagonal) is presented in the following figures, from top left to bottom right.

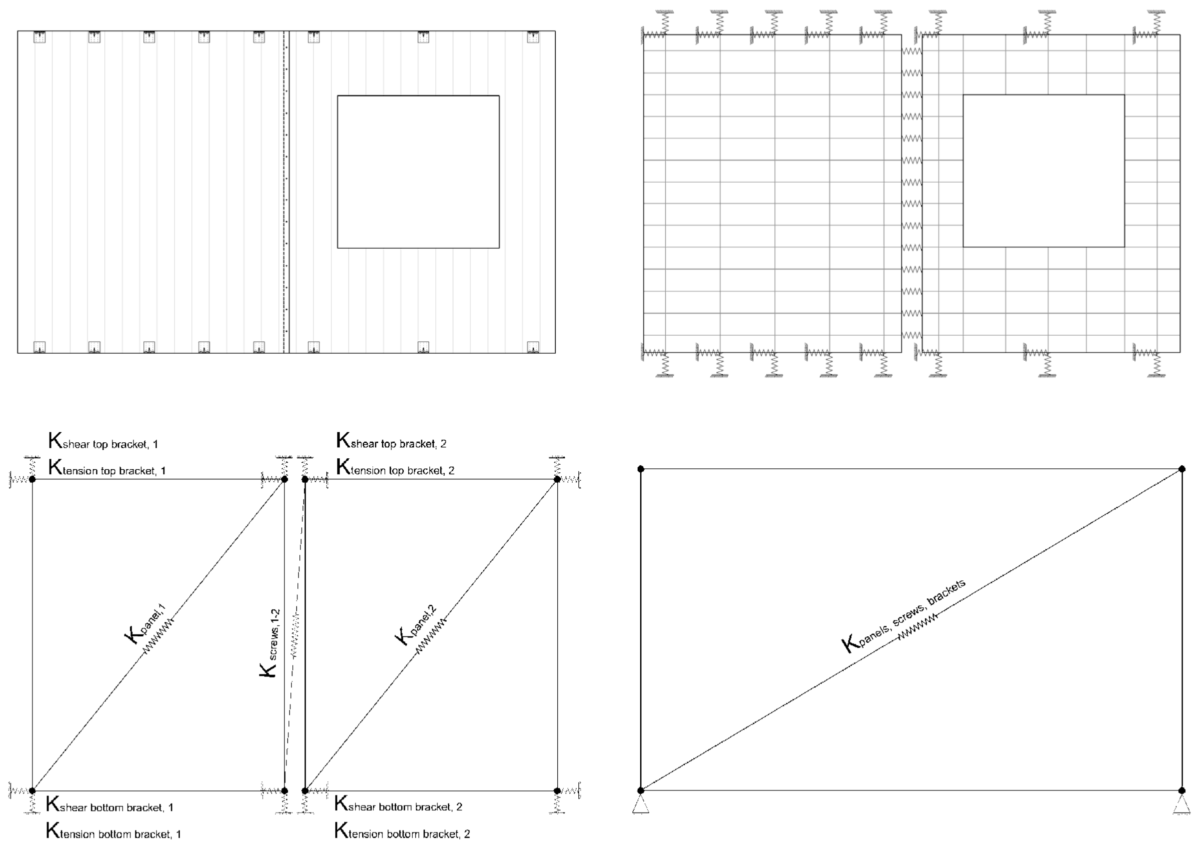

Fig. 1 (i) Actual crosslam wall assembly (top left), (ii) a detailed finite element (FE.) model (top right), (iii) a partially simplified FE model (bottom left) and (iv) the fully simplified FE model

The top left figure (*Fig. 1(i)*) shows a possible crosslam wall assembly. The left wall panel is full and the right panel has a window opening cut into it. Both panels are connected together at the adjacent sides over a step joint and self-tapping screws. At the top and bottom the panels are attached to the floor plates with angular brackets and/or hold-downs. The top right figure (*Fig. 1(ii)*) shows an exact FE model where shell elements are used to model crosslam panels and springs (linear on nonlinear) are used to model the brackets. Springs are also used for modelling the screws in the step-joint between the panels. In the bottom left figure (*Fig. 1(iii)*) the first simplification is shown – instead of using shell elements, the crosslam panels are modelled with substitution diagonals (trusses) and individual springs used in the exact model are joined together in discrete points in the corners of the new trusses. Such a model has already been used by researchers for nonlinear dynamic analysis (NLDA), however the trusses presenting crosslam panels were modelled completely stiff [2]. The latter simplification may be tolerable to some extent for nonlinear dynamic analysis. When using linear static analysis, too stiff trusses may contribute to errors that should not be neglected [3,4]. The bottom right figure (*Fig. 1 (iv)*) shows the fully simplified FE model, where the influences of the crosslam panels, top and bottom brackets and vertical connections are joined together in the substitution diagonal. In must be noted, however, that such a truss simplification is mostly suitable for linear elastic design as it is practically impossible to analytically join the nonlinear hysteretic response of all the aforementioned parameters in a single element. Linear elastic design on the other hand only demands the correct stiffness. The failure mechanism is only force dependent, hence a substitute diagonal can provide enough feedback. Though technically such simplified approach could also be used for nonlinear static analysis and displacement based design methods (i.e. the N2 method [5]) if several plastic hinges were to be incorporated in the diagonal element.

2. Equations for wall horizontal stiffness

We only present the equations for walls that are connected at the bottom (cantilever-like walls). Equations for walls connected at the top and bottom as well as for walls that have different top and bottom connection stiffness' are being derived in the scope of the project.

2.1 Stiffness equations

2.1.1 Crosslam panel stiffness

The stiffness of a single crosslam wall panel, connected at the bottom:

$$k_{panel} = \frac{3EI_{eff}0,8GA_s}{3EI_{eff}H + 0,8GA_sH^3} \tag{1}$$

where E is the main elastic modulus parallel to grain and I_{eff} is the effective radius of gyration for a given cross section. G is the shear modulus of timber, A_s is the effective shear cross section and H is the wall height. I_{eff} is calculated according to the method proposed by Blass and Fellmoser [6].

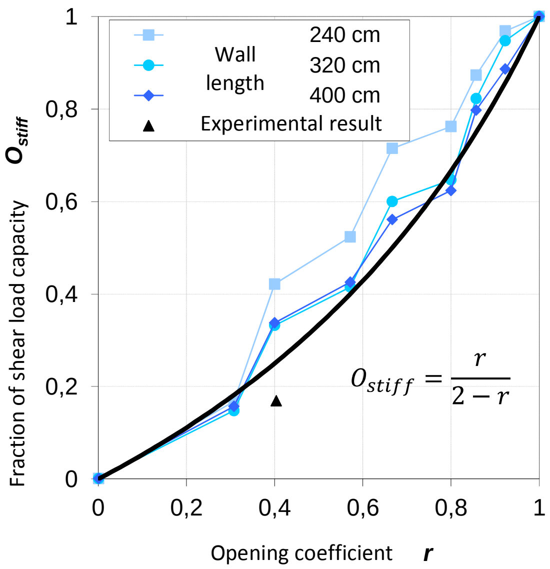

Fig. 2 Stiffness reduction of crosslam panels based on Sugiyama's panel area ratio (r) and stiffness reduction factors (O_{stiff}) by Dujic

The effective shear cross section A_s can be taken as the full cross section of a wall if the adjacent lamellas in a layer are glued together on the narrow side as well. If not a reduction of the shear cross section is appropriate [3, 4, 7, 8]. The influence of openings can be indirectly taken into account with Sugiyama's panel area ratio (r) [9] and stiffness reduction factors proposed by Dujic [10]. Therefore the stiffness of a panel with openings can be expressed as:

$$k_{panel,eff} = O_{stiff}k_{panel} \tag{2}$$

2.1.2 Wall connection stiffness

The bending stiffness of a panel's bottom connection can be expressed as:

$$K_{c,bend} = \frac{\left(\sum K_iL_i^2\right)}{H^2} + \frac{q_{vert}L_{eff}^2K_nL_n}{2H^2R_{c,n,Rd}} \tag{3}$$

where K_i is the stiffness of an individual connector, L_i is the distance of that connector from the point of rotation (A). L_{eff} is the length of the wall panel from the point of rotation to the other end of the panel and may be estimated as 0,9L (for moderately connected and loaded panels) where L is the complete wall panel length. The vertical line load on the top of a wall is denoted by q_{vert}. $R_{c,n,Rd}$ and K_n denote the design strength in the wall edge connection respectively. The stiffness values of connections and their strength can be derived in accordance with the Yasumura-Kawai procedure [11] if experimental results are available. Namely, stiffness equations from Eurocode 5 [12] yield to high values. The connection's shear stiffness is the sum of individual connectors shear stiffness ($K_{c,i,shear}$) and the contribution of friction stiffness:

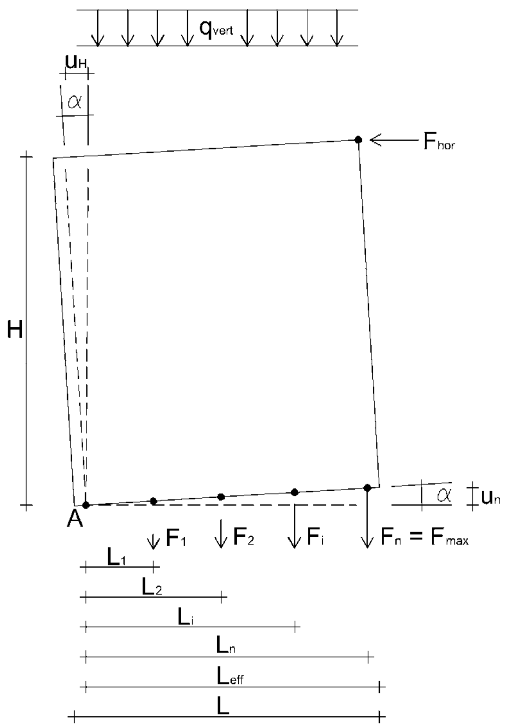

Fig. 3 Equilibrium of forces on a single crosslam panel

$$K_{c,shear} = \sum K_{c,i,shear} + \frac{q_{vert} L A_w c}{u_{slip,Rd}} \tag{4}$$

where $u_{slip,Rd}$ is the slip of the weakest connector at the design strength according to the bilinearised response curve (at the curves' shifting point), c is the dynamic friction coefficient and A_w is the wall contact surface cross section. The influence of screwed vertical connections between adjacent panels can also be implicitly taken into account and expressed in terms of a panels horizontal stiffness as:

$$K_{c,step} = \frac{K_{vert} L_{eff}^2}{H^2} \tag{5}$$

where K_{vert} is the sum of shear stiffness of screws in the vertical connection between two panels. However it must be noted that the upper equation is only valid up to a limited extent and for panels of approximately equal size.

2.1.3 Combined stiffness

We combine the horizontal stiffness of panels and connections into one single equation:

$$K_{wall} = \sum \left[\left(\frac{1}{k_{panel,eff,i}} + \frac{1}{K_{c,bend,i}} + \frac{1}{K_{c,shear,i}} + \right)^{-1} + K_{c,step,i} \right] \tag{6}$$

3. Case study

A crosslam wall panel setup similar to the one in Figure 1 with the exception of both walls being full (no openings) is analysed. The left panel is anchored at the bottom with 3 BMF 105 angular brackets. The right panel is attached with 5 brackets of the same type. No top connections are considered. The vertical step joint is connected with five 8 mm screws. The crosslam panels are 95 mm thick, 200 cm long (each) and 300 cm high. The bottom support is very stiff (i.e. concrete foundation). Friction is neglected and as no vertical load is applied we also presume that the rotation point (point *A* in *Figure 3*) is in the walls corner. A horizontal load of 100 kN is evenly applied over the top of the wall. The horizontal displacements at the top of the wall are compared; a precise finite element model (like in *Figure 1(ii)*) against the proposed simplified model (*Figure 1(iv)*). A relative displacement (displacement (ii) / displacement (iv)) comparison is presented in Figure 4. The following models are compared:

1	Timber panels only
2	Bottom connection 1 (1st panel)
2	Bottom connection 2 (2nd panel)
3	Step joint (panels modelled stiff)
4	Complete 1st panel
4	Complete 2nd panel
5	Complete wall
5	Complete wall, panels modelled stiff
5	Complete wall, without a step joint

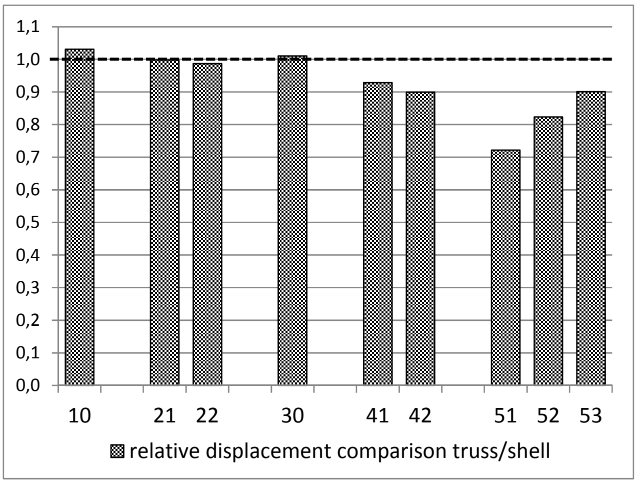

Fig. 4 Relative displacement comparison of precise FE models and the simplified proposal

4. Conclusions

Discrepancies occur for combined stiffness (42-53). The agreement between individual stiffness of panels (10), bottom connections (21, 22) and the vertical step joint connection (30) is very good. However, the step joint has a significant influence on the wall assembly if the panels are not modelled stiff – compression and tension strains in individual crosslam panels occur in the vertical direction of the step joint causing a more flexible construction (51). That is something the proposed model does not deal with yet – additional vertical strains still need to be implemented. Also, as we have taken the effective length of wall panels equal to their actual length, we end up with a somewhat stiffer system. An effective length of about 0,95 L would most likely be more in place.

5. Acknowledgements

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Determination of physical and mechanical properties of chestnut timber (Castanea sativa Mill.) from Spain

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Summary

Chestnut timber from Spain is not included as a structural timber in the European standard EN 1912 nor is it assigned to any strength class like other Spanish species. Therefore, a characterization of structural chestnut timber from Spain is carrying out according to the UNE EN 408:2010, and the characteristic values are calculated according to the UNE EN 384:2010. The values obtained allow the assignment of a strength class according to UNE EN 338:2010. Database is made for 1002 samples of three sections (40x100 and 40x150 mm) and five provenances (Asturias, Galicia, Catalonia, Extremadura and Castilla y León). In addition, some non-destructive techniques (ultrasonic, impact waves, vibrational analysis) are applied in order to establish the relationships between non-destructive variables and mechanical properties of chestnut timber. This characterization will allow the introduction of chestnut structural timber in the European standards and the revalorization of the value chain of this important species in Spain.

1. Introduction

Chestnut (Castanea sativa Mill.) is a very important species in Spain in construction scope, with an historical employment and proved structural capability. Although this, this wood is not included as structural material in Spain, and has no assigned resistant class from EN 1912, as other Spanish species like the four main pines and eucalyptus. EN 338 standard indicates strength classes in function of mechanical properties and density, for conifer and deciduous wood. In order to determine this strength class, a mechanical characterization according to

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EN 408 is necessary, and the values of modulus of elasticity, bending strength and density must be obtained. The determination of the characteristic values following the EN 384 standard allows the assignment of a strength class to the sample evaluated.

The characterization methodology of structural Spanish timber is well defined and developed in some works, mostly in conifer species [1] [2] [3] [4] [5] [6] [7]. In relation to deciduous species, the characterization experience is limited, with eucalyptus as only species normalized in the European standards. In addition, the Spanish standard of visual classification (UNE 56546) is not suitable for its application in other species, like chestnut.

Some works [8] [9], evaluates the mechanical properties of small dimension chestnut timber samples. But the absence of structural size mechanical tests in chestnut timber makes necessary the application of the established methodologies in order to know deeply its properties and to include it in the European standard as structural material. In addition, the application of the current standards will allows the evaluation of the adequacy of methodologies and criteria defined and the proposal of the consequent modifications.

Today, non-destructive testing has great importance on the structural timber sector. The research about the estimation of the mechanical properties through these methods has an increasing development and industry implementation. The predictive capability of non-destructive techniques to evaluate the mechanical properties of timber, such as methods based on the relationship between wave velocity and Young's modulus, has been widely studied. Many works, mostly related to conifer species, demonstrate the adequacy of these techniques to estimate the modulus of elasticity [10] [11] [12] [13] [14] [7] [15]. Thus, some non-destructive techniques have been evaluated in this project for the knowledge of the relationships and correlations between mechanical, physical and acoustic properties of the chestnut timber.

Therefore, the objective of this project is the application of the cited standards and methodologies in order to characterize the chestnut structural timber from Spain, obtaining a strength class and evaluating the adequacy of the current normative.

2. Methodology

2.1 Mechanical testing

The mechanical tests are carried out according to EN 408. This standard defines the methodology and conditions for the calculus of the most important mechanical and physical properties: modulus of elasticity, bending strength and density.

Density (ρ) is calculated from the weight and volume of each sample. The values of the static modulus of elasticity (MOE_{loc} and MOE_{glo}) and bending strength (MOR) are obtained from a four-point bending test (figure 1).

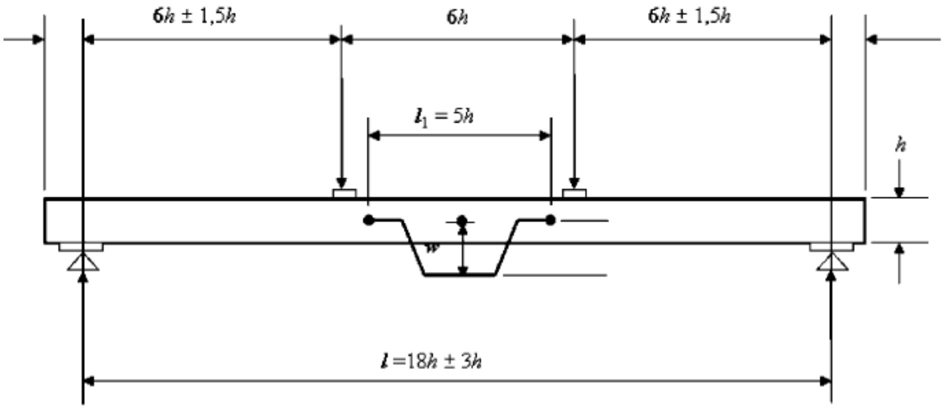

Figure 1. Four point bending test (EN 408: 2011)

Local and global moduli of elasticity are calculated with the expressions from the EN 408 standard (equation 1 and 2, respectively). In the same way, bending strength is calculated with equation 3.

$$E_{m,l} = \frac{a l_1^2 (F_2 - F_1)}{16 I (w_2 - w_1)} \ [\mathrm{N/mm^2}] \tag{1}$$

where

a = distance between the load point and the support point, in mm

l_1 = measurement basis length (5h), in mm

I = moment of inertia, in mm4

F_2-F_1 = increase of load, in newtons

w_2-w_1 = increase of deformation in mm, corresponding to F_2-F_1

$$E_{m,g} = \frac{l^3 (F_2 - F_1)}{b h^3 (w_2 - w_1)} \left[\left(\frac{3a}{4l} \right) - \left(\frac{a}{l} \right)^3 \right] [\mathrm{N/mm^2}] \tag{2}$$

where

a = distance between the load point and the support point, in mm

l = distance between supports, in mm (18h)

b,h = width and heigth, in mm.

F_2-F_1 = increase of load, in newtons

w_2-w_1 = increase of deformation in mm, corresponding to F_2-F_1

$$f_m = \frac{3F_{máx}a}{bh^2} \ [\mathrm{N/mm^2}] \tag{3}$$

where

$F_{máx}$ = maximum load, in newtons

a = distance between the load point and the support point, in mm

b,h = width and heigth, in mm

2.2 Visual classification and non-destructive testing

Before mechanical testing, a visual classification is carried out according to UNE 56546 Spanish standard. Visual classes are defined in function of the singularities of the wood (knots, grain deviation, deformations, etc.) and related to the EN 338 strength classes through EN 1912 standard. The singularities defined on this normative are measured, and visual classes are assigned to each sample.

Furthermore, the acoustic wave transmission velocity is measured by applying three different non-destructive techniques: ultrasound (CBT Sylvatest Trio-CBS), impact waves (Microsecond Timer, Fakopp) and vibrational analysis (Portable Lumber Grade, Fakopp). From these velocity values, the corresponding dynamic modulus of elasticity is calculated (Equation 4).

$$MOEdyn = \rho \cdot v^2 \tag{4}$$

where

ρ = density, in kg/m^3

v = wave transmission velocity, in mm/µs

3. Results

Table 1 shows the preliminary results from 250 samples [16]. Modulus of elasticity, bending strength and density were calculated.

Table 1. Mean values of local ($E_{m,l}$) and global ($E_{m,g}$) modulus of elasticity, characteristic value of bending strength ($f_{m,k}$) and density (ρ_k)

Section (mm)	N	$E_{m,l}$ (kN/mm^2)	$E_{m,g}$ (kN/mm^2)	$f_{m,k}$ (N/mm^2)	ρ_k (kg/m^3)
(40x100)	126	13,09	12,10	29,17	499,69
(40x150)	124	12,15	11,12	25,30	499,32
Todas	250	12,63	11,61	26,28	499,95

Relationships between modulus of elasticity and non-destructive variables (dynamic modulus of elasticity and wave velocities) have been established, considering the preliminary group of samples. Good correlation coefficients were obtained, and some models were constructed. The determination coefficient for each models are shown in table 2.

Table 2. Linear regression models for the estimation of modulus of elasticity ($E_{m,g}$) with ultrasonic, impact wave and vibrational wave velocity (V_u, V_i and V_v), density (ρ) and dynamic modulus of elasticity as independent variables.

Dependent variable	Independent variable	R^2	Error (u)
$E_{m,g}$	$V_u + \rho$	0.698	0.698
	$V_i + \rho$	0.697	0.758
	$V_v + \rho$	0.666	0.812
	MOE_{dynu}	0.701	0.796
	MOE_{dyni}	0.686	0.806
	MOE_{dynv}	0.689	0.860

4. Discussion and conclusions

The end of the characterization of the samples of all provenances is the first step for the process to include the Spanish chestnut timber in the European normative.

In the other hand, the evaluation of the results of the non-destructive tests will allows the development of mathematical models for the indirect estimation of chestnut timber properties and its future application in some technical and industrial processes.

The adequacy of the Spanish visual classification will be evaluated through the analysis of the visual classes assigned and the mechanical properties obtained. Consequently, some modifications will be proposed for the correct adaptation of the UNE 56546 standard to the structural chestnut timber.

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Analysis of the stress state of a halved and tabled traditional timber scarf joint with finite element method and comparison with the theory of strength of materials

Jose R. Aira[1], Francisco Arriaga, Guillermo Íñiguez-González[2], Manuel Guaita[3], Miguel Esteban[4]

Summary

The objective of this study is to determine the stress distribution in the halved and tabled traditional timber scarf joint with the finite element method and its comparison with the values obtained using the theory of Strength of Materials. The stress concentration areas and the influence on the results due to the refining of the mesh are studied in order to determine the mesh size that provides the stress values more consistent with the theory.

1. Introduction

Joints are the points of transmission of forces between the members of a timber structure. In traditional joints stresses are transmitted from one piece to the other one by means of carpentry works that balance the axial and shear stresses through local compressions, tangential stresses and friction between the contact faces. The metallic elements are usually incorporated with the unique mission of keeping continuity of the contact faces [1].

The halved and tabled scarf joint consists of an end joint transmitting the tension axial force N through compression parallel to the grain located in the cross-section area of the notch $b \cdot t$, and this compression is transmitted to the entire cross-section through shear stress at the surface $b \cdot l$ (Fig. 1).

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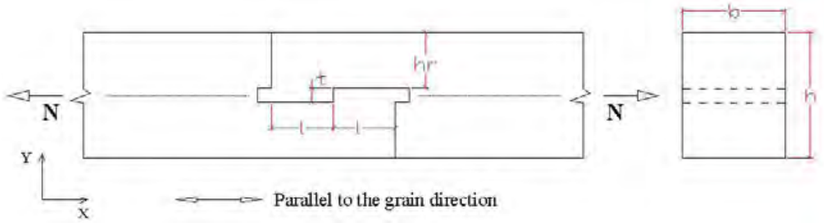

Fig. 1 Halved and tabled scarf joint

Moreover, the narrowing of the section in each of the pieces, resists the tensile force combined with a bending moment M caused by the eccentricity of the axial force N, producing a rotation that tends to disassemble the joint [2]. This effect can be avoided by making notches in the ends of the tabled joint or by placing metal straps. For easier installation procedure, a wedge is placed on each side to ensure tightly contact between the members (Fig. 2).

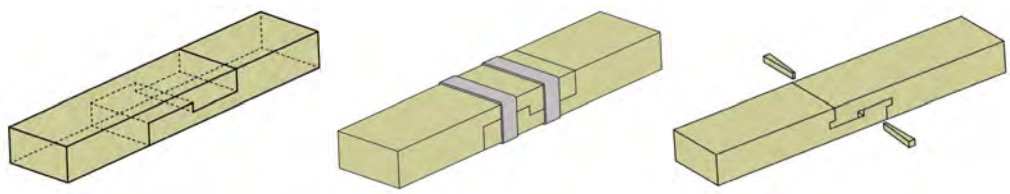

Fig. 2 Different arrangements of the halved and tabled scarf joint

2. Material and methods

2.1 Description of the model

The geometric parameters of the studied joint are: hr = 60 mm, t = 30 mm, h = 150 mm, b = 50 mm, l = 90 mm. To model the joint by FEM, the left part is considered coerced on sliding supports in all nodes preventing displacement in the X axis, and a fixed support in the upper left node to prevent movement in the Y axis. The right part receives the external load of 10 kN which is uniformly distributed in the entire cross-section. A friction coefficient between contact faces of 0,467 is considered.

The three possible failure modes have been studied in the joint: a) Bending-tension failure corresponding to the reduced section of the piece subjected to tensile and bending stresses, σ_x, b) Local compression failure corresponding to the section of the notch subjected to compression stress, σ_x, and c) Shear failure corresponding to the section of the horizontal plane subjected to shear stress, τ_{yx} (Fig. 3). The critical sections are studied by comparing the stress values obtained by the application of FEM to the values obtained through the formulation of the classical theory of strength of materials in order to determine the influence of mesh size on results and the coincidence of the stress distributions obtained with theoretical values.

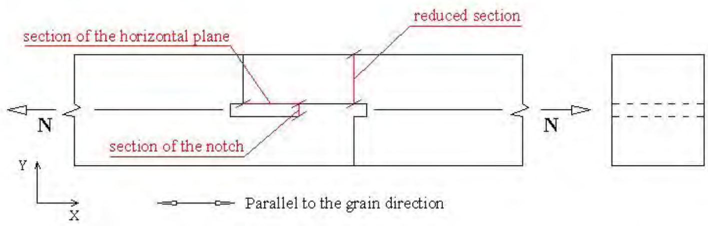

Fig. 3 Critical cross-sections

2.2 Software

The finite element analysis is made by a plane stress linear static study that allows the consideration of the thickness of the pieces. Wood is considered as an orthotropic material and the values of the elastic properties perpendicular to the grain are achieved by the arithmetic average in the radial and tangential directions. In order to perform the numerical simulation of the joint, each piece is modeled in the ANSYS finite element software taking the element of its internal library called PLANE42. This element is used for two dimensions modeling of solid structures and can be used either as a plane element (plane stress or plane strain) or as an axisymmetric element. The element is defined by four nodes having two degrees of freedom at each node (translations in the nodal x and y directions)and it has plasticity, creep, swelling, stress stiffening, large deflection, and large strain capabilities (Fig. 4) [4]. The model includes the simulation of contact between surfaces. Groups of two different lines are defined in the contact zone established. Each of the lines belong to a different solid but having the same coordinates and geometric position in order to obtain coincidence at the nodes of each line. These lines of friction are meshed with one dimension contact elements in the direction of the lines in order to define the surface to surface contact [3]. Thus, the contact pair is set using the internal library elements called TARGET and CONTACT.

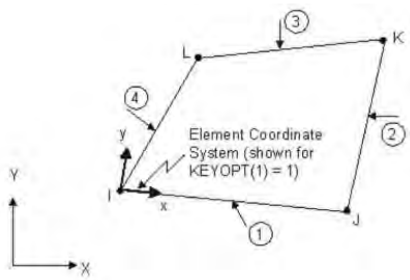

Fig. 4 PLANE 42 element

3. Results

3.1 Stresses distribution

The areas with stress concentration and those with lower stress are studied. To identify graphically these regions, distribution of normal stresses σ_x and distribution of tangential stresses τ_{yx} are showed from the ANSYS graphical output. Uniform mesh of size 2 mm is used (Fig. 5).

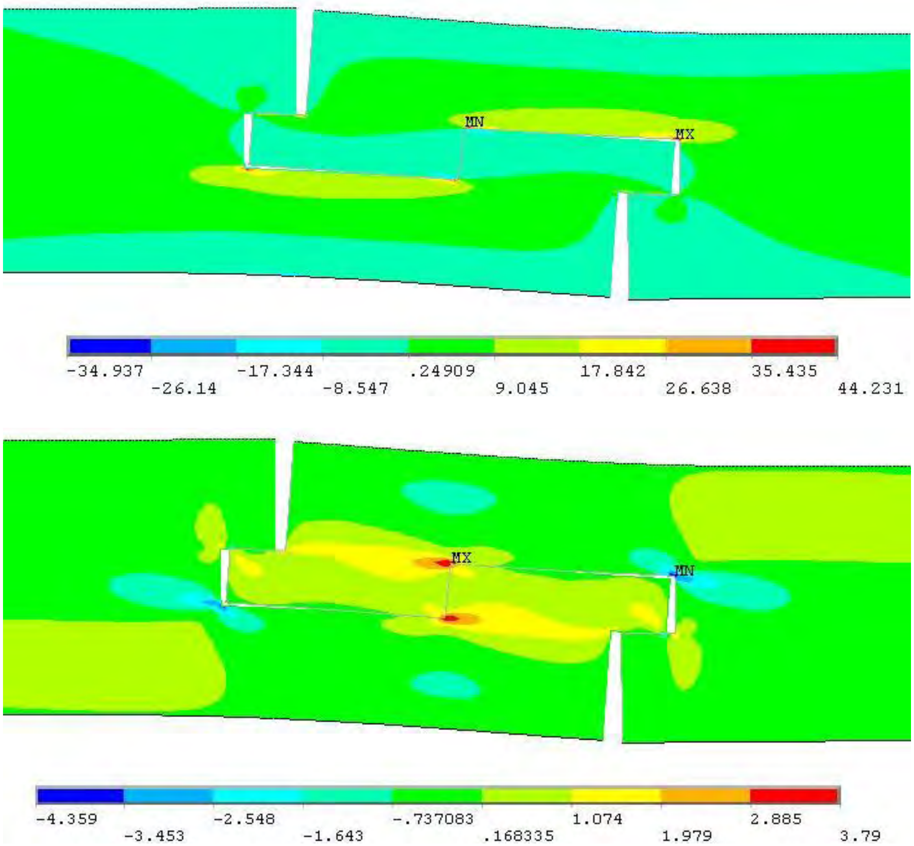

Fig. 5 Distribution of normal stress σ_x (on top) and shear stress τ_{yx} (at the bottom) in N/mm^2

The graphical output shows a central symmetry of the isobar lines distribution where the centre of symmetry corresponds to the geometric centre of the joint. In the reduced section, subject to normal stress (bending combined with tension), there is not a point of high stress concentration except in the bottom where there is a concentration of tensile stress due to the abrupt decrease of the effective cross-section.

In the section of the notch, subject to normal stress (local compression), there are two areas of high compression stress concentration, one located at the top of the section and one in the bottom of it.

In the section of the horizontal plane, subjected to shear stress, stress values are close to zero at the end of the heel, increasing progressively to high levels of stress near the central notch.

3.2 Study of the mesh size

In the finite elements model four different uniform mesh sizes are used: 10 mm, 5 mm, 2 mm and 1 mm. The computer consuming time for size 2 mm and 1 mm is too long to be functional so a type of progressive mesh is required. The progressive mesh should have the same small size in stress concentration areas and increase in size progressively when the stress concentration decreases (Fig. 6). Using a progressive mesh instead of a uniform mesh, the number of nodes and finite element model is lower and consequently the number of degrees of freedom and equations to be solved by the software is also lower.

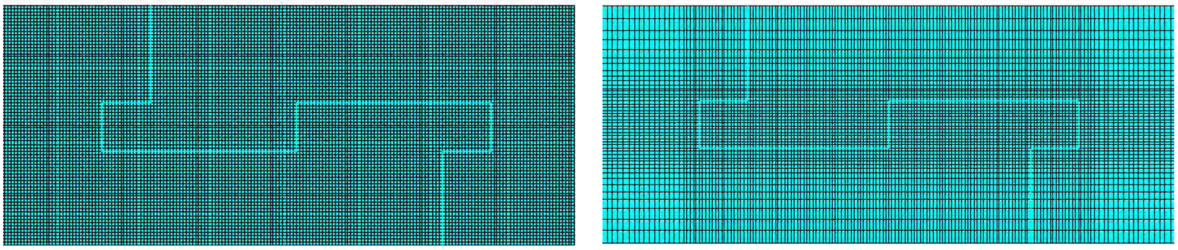

Fig. 6 Uniform mesh and progressive mesh

To compare the accuracy of both types of mesh, the stress distribution at the critical sections is analyzed using a uniform mesh and a progressive mesh with sizes 2 mm and 1 mm.

The stress distribution for both types of mesh is identical. The stress values of the uniform mesh of size 2 mm are the same to the stress values of the progressive mesh of minimum size 2 mm. The same applies to uniform mesh of size 1 mm and the progressive mesh of minimum size 1 mm. Therefore it is possible to use a progressive mesh to provide the same precision in the results that a uniform mesh, but using a much lower computer time.

After checking the validity of the progressive mesh, to achieve greater accuracy in the results, a progressive mesh of minimum sizes 2 mm, 1 mm, 0,5 mm and 0,2 mm are used in the analysis of the critical sections.

3.3 Failure mode A (bending combined with tension)

According to the theory of Strength of Materials, the normal stress in the reduced section is obtained by the algebraic addition of the normal stress produced by the axial force N and bending moment M (Fig. 7). Therefore, the normal stress σ_x is given by the expression:

$$\sigma_x = \frac{N}{b \cdot h_r} \pm \frac{M \cdot y}{I} = \frac{10.000}{50 \cdot 60} \pm \frac{450.000 \cdot 30}{900.000} = \begin{cases} -11,67 \text{ N/mm}^2 \text{(compression)} \\ 18,33 \text{ N/mm}^2 \text{(tension)} \end{cases} \tag{1}$$

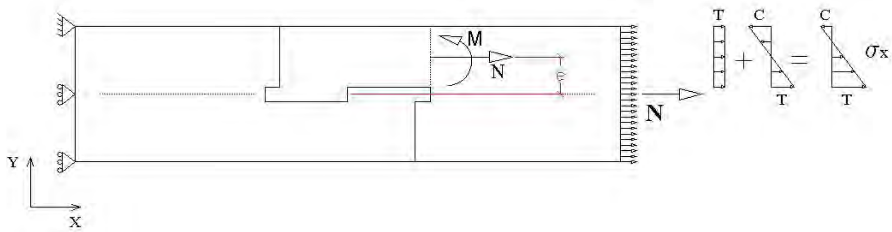

Fig. 7 Reduced section subjected to tensile force combined with a bending moment

To compare the results obtained by FEM with theoretical values, a graph with Cartesian axes is made (Fig. 8). The graph shows the stress distribution obtained by FEM along the reduced section and the theoretical stress distribution. The vertical axis represents the reduced section height in mm and the horizontal axis represents the normal stress in direction parallel to the grain σ_x in N/mm^2.

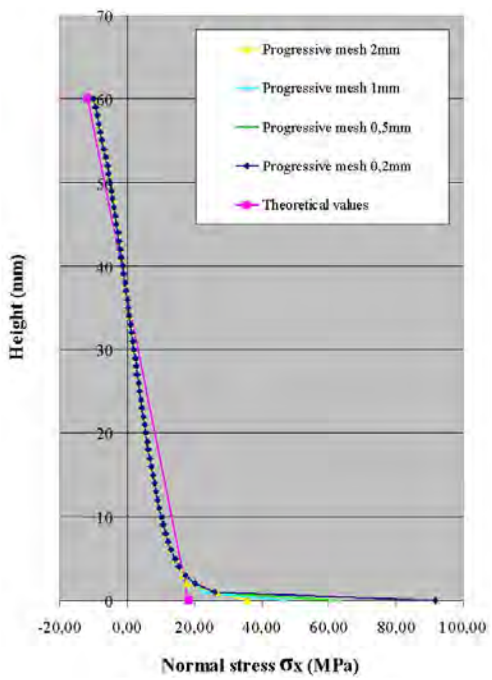

Fig. 8 Normal stress distribution in reduced section

The stress distribution for different mesh sizes is similar between the heights of 10 mm and 60 mm in the reduced section. That is, the values of compression stress in the top of the section are very similar for different mesh sizes. The tensile stress values are also coincident except for the point where there is a stress concentration. At this point, smaller mesh size indicate higher stress values by FEM.

When the volume of stress is calculated for different mesh sizes, it is observed that the values obtained are lower than the theoretical value.

When the mesh is refined, the accuracy increases and the values obtained are approaching to the theoretical value. However, with the smaller mesh size (0,2 mm), the volume of stress obtained is even greater than the theoretical value.

3.4 Failure mode B (local compression)

The normal stress σ_x in the section of the notch (Fig. 9) can be obtained using the following expression:

$$\sigma_x = \frac{N}{b \cdot t} = \frac{10000}{50 \cdot 30} = 6,67 \text{ N/mm}^2 \tag{2}$$

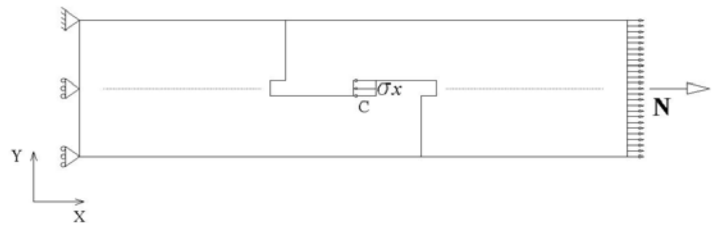

Fig. 9 Section of the notch subjected to compression stress

In the same way, a graph with Cartesian axes is made (Fig. 10) where the vertical axis represents the height of the section of the notch in mm and the horizontal axis represents the contact pressure σ_x in N/mm^2. The stress distribution is perfectly symmetrical. There are two areas of stress concentration, one located at the top of the section (height = 30 mm) and other located in the bottom of it (height = 0 mm). In these areas, stress values by FEM increases with decreasing mesh size. In the rest of the section where the stress concentration is low, the stress distribution for different mesh sizes is similar and close to the theoretical value.

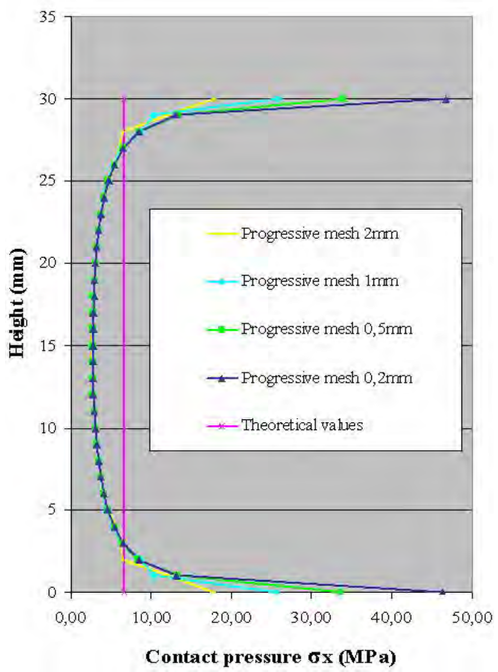

Fig. 10 Normal stress distribution in section of the notch

When the volume of stress is calculated for smaller mesh sizes, it is observed that the values obtained are lower than the theoretical value. When the mesh is refined, the accuracy increases and the values obtained are approaching to the theoretical value.

3.5 Failure mode C (shear)

Assuming a uniform distribution in the entire section of the horizontal plane (Fig. 11), the shear stress can be obtained by the following expression:

$$\tau_{yx} = \frac{N}{b \cdot l} = \frac{10000}{50 \cdot 90} = 2,22 \text{ N/mm}^2 \tag{3}$$

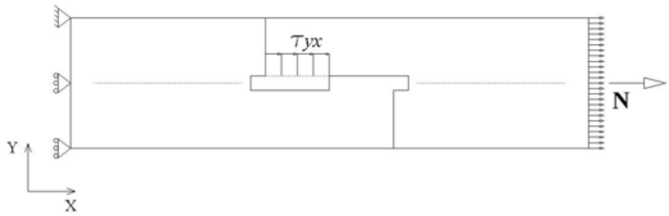

Fig. 11 Section of the horizontal plane subjected to shear stress

As in other critical sections, a graph with Cartesian axes is made (Fig. 12) where the vertical axis represents the shear stress τ_{yx} in N/mm^2 and the horizontal axis represents the length of the horizontal plane in mm.

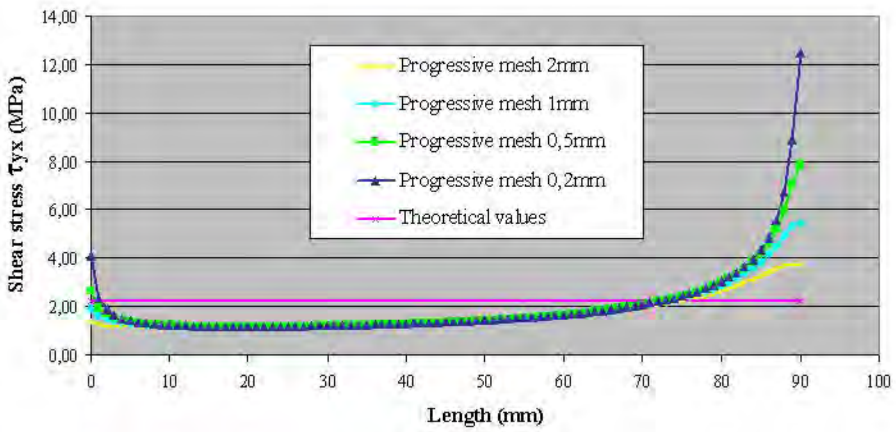

Fig. 12 Shear stress distribution in section of horizontal plane

The stress distribution shows values close to zero at the end of the heel (length = 0 mm) and it increases when approaching the beginning (length = 90 mm), where stress concentration occurs. In this area, the shear stress value by FEM increases with decreasing mesh size.

4. Conclusions

Through the analysis of the stress distribution in the different sections of study for each mesh size and comparing the values obtained from the FEM and the classical theory of Strength of Materials, can be concluded:

- The results show a central symmetry of the isobar lines distribution where the centre of symmetry corresponds to the geometric centre of the joint.
- In areas where stress concentration is lower, different mesh sizes show similar stress values. In areas where stress concentration occurs, the same values increase considerably with the refinement of the mesh being necessary to refine it enough to collect the maximum stress.
- When the volume of stress is calculated for smaller mesh sizes, it is observed that the values obtained are lower than the theoretical value. When the mesh is refined, increases the accuracy and the values obtained are approaching to the theoretical value. However, an excessive reduction of the mesh size can result in volume of stress higher than the theoretical value due to very high stress concentration in specific areas.
- Taking into account the computational resources and accuracy of the results, the correct mesh size is a progressive mesh which combines a large mesh size, in areas where there is not stress concentration, with a mesh size refined enough in the stress concentration areas.
- Comparison of normal stress levels obtained by the FEM and the classical theory shows small differences except at points of stress concentration.

5. Further research

- Consider into the FEM the compression plastification due to high stress concentration that occurs in abrupt changes in section, and its experimental validation.
- Establish an appropriate failure criterion for this type of joint by FEM and experimental confirmation.
- Achieve geometric optimization of the joint searching a maximum structural efficiency with minimal volume of material. It will be analyzed from a theoretical point of view using the theory of Strength of Materials and by applying the FEM. Then, the results obtained by both methods will be compared and checked by mechanical tests.
- Finally, the study will include the analysis of other types of traditional joints between structural elements of wood.

6. Acknowledgements

Ministry of Science and Innovation, BIA2010-18858: "Non-destructive techniques for structural clasification of wood in new construction and rehabilitation".

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F.E.M. analysis of the strength loss in timber beams of *Pinus sylvestris* due to the presence of circular knots

Manuel Guaita[1], Vanessa Baño[2]

Summary

A specific finite element model was used to simulate simply supported timber beams containing singularities, such as knots and grain deviation, and to predict their maximum load in bending. The model considers the elastoplastic constitutive law of timber and simulates a four points bending test according to EN 408 and was validated using ten 45 x 145 x 3000 mm beams of Scots pine. Four sub-models reproduced different knot conditions in order to find the best simulates real behaviour: beams without knots, knots considered as a hole, adherent knot and partially adherent knot. 165 virtual beams were modeled varying the position and size of knots and considering the local grain deviation and their influence on load capacity was studied.

1. Introduction

Timber beams usually contain singularities, such as knots, fissures, wanes, holes, etc. The most relevant singularities are knots and their local grain deviation in the area surrounding them, which affect the strength and stiffness of timber beams. When a knot is located in the tension side of a beam, the local grain deviation produces in the fibres tension parallel to the grain, perpendicular to the grain and shear stresses. Usually, the rupture begins in the elastic phase with a fissure due to tension perpendicular to the grain around the knot together with tension parallel to the grain. When the rupture occurs in the elastoplastic phase, this behavior can only be analysed using the finite element method [1].

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Many studies combining FEM techniques and the theory of strength of materials to predict the ultimate capacity of solid sawn timber considering the influence of the presence of knots and local grain deviation [2], [3], [4], [5]. Hole placement with respect to beam height and relative hole size with respect to beam height were studied by Danielsson et al. [6] and Williams et al. [7].These authors pointed out however that these stresses are different to stress concentrations around a natural knots in a beam, which presents local grain deviation and a relatively smooth flow of stress occurs. Studies of the ultimate load capacity on sawn timber with presence of knots and local grain deviation were developed by several authors [8], [9], [10].

The first objective of this work is to describe a finite element model to simulate the behavior of timber beams with knots and local grain deviation and to predict failure load and the location of the rupture, in relation to size and position of knots. The second objective is to analyse the influence on bending strength (f_m) of different sizes and position of knots and their associated grain deviation, throughout the height of the central section of beams and to study the influence of the presence of knots on the stress distribution.

2. Material and methods

2.1 Material tested

Ten 3000 mm long specimens of sawn timber of Scots pine (*Pinus sylvestris* L.) with a cross section of 45x145 mm were tested to validate the model, selected by tangentially cut beams which showed through knots positioned near to the edge of the tension side. The mean density value was 556 Kg m^{-3} and the knot diameter was between 28.5 and 43.7 mm.

2.2 Mechanical properties of timber considered for the numerical simulation

The local modulus of elasticity (E_m) and bending strength (f_m) were obtained for each experimental beam according to the EN 408 standard [11]. The elastic constants adopted where obtained from previous experimental work on small clear specimens, Table 1. The ortotropic and elastoplastic behavior laws were modeled taking into account the curve zone of stress-strain in compression and a different modulus of elasticity in tension to compression was considered, Table, [12].

Table 1. Modulus of elasticity for elastic and ultimate stress in tension, in compression parallel to the grain and in shear for P. sylvestris L

Modulus of elasticity in compression parallel to the grain (N·mm^{-2})	$E_{c,0} = 8100$
Modulus of elasticity in tension parallel to the grain (N·mm^{-2})	$E_{t,0} = 9720$
Modulus of elasticity perpendicular to the grain (N·mm^{-2})	$E_{90} = 332.7$
Shear modulus of elasticity (N·mm^{-2})	$G = 623.8$
Tension parallel to the grain (N·mm^{-2})	$f_{t,0,u} = 89$
Compression parallel to the grain (N·mm^{-2})	$f_{c,0,u} = 57$
Yield stress in compression parallel to the grain (N·mm^{-2})	$f_{c,0,y} = 39$
Tension perpendicular to the grain (N·mm^{-2})	$f_{t,90,u} = 10$
Shear (N·mm^{-2})	$f_{v,u} = 14$
Compression perpendicular to the grain (N·mm^{-2})	$f_{c,90,u} = 7$

Table 2. Relationship between stress level and modulus of elasticity in compression parallel to the grain [12]

Stress $\sigma_{c,0}$ (N·mm^{-2})	**$E_{c,0}$ (N·mm^{-2})**
$\sigma < -57$	10
$-57 \leq \sigma < -55$	500
$-55 \leq \sigma < -52$	1100
$-52 \leq \sigma < -50$	2000
$-50 \leq \sigma < -45$	2700
$-45 \leq \sigma < -41$	3300
$-41 \leq \sigma < -39$	4200
$-39 \leq \sigma < 0$	8100

2.3 Numerical simulation

The four points bending test was simulated by finite element method using software ANSYS 12.0, ANSYS, Inc., Canonsburg, USA. The geometry of the beams was created by rectangular areas and a 4-node quadrilateral elements plane stress (Plane 42) from ANSYS library and an uniform meshing was used.

Four models were developed to characterize the knots, and these were modeled incorporating the material properties of the wood in the perpendicular to the grain direction: 1. Clear beam, without knots, 2. Knots considered as holes in the beam, 3. Adherent knots, where there is structural continuity of knot material and beam material and 4. Partially adherent knots, artificially simulated by means of a contact spring between the two materials.

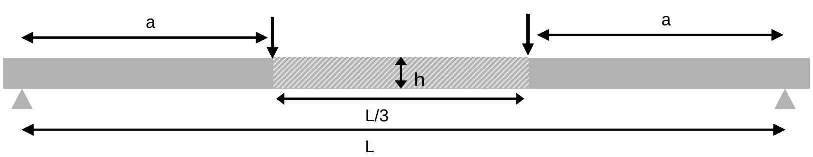
A load-stepping method was developed in ANSYS in order to obtain the maximum load capacity of beams and stress distribution. The model automatically ends when one element of meshing reaches the value of 1 in the quadratic failure criterion [13].

$$\left(\frac{\sigma_{c,0}}{f_{c,0,u}}\right)^2+\left(\frac{\sigma_{c,90}}{f_{c,90,u}}\right)^2+\left(\frac{\tau}{f_{v,u}}\right)^2\leq 1 \tag{1}$$

$$\left(\frac{\sigma_{t,0}}{f_{t,0,u}}\right)^2+\left(\frac{\sigma_{t,90}}{f_{t,90,u}}\right)^2+\left(\frac{\tau}{f_{v,u}}\right)^2\leq 1 \tag{2}$$

where $\sigma_{t,0}$= applied stress in tension parallel to the grain; $f_{t,0,u}$= ultimate stress in tension parallel to the grain; $\sigma_{t,90}$ = applied stress in tension perpendicular to the grain; $f_{t,90,u}$ =ultimate stress in tension perpendicular to the grain; τ = applied shear stress; $f_{v,u}$ = ultimate stress in shear; $\sigma_{c,0}$ = applied stress in compression parallel to the grain; $f_{c,0,u}$ = ultimate stress in compression parallel to the grain; $\sigma_{c,90}$= applied stress in compression perpendicular to the grain and $f_{c,90,u}$= ultimate stress in compression perpendicular to the grain.

Once the numerical model was validated, it was used to create new virtual beams in order to analyse the effect of the presence of knots in the stress distribution and strength of timber. For this objective, five size of knots and 11 positions throughout the height (h) of the beams were considered. The diameter of knots considered were: 10 mm, 20 mm, 30 mm, 40 mm and 50 mm and the positions throughout the height of the beam were: 0 when the knot was positioned on the neutral axis, +0.1 h, +0.2 h, +0.3 h, +0.4 h, +0.5 h, -0.1 h, -0.2 h, -0.3 h, -0.4 h and -0.5 h, Figure 1.



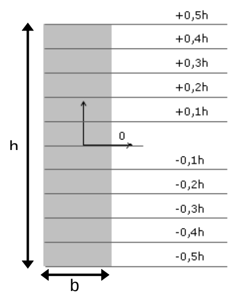


Figure 1. Cross section of the beam: positions of knots throughout the height (h).

3. Results and discussion

3.1 Results of the mechanical testing and the numerical simulation of experimental beams

Of the ten beams tested, seven broke due to the influence of the knot and the remainder broke as a result of a tension parallel to the grain. The bending strength and the position of failure in beams are shown in Figure 2, [14].

Maximum load and position of rupture obtained from numerical simulation were analysed for all sub-models in order to obtain the best predictor of the experimental results. The numerical simulation of knots as holes presented the best predictions with respect to results from experimental beams when knots were positioned in the tension side of the beam. There was a strong correlation between the numerical prediction and the experimental results for the maximum bending load (r^2=0.88), Figure 2.

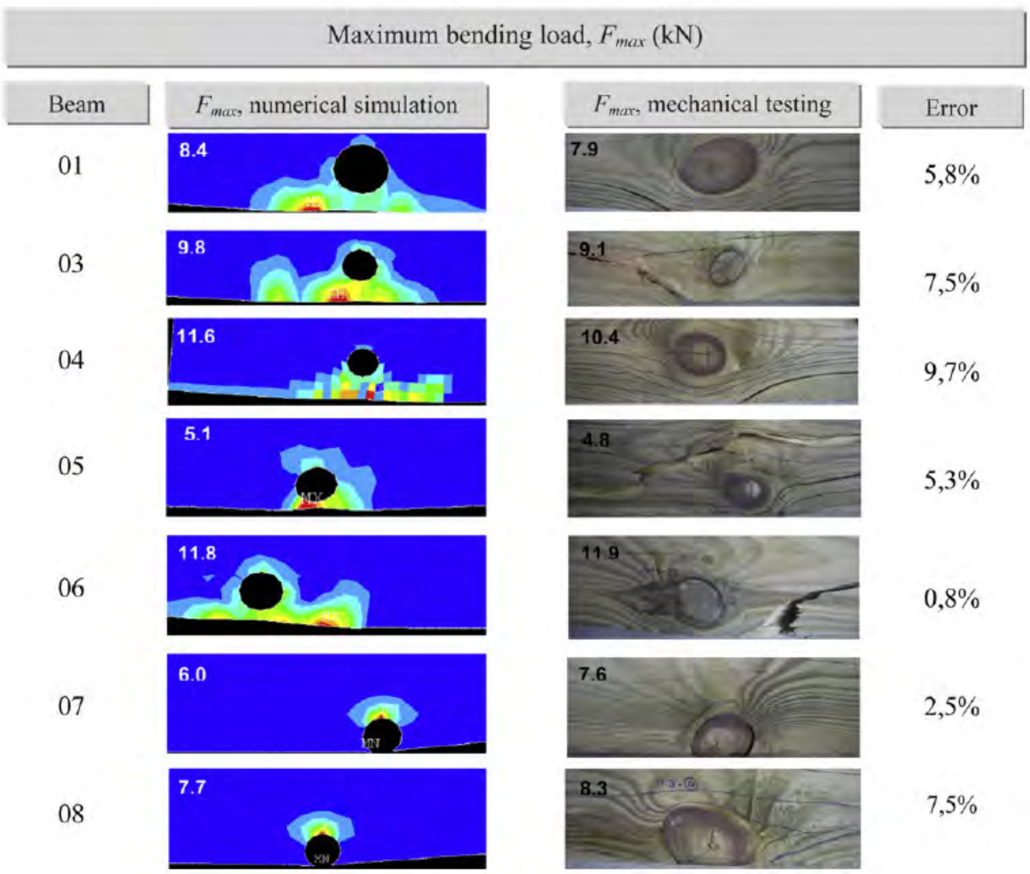


Figure 2. Numerical vs. experimental test results for maximum bending load

3.2 Results of the virtual beams

From the numerical results of the virtual beams, the following studies are being carried out: study of the variation of bending strength for different types of knots modeled, analysis of the influence of knots on the stress distribution in a beam cross-section, analysis of the influence of normal and shear stresses in the rupture criterion and finally the analysis of the variation of MOR according to the sizes and positions of knots.

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