Bern University
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COST Timber Bridge Conference – CTBC 2014

25–26 September 2014
Bern University of Applied Sciences
Biel, Switzerland

Edited by
Steffen Franke
Bettina Franke
Robert Widmann

EMPA Materials Science & Technology

COST European Cooperation in Science and Technology

▸ Institute for Timber Construction, Structures and Architecture

COST ACTION FP1004 and FP1101

COST TIMBER BRIDGE CONFERENCE
CTBC 2014

cost FP1101
ASSESSMENT, REINFORCEMENT AND MONITORING OF TIMBER STRUCTURES

Berner Fachhochschule
Haute école spécialisée bernoise
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PREFACE

Bridges are the most demanding of structures. They are often long-span, subject to large and variable dynamic loads, exposed to the weather, require low maintainance and, being seen as “gateway” or “landmark” structures by the public, are subject to close scrutiny.

Timber bridges occupy a special place within the timber engineering community. For many professionals involved in this field, from the planning engineer, to the timber manufacturer and to the responsible for the operation and maintenance – the owner -, timber bridges represent the highest achievement in the art and science of wooden architecture and construction.

Within Europe (and in the rest of the world) there are many different approaches to deal with this topic. The history of the wooden bridges has seen many stories – for example the ones old bridges which have survived for centuries, and of those in need of repair or even demolition after only a decade of being in use. The combination of large spans with heavy loads and major climatic actions on bridges is a big challenge for everybody involved in planning, constructing and maintaining these special timber structures. This requires that all effected parties have, in addition to the possibly large emotional devotion to such an object, above all, an extensive knowledge and experience in the planning, implementation and maintenance of timber bridges. Since timber bridges are not built in a dozen every day, it is important that an active exchange of knowledge between parties takes place. This is what the COST Timber Bridges Conference CTBC2014 in Biel aims to contribute to.

At this conference, which is jointly supported by the COST Actions FP1004 and FP1101, young researchers as well as old hands in the field of timber bridges get the opportunity to share their own knowledge while –at the same time- learning from others. The joint participation of the two Cost Actions, FP1004 focusing on Enhamcement of mechanical properties of timber products and structures, FP1101 focusing on Assessment, Strengthening and Monitoring of existing timber Structures, ensures that a vast field of knowledge is being covered. This ranges from the enhancement of wooden materials and components for new timber bridges to the assessment, strengthening and monitoring of existing bridges, both modern and historic ones. Following the conference, in which the theoretical foundations are highlighted, a field trip and excursion to existing old and new timber bridges will provide the opportunity to learn from real cases and continue the knowledge exchange on-site.

The programme of the conference is designed to stimulate and facilitate discussion from the delegates, and we hope that it will contribute to build bridges between young and old researchers, theorist and practitioners as well as between fellows from regions with high timber bridges "populations" and people coming from regions where even the term "timber bridge" sounds very strange. Besides the genuine interest for the subject, that gave us the impetous to organize this event, building these last mentioned bridges is a main goal of the COST framework.

Let's go for it!

Representing the organizing committee

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Steffen Franke – Organizer and Host

Bettina Franke – Organizer and Host

Dina D'Ayala - Chair COST Action FP1101

Richard Harris - Chair COST Action FP1004

September 2014

About COST Action FP1004

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

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

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

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

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

- 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

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

- 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.

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

- 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

About COST Action FP1101

Assessment, Reinforcement and Monitoring of Timber Structures

In recent years, the use of timber in structures has become particularly important, considering that it is the only truly renewable building material and carbon storage. Timber has been used as structural material for centuries and numerous examples demonstrate its durability if properly designed and built and when adequate assessment and monitoring has been applied. The objective of the Action is to increase the acceptance of timber in the design of new structures and in the repair of old buildings by developing and disseminating methods to assess, reinforce and monitor them. The need for assessment, reinforcement and monitoring of timber structures can arise from multiple motivations such as the expiration of the planned lifetime, materials aging, exceptional incidents, and ever more important, a change of use. The Action will benefit from multidisciplinary views of the problems and followed innovative solutions by the involved stakeholders, enable synergies between them and provide an effective way of discussing and disseminating the results from ongoing projects within this research area to the European industry. The Action will increase the confidence of designers, authorities and end-users in the safe, durable and efficient use of timber and consequently increase its use in construction.

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

WG 1: Assessment of Timber Structures

- Increasing knowledge and experience in non- and semi-destructive techniques and combinations thereof to improve applicability of results to asses the remaining structural capacity;
- Compiling methods which deliver reliable and robust results that can subsequently be incorporated into analytical and probabilistic structural models;
- Promoting the cross-validation of data obtained during inspection using similar methods in different projects;
- Combining visual grading, vibrational methods and mechanical tests, for the decay characterisation and the mechanical characterisation of the material;
- Developing specific in-situ grading standards to both estimate individual member strengths as well as obtain accurate quantification of deterioration.

WG 2: Reinforcement of Timber Structures

- Identifying and categorizing types of deterioration, damage and failure of timber structures, weak zones and their relevance for safety;
- Facilitating the decision-making process for choosing an appropriate reinforcement method with consideration of cultural heritage aspects, creating a handbook of reinforcement solutions for the main categories of problems;
- Analyzing the relationship between reinforcement techniques and protection technologies, as i.e. coatings to prevent decay and/or fire resistance;
- Evaluating solutions regarding their function as seismic reinforcement (energy dissipation capacity) and their suitability in the preservation ofcultural heritage (re-treatability, material compatibility);
- Developing computational concepts that allow for safe and reliable design of reinforcing measures.

WG 3: Monitoring of Timber Structures

- Identifying relevant properties that should be monitored, including indoor and outdoor environments and correlation with structural monitoring;
- Clustering long term experiments according to the involved risks as basis for the development of advanced diagnostic tools and technologies;
- Adapting digital image processing, remote and continuous data acquisition and early warning systems;
- Defining criteria for the efficiency control of the proposed and /or applied monitoring approach by means of numerical simulations and/or field and laboratory testing;
- Developing practical-operative guidelines and monitor schemes for the maintenance of timber structures, e.g. survey, documentation, on-site inspections decision making guidelines

This Action will advance and disseminate the knowledge regarding the assessment, reinforcement and monitoring of timber structures, specifically through:

- Benefiting from innovative methods and technologies that are available for other building materials worldwide but are not being adapted to timber structures;
- Maximising and coordinating research and innovation, and broadening the knowledge for the assessment, reinforcement and monitoring of timber structures;
- Disseminating the harmonized knowledge by developing guidelines for assessing, reinforcing and monitoring timber structures.

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COST ACTION FP1004

Review on on-site splice joints in timber engineering

Martin Cepelka[1], *Kjell Arne Malo*[2]

Summary

There are various examples of use of moment resisting splice joints in the building industry. These are mostly placed in planar structures (frames, arches) and indoor conditions. There are fewer examples of usage for timber bridges. Introduction of moment-resisting splice joints in timber bridge engineering would allow new design possibilities and thus increase competitiveness of timber bridges. This paper aims to review different known solutions for on-site splicing and hence give a basis for further development. Five connection techniques with possible application for timber bridges are shortly introduced. Pros and cons are discussed and special focus is laid on use in moment resisting joints. The efficiency of connection is described by efficiency factor μ = capacity of connection / capacity of connected timber members. Moreover, two examples of splice joints recently used for timber bridges in Norway are presented.

Key words: splice joints, moment resisting connections, rotationally stiff joints, timber bridges

1 Introduction

Development of glued laminated timber (glulam) and block gluing, where elements are in parallel, brings an opportunity for production of timber elements with nearly unlimited cross-sectional dimensions. The fact that glulam has an excellent strength to weight ratio compared to steel and concrete, promotes glulam to be used in structures with large spans. However, length of timber elements is limited due to production and transportation issues. In case of pressure impregnation, size of pressure treatment vessels is a limiting factor as well. To obtain large spans, either truss structures or splicing of elements are necessary. Truss structures are characterised by a high number of connections which are expensive and time consuming. When used in outdoor conditions such as timber bridges, connections have increased risk of decay due to moisture exposure.

Splice joint (in some literature defined also as end or butt joint) is a connection technique used to assembly two elements end to end so as one continuous element is created. When carried out on building site, elements can be either connected at its final location in the structure or pre-connected on ground and lifted in its entire length. When assembled on ground, additional joint manufacturing can be carried out with climate controlled conditions and by more heavy manufacturing tools in production tents. As a result, gluing technique can be used and high precision can be achieved which is especially of importance for moment-resisting joints.

Splice joints are nowadays mostly used to carry shear and axial forces and considered often as pinned joint in the design, even though geometry of connection introduces restriction in rotation and thus a certain bending stiffness. It is assumed that the expected rotations are small and initial slip of connection allows certain rotational deformation without presenting extra significant stresses. Recent development in connection techniques demonstrates the ability to achieve rotationally stiff joints with high load carrying capacity and ductile behaviour.

2 Review of on-site splicing techniques

Slotted-in steel plates and dowels

Timber elements are connected via steel plates which are mounted to timber by dowel type fasteners. Steel plates can either be placed externally (forming steel brackets) or slotted into timber members. Steel brackets are due to durability issues not recommended for use for bridges since a limited possible moisture dry-out in interface between steel brackets and timber surface. Weathering exposure of steel/timber interface is omitted by slotting steel plates inside timber elements. Connection technique using slotted-in steel plates and dowels has recently been widely used in modern timber bridge design. The joint is usually built up of several steel plates of thickness 8 or 10 mm and dowels of diameter 10, 12 or 16 mm [1].

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The technique is commonly used to carry axial and shear forces as for example in nodes in truss structures and hinges in arches. Figure 1 presents a splice joint used in Kjøllsæter Bridge, Norway. This is a typical example of node in truss structure carrying axial forces by using slotted-in steel plates and dowels. Gap between end faces of elements is filled with acrylic mortar which allows a direct contact pressure transmission and protects end faces from moisture.

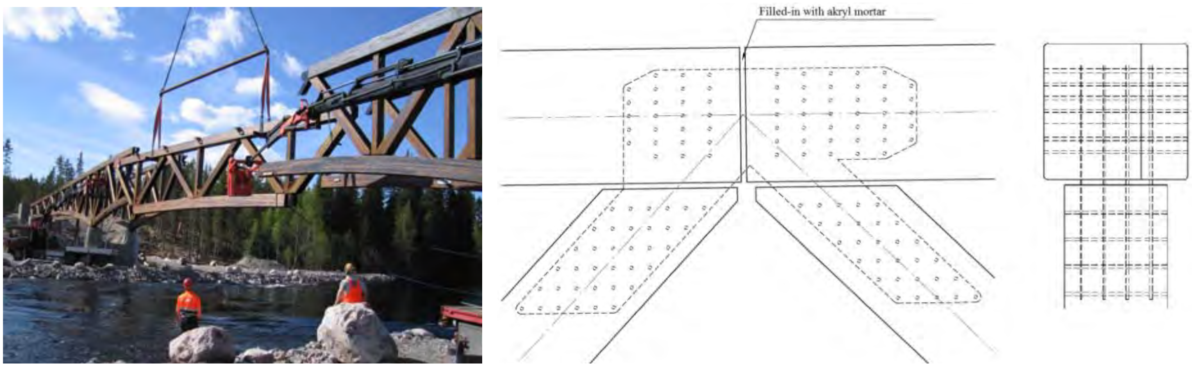


Figure 1: Kjøllsæter Bridge (2005), Norway, left: truss structure under assembling, centre and right: detail of connection at top chord, source: Moelven Limtre AS and Sweco Norge

On the contrary, there are not many examples of splice joints carrying bending moments. Figure 2 presents a moment-resisting splice joint used in Leonardo Footbridge, Norway. Slotted-in steel plates and dowels at top and bottom edge together with a shear key placed in the middle of the sections are implemented to carry bending moments and shear force respectively. The shear key consists of welded steel gusset connected to timber by a combination of transversal dowels and rods glued-in from the end face. Axial compression force is transmitted by contact of end faces since gap between elements is filled with acrylic mortar. Weathering degradation and creep of acrylic mortar is a common concern because additional contraction of mortar overloads dowels which are not designed to carry axial force. However, a direct contact of timber end faces would set a high requirement on manufacturing precision, and in reality, there will always occur a surface inaccuracy to some extent between such large elements. Regular inspection of joint is therefore important. Upper sides of Leonardo footbridge arches were later protected by metal cladding to prevent water from leaking into possible gaps or cracks.

Figure 2: Leonardo Footbridge (2001), Norway, left: the central arch under assembling, right: 3D model of the splice joint

When transmitting bending moments, dowel-type fasteners impose concentrated local forces in timber in an angle to grains [2]. The bending resistance is then governed by a combination of tension perpendicular to grain and longitudinal shear stresses – two weakest strength properties of wood. Risk of a local failure by tension perpendicular to grain, so-called “splitting”, is related to a slenderness and number of fasteners in a row parallel to the grain. The slenderness is defined as a ratio of an embedded length in timber to a diameter of fastener. The less slender the fastener is the higher concentrated forces are imposed to timber. The increasing number of fasteners parallel to the grain increases risk of splitting. Splitting tendency can also be initiated by shrinkage cracks. The failure mode by splitting is brittle and very low ductility of connection is hence achieved. Adequate spacing and end-distances must be ensured to prevent splitting of timber. Requirements to spacing of fasteners often lead to large area of connections and thus increase the necessary timber dimensions. Efficiency factor of connection consequently decreases. The efficiency is usually stated in a range μ=0,4-0,6 [3] . Generally, higher number of thinner plates and lower diameter of dowels provide a smoother stress distribution and more ductile behaviour.

For moment-resistant connections, besides capacity, stiffness and ductility play major roles. Although the holes to accommodate dowels should be tight fitting to allow for a direct contact between dowel and timber, material tolerances and requirements to fast mounting make tight-fitting unreliable. Moreover, if dowel fits too tight in hole, splitting can be initiated. Dowel-type connections might therefore be accompanied by a significant initial slip and an unreliable stiffness [3].

Above mentioned shortcomings can effectively be enhanced by reinforcing the joint area. Recent research reports an excellent reinforcing effect of self-tapping screws placed perpendicular to grains. F. Lam et al. report in [4] increasing capacity of column-joist connection by factor 2 for monotonic loading and observe a very ductile failure mode. In a wide series of bending tests, splitting did not occur in any specimen even for a cycling loading. The failure mode was a plug shear on the tension side of the beam. This indicates that screws have the capacity to carry imposed stresses perpendicular to grain direction, thereby changing the failure mode to parallel to grain failures. Later in [5], effect of increased bolt dimensions and decreased edge distances were studied. The results show an amplifying effect of reinforcement and thus an additional increase in capacity while maintaining the ductile behaviour. The efficiency factor in bending of μ=1 was achieved, making the timber joist the weakest part of the connection. F. Brühl et al. report in [6] and [7] a highly ductile behaviour and remarkable rotation capacity of splice connection in tension and bending tests. Series of tension tests with a various dowel arrangement have been carried out to study behaviour of a group of fasteners. Test results have been standardized to a bearing resistance of a single fastener. The results show a good accordance of an initial stiffness and a bearing resistance independent of the dowel arrangement. It is also shown that the effective number of fasteners is equal to the installed number and hence that it is not necessary to reduce the number of fasteners in a row parallel to the grain as required now in Eurocode 5 [8]. The bending tests show a high rotation and moment-carrying capacity. Higher capacity based on a larger lever arm was achieved with alignment of fasteners where more fasteners were placed in fewer rows. With such alignment splitting failure on the edge distance of the connection was observed. However, the failure was still ductile, since wide opening between connected parts had already been developed before splitting occurred.

Glued-in rods

Load transfer from rods embedded lengthwise to timber is characterized by a good "flow-of-forces". Stress distribution is provided by means of shear force continuously along the rods. The rods are fully embedded in timber members and thus protected. The predrilled holes are filled with epoxy resin and there is therefore no clearance between bars and wood. Initial slip is thus avoided. High strength epoxy resins are available on market. High stiffness and pull-out capacity is hence achieved. For large joints, multiple rods are necessary and brittleness of adhesive could lead to a progressive failure in a group of rods. Ductile behaviour is therefore very important. Series of bending tests with different layout of fasteners have been carried out by N. Gattesco et al. and results are presented in [9]. A very ductile behaviour was achieved by using low grade steel bars for monotonic tests. However, for cycling loading, after the yielding point of bars is reached and bars are subjected both to tension and compression, instability failure occurs by lateral bending of bars. As a result, brittle failure of timber in lateral direction is developed.

Figure 3: Splice joint with glued-in rods, left: rods glued into one member of the joint, right: plastic deformation of the joint, [9]

The shortcoming of usage of glued-in rods is a production difficulty. In the above shown example, rods were put inside holes using a wire guide to centre them. The epoxy was injected through holes drilled perpendicularly to each embedment hole near its end. For large glulam sections, more rods placed in one layer beside each other would be necessary making the injection of bars not placed near edges difficult. Effectiveness of the grouting operation cannot be visually checked. Experience in reviewing failed joints due to inadequately mixed and incorrectly applied epoxy on site limits the production to a climate controlled environment with quality control and skilled personnel [10]. E. Gehri presents in [11] a jointing technique combining glued-in bars and pinned steel connections. The principle of this so-called GSA®-technology is shown in the Figure 4. Rods are glued to timber sections in factory conditions with welded or screwed special steel pin-joint at the end. Pinned steel-to-steel connection is thus created with very fast on-site mounting. The ductile behaviour is achieved by a combination of low-grade steel and pre-defined glue-free deformation zone of rods. Uneven distribution of forces in a group can thus be absorbed by plastic deformation of rods. Determinant for the pull-out capacity of the rods is shearing capacity of the timber. Reinforcing of high stressed areas at edges by using hardwood lamination can increase the load bearing capacity of connection up to efficiency μ=1 with simultaneous ductile behaviour.

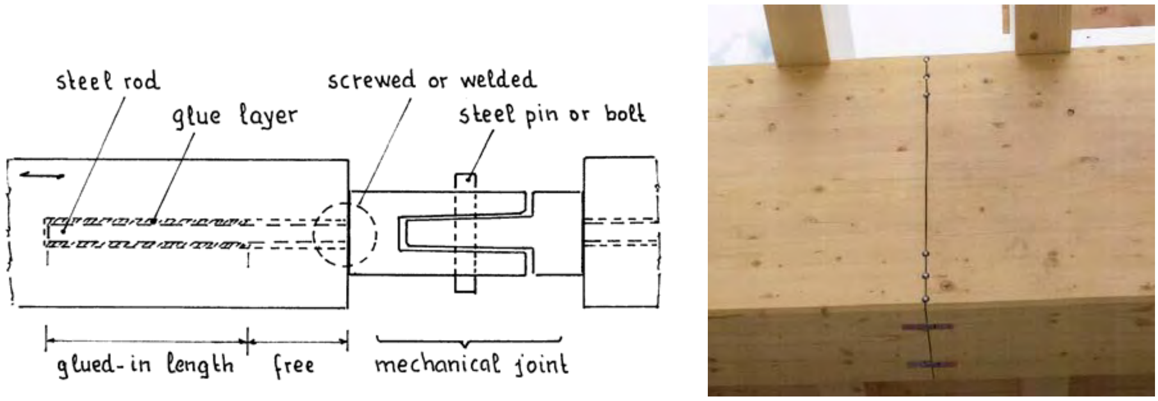


Figure 4: GSA®-technology, left: the principal of joint, right: moment-resisting splice joint of a beam, [11]

Connection with glued-in rods presents a very complex system with a specific stress distribution since three different materials with distinctly different properties are combined. M. Stepinac et al. describe in [12] the current state in design, research and industry difficulties on the basis of comparison of design rules and online survey sent to scientists, timber industrialists and structural designers all over Europe. It is concluded that despite many research projects, there are still a lot of outstanding issues that are not clarified and where common agreement has not been reached. Lack of standardized test setups leads to uncertain conclusions from conducted tests and hinder further investigation of problems such as duration of load, fatigue, interaction between axial and lateral load and dynamic climatic tests. As a result, there are still no universal design rules and technical guidelines. In 2003 it was decided to discard the Annex C in Eurocode 5-part 2 [13] which was dealing with design of glued-in rods. There are, up to date, no design rules in current version of Eurocode 5.

Self-tapping screws and threaded rods

Self-tapping screws (STS) and threaded rods are characterized by full threads and hardening after rolling the thread. Hardening increases tensile strength and thus bending and torsion capacity. STS are manufactured usually up to diameter 13 mm and used without pre-drilling of holes. Threaded rods are usually of diameter 16 or 20 mm and are installed in pre-drilled holes.

Great performance of STS as a reinforcement in perpendicular to grain directions has recently been well-documented by a current research. There are, however, few examples of research and references of STS as a primary fastener in splice joints. Following Figure 5 presents a possible design of tension splice joints with STS. The two details in the left are proposed by a screw manufacturer SFS Intec based on a steel-to-timber solution with so-called WR screws. The design in the right is a direct timber-to-timber splicing presented by E. Gehri in [14].

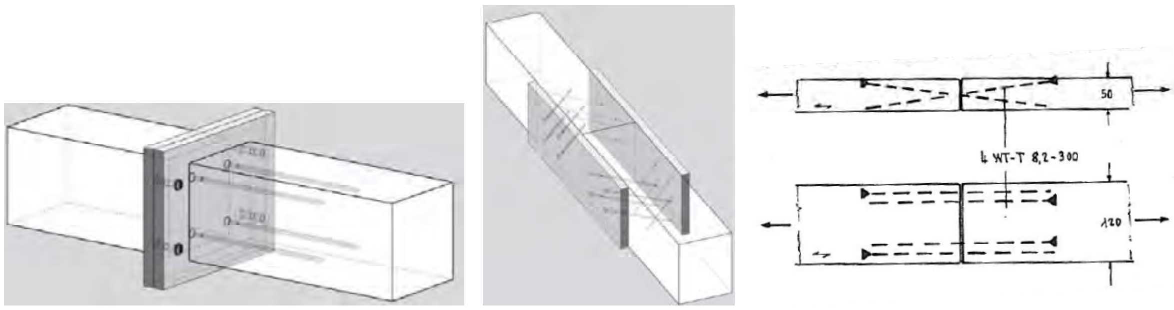
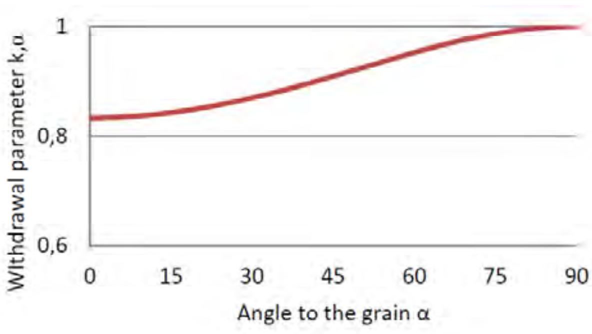


Figure 5: Tension splice joints, left and centre: steel-to-timber splice joint: source: SFS WR reference brochure, right: splice joint timber-to-timber, [14]

Screws and threaded rods are intended to be axially loaded since they show great withdrawal and pushing-in resistance. Proper positioning of fasteners relative to the direction of grains and loads significantly affects both load-carrying capacity and stiffness of the connection. Withdrawal capacity at an angle to the grain is described by the Hankinson function used as a basis for equation (8.38) in Eurocode 5 (in amendment A1) [8]. In the Figure 6, the reduction of withdrawal capacity relative to the angle to the grain is demonstrated by a fictive factor k_α. It should also be mentioned that Eurocode 5 [8] only provides the design values for both withdrawal and pull-through resistance of screws for the angle to grain $\alpha \geq 30°$.



$$k_{\alpha} = \frac{1}{\sin^2 \propto +1{,}2\cos^2 \propto}$$

Figure 6: Hankinson function as used in Eurocode 5, amendment A1

In [15] and [16] H. J. Blass demonstrates, for a simple lap joint transferring a tensile force, the effect of inclination of screws under degree of 45º or 90º relative to the loading (in this case also grains). It is concluded that connections with screws inclined under 45º show an increase of 50% in the load-carrying capacity and increase of slip modulus by a factor up to 12 compared to the case of screws loaded perpendicular to their axes. The failure mechanism is shown in the Figure 7. In the left, the force is mainly transferred by an axial withdrawal force in screws and a compression force component between the timber members based on a truss-like system equilibrium. In the right, the load-bearing capacity is governed by bending capacity of screws and timber embedding strength.

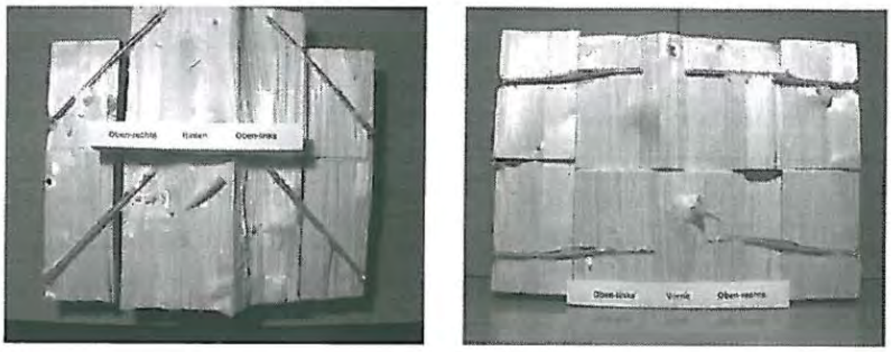


Figure 7: Deformation of tension lap joint, left: with screws inclined under 45º, right: with screws inclined under 90º, [15]

The benefit of a high withdrawal and pushing-in resistance of STS is well utilized by using so-called tension and compression plates (on market known also as ZD-plates), which distribute load to screws in force couple so as each screw is loaded either by a pure tension or compression axial force. The principle of the connection is demonstrated in the following Figure 8 on a trademarked SWG system. Screws are driven into timber under angle of 30º via a “base-part” of steel ZD-plate. Then a “top-part” is applied and the ZD-plate is bolted to a steel bracket.

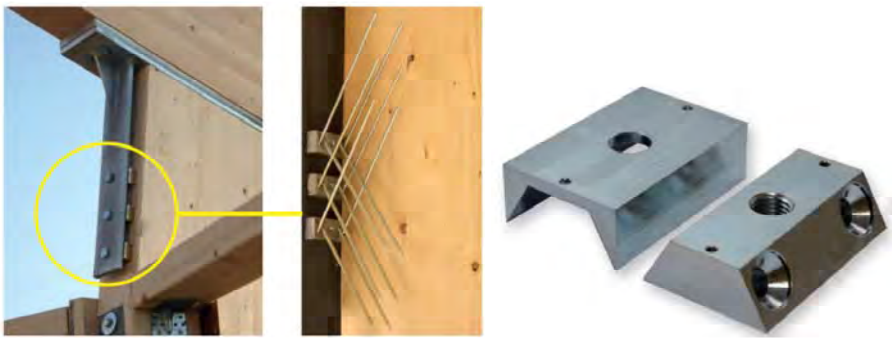


Figure 8: Joint with ZD-plates and STS screws, left: portal frame joint, centre: detail of screw assembly, right: ZD-plate, source: www.swg-produktion.de

M.Closen et al. present results of an experimental study on moment-resisting joist-column connection with STS and ZD-plates in [17]. The results show an excellent moment capacity with efficiency μ=2 related to a design moment capacity of joist and very stiff behaviour. However, a relatively low ductility of the connection was observed.

Ongoing research at NTNU University in Trondheim, Norway shows a great potential of long threaded steel rods used as a primary fastener in moment resisting joints. P. Ellingsbø et al. present in [18] results of an experimental test of a steel-to-timber connection of a glulam cantilever beam. The cantilever beam is a model of a propeller in a water turbine and must be designed for large moment actions at the connection. The principle of the connection is to achieve a steel failure in rods by introducing a sufficient embedded length of rods. Small variation in steel properties would then provide a reliable connection with predictable behaviour.

Prior to test of the connection a withdrawal test of 16 mm rods was carried out. Results showed that a failure mode changed from timber to steel when the effective embedded length exceeded 600 mm and with length more than 800 mm a steel failure was exclusively observed. Finally, an experimental model was conducted with a simplified geometry of the connection. Long threaded rods were installed into pre-drilled holes via a steel plate. The tension force was carried by one or two rods respectively in different test setups. The rods were inclined under 30º in angle to the grain. Based on results from withdrawal tests an effective embedded length of 1000 mm was chosen. Compression was transmitted by a contact pressure at the end face of the beam. Since a cracking occurred at the compression side, another rod in angle to the grain of 60 º was applied to prevent splitting due to stresses perpendicular to grain. The results showed that when one tension rod was applied a steel tension rupture exclusively occurred proving the findings from the withdrawal tests. However, for a case with two rods, a group effect was observed decreasing both rotation capacity and rotational stiffness. Further tests with multiple rods are necessary to describe the group effect more closely. Effects of moisture content on the stiffness and capacity were studied for specimens with 12 and 24% moisture content. The different moisture content had no effect on the capacity of the connection. The rotational stiffness slightly decreased with the increased moisture content.

Glued connection

The splicing technique under the trademark HESS LIMITLESS was presented by S. Aicher et al. in [19] and [20] and it is covered by a German and international patents. The joint is formed by a combination of large finger joint, wedge shaped fitting and high premium (P-) lamination. At present the maximum height of the section is limited to 2 m. The factor of efficiency of the joint μ, in in-plane bending and shear, is as high as 1. High precision in manufacturing is required for cutting and gluing of the large finger joint. The production on site therefore takes place in climate controlled and specially equipped tents.

The splicing technique is presented in the Figure 10. The wedge shaped fitting with height H/6 (where H is the height of the entire cross section) is placed at the tension edge of the section and it is glued to bottom parts of both connected members. The remaining 5H/6 height of the section is joined by the large finger joint.

In case of loading by tension or compression parallel to the grain or out-of-plane bending, the wedge shaped fitting must be placed on both edges. The efficiency factor of the corresponding capacities is then decreased to

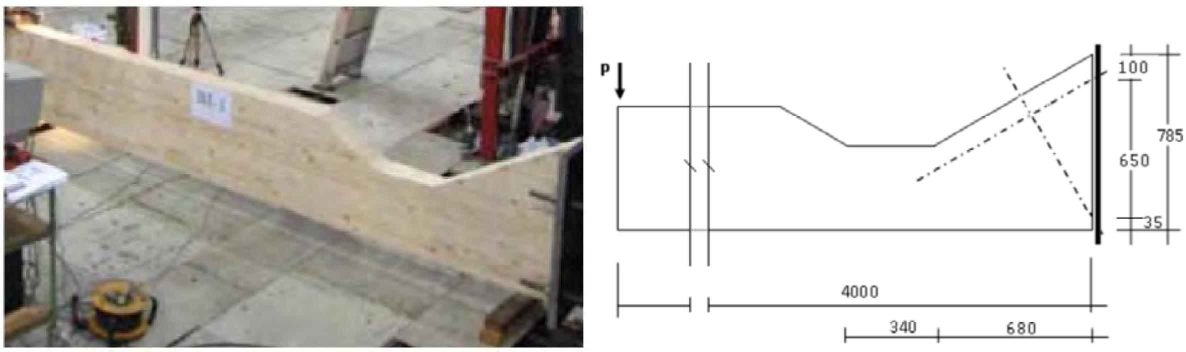


Figure 9: Test setup for cantilever beam connected by threaded rods to a steel plate, [18]

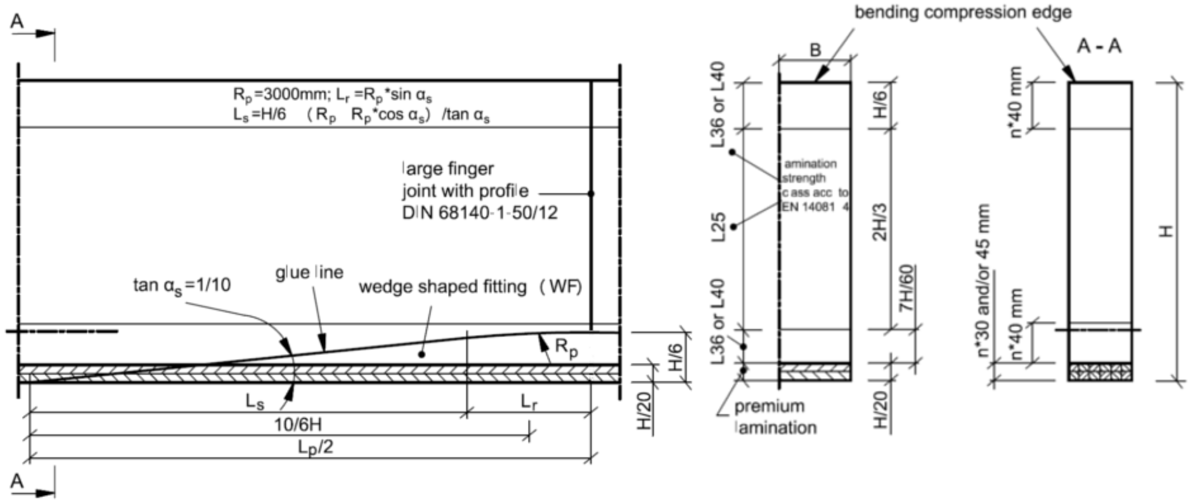


Figure 10: Principle of glued joint of high efficiency, [19]

μ=0,9 and μ=0,85 respectively. Glulam beams are build-up of different strength class laminations. The outer bending and compression parts are composed of L36 or L40 laminations, while the inner part consists of L25 laminations. Resulting class of the glulam beam is GL35c with L36 laminations at outermost edges and GL38c with L40 laminations.

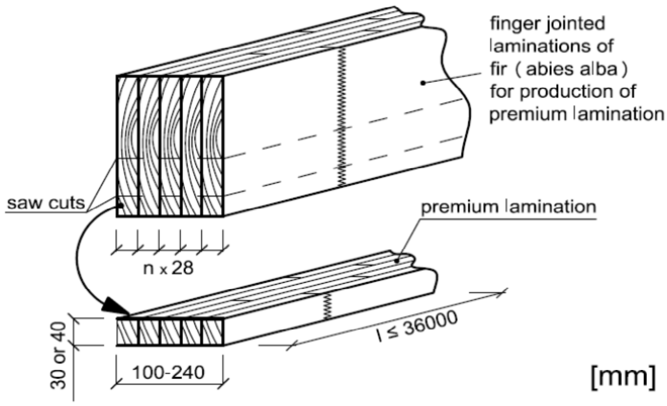


Figure 11: Premium laminations, [19]

The outermost tension edge of both beam and wedge shaped fitting is made of so-called Premium (P-) laminations. P-laminations contribute essentially to the load carrying behaviour of the joint and beam. By high tension strength and low MOE related to bending strength, the strength raising stress distribution is achieved. As shown in the Figure 11, the components of P-laminations consist of parallel arranged finger jointed fir laminations. The required bending and tension strength is $f_{m,k} \geq 45$ MPa and $f_{t,k} \geq 29$ MPa and mean MOE in tension is equal to $E_{0,mean}=12500 \pm 500$ MPa.

BVD anchor bolt connection

The connection consists of cylindrical steel or gusset anchor bolt (on market known also as BS-connector) and orthogonally placed dowels in 2 directions. The dowels are placed in half circle holes in the anchor bolt and a rigid load transfer is achieved by injection of a high-strength, non-shrinking cement grout via infill hole in the anchor bolt. Depending on load amplitude, anchor bolts differ in length, diameter and amount of incorporated dowels (4-24 pieces). Several anchor bolts can be placed in parallel in large joint; this is, however, not covered explicitly by the technical approval at present [19]. Load is transferred from timber members via dowels to the anchor bolt which is consequently loaded by an axial tension or compression force. The German building approval specifies that BVD anchor bolt connection shall not be loaded by shear force except from self-weight of respective timber members.

Compared to connections with slotted-in plates and dowels, a higher capacity is achieved due to more effective load transfer by two-directional placement of dowels. Efficiency in matter of net cross-sectional area is also higher since predrilled holes for anchor bolts occupy less area than slots for steel plates. Anchor bolt is fully embedded and thus protected in timber element and there are no slots in upper and lower faces of timber where water could leak into section as in case of slotted-in plates. However, the common failure mode of the connection is, alike for all dowel type fastener connections, due to premature splitting of timber. S.Aicher et al. report in [19] an efficiency factor for tension resistance calculated according to German approval in the range of μ=0,35-0,54 depending on different anchor bolt types and cross sections.

The rather low joint efficiency is caused by splitting failure due to tension stresses perpendicular to grain. Therefore an enhancement by application of reinforcement by lateral self-tapping screws has been studied in series of tests. The test results prove that no failures due to premature splitting occurs and a common failure mode is associated with a fastener yielding combined with a block shear or a tension failure in the wood. Thus ductile behaviour with considerably higher failure loads are achieved and efficiency factor increases to μ=0,7-0,9.

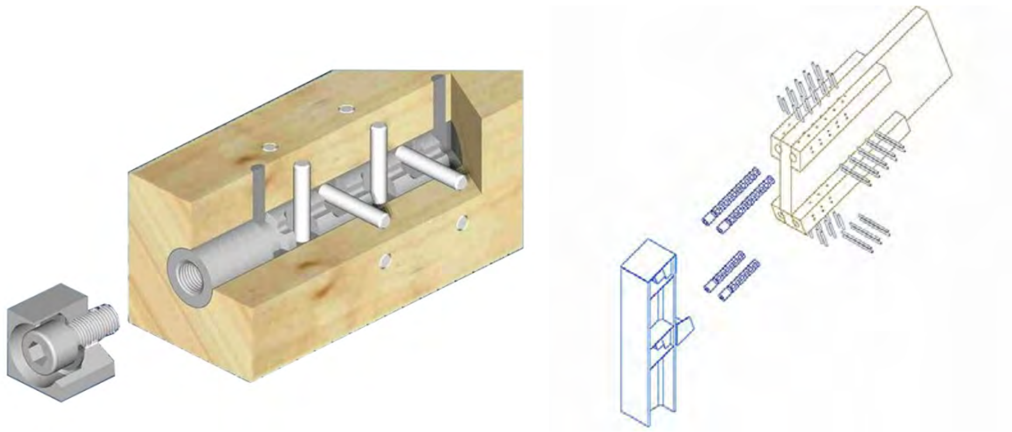


Figure 12: BVD anchor bolt connection (example for use in hybrid steel/timber joint - for splice joints, the anchors in timber members are mutually connected by a connecting steel part), Source: www.bertsche-office.de/us

3 Conclusion and future work

Development of moment-resisting splice joints for large dimensions glulam elements is of interest to achieve longer spans. Given examples of research and sufficient use in the building industry for various connection techniques give a good basis for a wider implementation of splice joints into timber bridge engineering praxis. Further development is nevertheless needed for use in timber bridges. For arches for example the resistance to out-of-plane bending moment and shear force as well as behaviour under combination of stresses in different directions need to be closely studied. Care must be taken to detailing for durability issues under weathering conditions corresponding to service class 3. Moisture movement and thus moisture induced stresses tangentially and radially to grains in large glulam cross sections must also be respected in the design. The requirement to the design working life of such connections is for instance in Norway 100 years.

4 Acknowledgment

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Review of pre-stressed timber bridge decks: durability, lay-out and structural systems

Francesco Mirko Massaro[1], *Kjell Arne Malo*[2]

Summary

Stress-laminated timber bridges are more and more used nowadays, but the knowledge about their long-term behaviour are still not sufficient. The present review shows critical details of the timber bridges with regards to durability. Despite some of them are very common, there is still a need to improve their properties to limit the effects of high moisture content. Decks are often protected physically and chemically against external biological attacks, but environmental issues have raised questions about the use of toxic chemical treatments. Moreover, it is known that the mechanical pre-stress level decreases during the lifetime. All these issues have to be investigated to guarantee long technical lifetime.

Key words: Stress-laminated timber decks, durability, water traps, moisture content, tension level, lifetime.

1 Introduction

Durability of timber bridges is a key factor for their design. Timber bridges in the EU should in general have a minimum lifetime of 100 years and we have good examples of well-designed and durable bridges.

In the present paper a review is made of the stress-laminated timber decks with special attention to risk of elevated moisture in the bridges, use of preservatives, loss of pre-stressing, local damages, maintenance and poor details. All these elements are relevant with regards to durability.

Stress-laminated timber decks were used for the first time in Canada in the late 70's and later in the USA and in the Nordic European countries. Timber laminations are held together by pre-stressed rods that are generally inserted into pre-drilled holes in the middle of the depth of the deck.

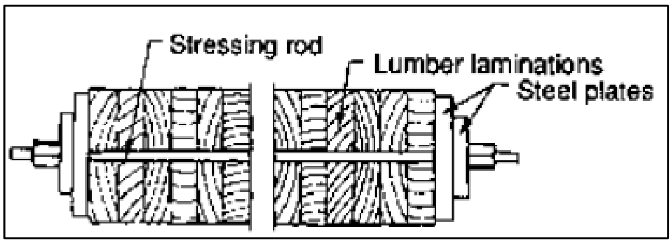

Figure 1: Section of a pre-stressed timber deck [1]

The pre-stressing generates lateral stress between laminations which then work as a plate due to the friction between the lamellas, giving a better distribution of loads. However, there is a reduction of longitudinal stiffness due to the presence of butt joints of the lamellas. They extend the laminations in longitudinal direction but they cannot transfer bending moment so the full flexural capacity is reached only at a certain distance from the butt joints.

This kind of system for timber decks is used both for new constructions and for improvements and restorations of existing bridges.

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Figure 2: Bridge with one row of pre-stressed rods

Figure 3: Bridge with two rows of pre-stressed rods

2 Mechanics of stress-laminated timber decks

The stress-laminated timber decks have to be considered as plates with orthotropic behaviour, according to [2].

Generally the elastic parameters E_L (modulus of elasticity parallel to the grain) and G_{RL} (shear modulus in longitudinal direction) are clearly specified in timber bridge guidelines and codes specification, whereas others parameters are still subject of research.

Ritter in [1] suggests to consider the transversal modulus of elasticity E_T and the in-plane shear modulus G_{LT} respectively equal to the 1.3% and to the 3% of E_L.

Eurocode in [2] also suggests the use of percentage values and give the values shown in Table 1 for softwood laminations.

Table 1: Eurocode suggested values for elastic parameters

Type of wood	E_T/E_L	G_{LT}/E_L	G_{LT}/G_{RL}
Sawn	0.015	0.06	0.08
Planed	0.020	0.06	0.10

The pre-stressing force F_{ps} necessary to give lateral stresses and avoid gaps between the lamellas, can be easily calculated considering that it has to support the transverse moment M_T given by the loads (1).

$$F_{ps} \geq 6\frac{M_T}{h} \tag{1}$$

Moreover, it has to be able to prevent vertical slips, thanks to the friction among the lamellas according to Eq.(2), wherein V_T is the transverse shear acting on the deck and μ is the friction coefficient.

$$F_{ps} \geq \frac{V_T}{\mu} \tag{2}$$

The respect of these two conditions is strictly necessary to allow the laminations to work as a plate.

The level of pre-stress is extremely important for stress-laminated deck because it generates the friction force between the laminations. The friction is commonly modelled by assumed Coulomb friction given by Eq.(3) .

$$F_{fr} = N \cdot \mu \tag{3}$$

The friction force F_{fr} is function of the compression N induced by the pre-stress and of the coefficient of static friction μ, which is a material property.

However, the pre-stressing force cannot be higher than the strength of the steel bar or the compression strength perpendicular to grain of timber under the anchorage plate.

It has to be taken into account that the pre-stressing force is not constant during the lifetime of the bridge but it will suffer variations due to the changes of temperature and moisture content. Furthermore, wood exhibits considerable creep which will decrease the pre-stressing force over time.

The above-mentioned variation ΔF_{ps} can be evaluated as [3]:

$$\Delta F_{ps} = \Delta\sigma_S \cdot A_S \tag{4}$$

where

$$\Delta\sigma_S = \frac{E_{90} \cdot A[(\alpha_{90} - \alpha_S)\Delta T - \beta_{90}\Delta m]}{\frac{E_{90} \cdot A}{E_S} + A_S} \tag{5}$$

In the equations above $\Delta\sigma_S$ is the variation of stress in the steel bars, A and A_S are the areas of timber and steel bar respectively, E_{90} and E_S are the modulus of elasticity of timber orthogonal to grain direction (long term mean value) and of steel respectively, α_{90} and α_S are the thermal expansion coefficient of timber orthogonal to grain and of steel, β_{90} is the moisture expansion coefficient of timber, ΔT and Δm are the variations of temperature and moisture content.

The influence of moisture content on the variation of pre-stressing force is higher than the influence of temperature, as explained in [4]. In Figure 4 and Figure 5 is shown the variation of pre-stress changing respectively temperature and moisture content. The expansion coefficients used for timber comes from [5]. Effectively, varying moisture content there is a huge effect on the pre-stressing force compared to the effect of temperature.

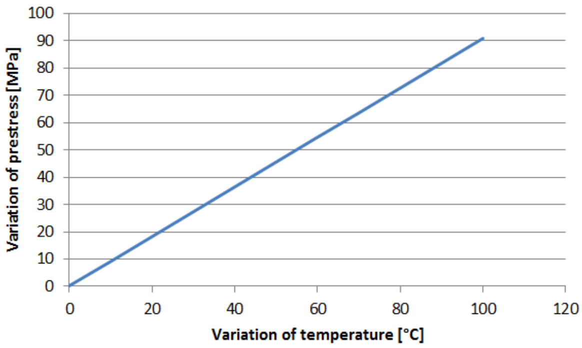

Figure 4: Influence of temperature on pre-stressing force

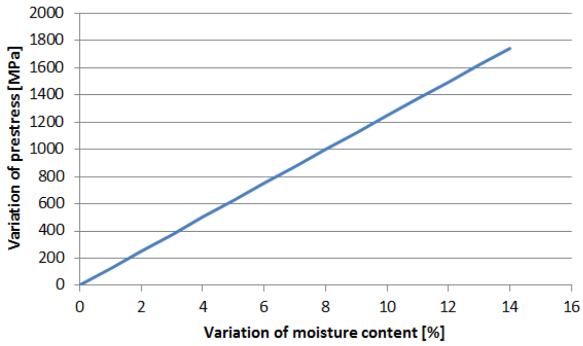

Figure 5: Influence of moisture content on pre-stressing force

3 Durability issues

In the last decades, the use of pre-stressed timber decks has spread in Northern Europe. Pre-stressed decks with creosote impregnation of lamellas are commonly used. Nevertheless there are still needs for further developments especially about durability issues. In some cases, the expected lifetime of the bridges is questionable due to unforeseen decay.

Recently, data about the actual conditions of existing bridges has been collected.

The comparison of collected data allows us to understand better the risks that affect the timber bridges and consequently tend to decrease their service life.

4 Moisture content and water traps

High moisture content is the main reason for decay and it can be due to improper protection.

It is known that timber can suffer biological attacks under certain conditions. Fungi grow only when temperature is over 0°C and moisture content (MC) is above 20%. Hence, moisture content exceeding 20% has to be avoided to prevent biological attacks. Moisture content is dependent on the relative humidity and temperature of the air.

Often improper design creates water traps in the deck which are the starting points for fungi growth and subsequent development of decay. Particular attention should be paid to structural details, especially in cases where it is difficult to inspect.

Edges and abutments should be designed with particular attention to avoid standing water. It could be useful create a little gap between the timber lamellas and the concrete to prevent water traps. Moreover, the joint between timber and concrete is also easy to inspect (Figure 8) [6]. However, this might necessitate changes in design as contact between the materials is often used to transfer both horizontal and vertical forces. In Figure 7, there is a gap between the parapet and the plate on the outer lamella. This allows the water to fall down preventing the accumulation of water on the detail. It will also ease more rapid drying of wetted wood.

Figure 6: Structural detail which should be evaluated with respect to water trapping

Figure 7: Good detail with respect to standing water

Figure 8: Free space below intersection between timber and concrete (Åsta bridge)

Figure 9: V-shaped column (Sørliveien bru)

V-shaped columns appear in several bridges, often due to structural reasons, but their design with respect to moisture trapping and on durability of timber should be evaluated. Water slides down through the column and it can be captured and stand in the concavity, increasing the moisture content. It could be useful to protect the inner face of the column in the proximity of the joints with physical covering. The cladding is commonly used to protect inclined frames near the connections, as it is shown in Figure 10.

Figure 10: Cladding of an inclined frame (Flisa)

Some physical and/or chemical remedies are usually needed to protect the wood against the climate loading. In Figures 3, 6 and 7, the outer lamella is physically protected to avoid standing water. In this way, the water falling from the deck is directed outwards and is passing the lamellas on the outside [7].

Furthermore, due to the possibility of the timber cracking is necessary to cover the upper surfaces of the bridges to avoid that the water can enter inside the cracks. The physical protection is often realized through a metal (copper, zinc) cladding. An example of cladding can be found in the Leonardo da Vinci Bridge (see Figures 11 and 12). The cladding was inserted after the appearance of fungi caused by the water entering in the cracks.

Figure 11: Arch of Leonardo Bridge before the cladding (NPRA)

Figure 12: Arches of Leonardo Bridge after the zinc cladding

To guarantee the lifetime, the use of preservative treatment is compulsory in some countries. Decks treated with preservatives are less vulnerable to attacks from biological organisms. However, the use of preservatives is becoming more restrictive due to possible effects on the environment.

The preservative can be oil-borne or waterborne. Generally, the first ones are more effective than the second ones. The most common chemical preservative is creosote. It is an oil-borne and coal tar-based product and it is very effective as preservative but its use is a topic for discussion in many countries. The use of pentachlorophenol and copper napthenate is an alternative to creosote. The waterborne preservatives are generally less effective but can be used for bridges or elements that require a shorter lifetime. In Norway, to guarantee the requested lifetime of 100 years, is common to protect timber with a double treatment. Timber is impregnated progressively with both kind of preservative [8]. In Sweden, and in some other countries, the use of chemical treatment containing creosote is forbidden and consequently bridge designers have to pay more attention to the details. The covering of the whole deck through waterproof membrane and metal cladding could be a solution to reach the requested lifetime [9]. But Swedish requirements are less strict (40-80 years) than some other countries as Norway (100 years), so there is the need for more information about the long term behaviour of this system.

5 Open decks and membranes

However, the chemical protection is probably not enough to secure timber decks very long lifetime. Decks should be covered with a waterproof membrane to avoid that standing water increases the moisture content.

The next pictures show examples of bridges, in which the deck is not covered with a membrane. It is possible to see the damage suffered by the timber lamellas due to standing water, even if they were treated with creosote. For this reason, open decks are not more used in new bridges.

Figure 13: Damage on deck due to standing water (Måna bridge)

Figure 14: Damage on deck due to water (Nesoddvegen bridge)

One of the most common membranes is the “Topeka”, which is a mixture of dried and heated stone material and bitumen and gravel, used also for wearing course that have to sustain heavy traffic level. But often Topeka might be softened by the creosote oil and for this reason it can be pushed aside by the tire pressure or braking forces can push the wearing course. Another used membrane is the “Micorea”, a polymer modified membrane, but blisters have been experienced and this can lift the asphalt layer. One of the modern technologies used is the combined mastic asphalt (Støpeasfalt) that has a higher content of filler than Topeka [10]. This is more stiff than usual and could crack in the winter but for the moment there is a limited experience to evaluate if it is watertight when this happens.

Figure 15: Støpeasfalt (left side) used on Møllendammen bridge

6 Dirt and vegetation

Water traps or weather conditions are not the only cause of elevated moisture content in timber. It should be paid attention on the cleanliness of the bridge. Indeed, dirt, debris or vegetation can raise moisture content of timber, if they are in its proximity. Structural details should be evaluated in order to prevent accumulation of moisture trapping materials close to the bridge. Furthermore, in order to guarantee lifetime, a good maintenance is recommended.

Figure 16: Structural detail which should be evaluated with respect to accumulation of dirt

Figure 17: Vegetation touching the timber elements

7 Tension rods

It is important to control the loss of pre-stress to assure the structural efficiency of the deck. It is difficult to increase the initial tension level because it would cause local wood damages. However, the level of tension decreases year after year due to the relaxation and therefore it is commonly accepted to re-stress the rods after about 25 years to obtain the initial level of pre-stress and guarantee the stiffness of the deck. There are not known cases of unexpected losses but the information are still limited.

There are some possible improvements for the anchorage systems to reduce losses. The reinforcement with screws reduces the loss of pre-stressing thanks to the deeper transmission of the load reducing local compressive load [11].

The pre-stressing nuts have to be hot-dipped galvanized, otherwise they could corrode with the sharp reduction of the lifetime. Moreover, the corrosion can spread into the rods, probably enhanced by copper ions dripped onto the pre-stressing rods.

However, to guarantee the requested service life, maintenance is always required and has to be planned during design.

8 Conclusion

The stress-laminated timber decks are very efficient but it is possible to see that there is still a big margin of intervention to improve durability, studying better details and solutions against standing water.

Questions related to pre-stress level could be solved with further studies regarding the interaction between tension rods and timber decks.

9 Acknowledgments

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Some Physical and Mechanical Properties of Laminated Wood Panels Manufactured with Nanoparticles Filled Poly (vinyl acetate) Adhesive

Deniz Aydemir[1], Nurgul Tankut, Ali Naci Tankut, Timucin Bardak, Eser Sozen

Summary

Laminated wood panels are one of the engineered products bonded with various layers of wood. These layers are laid and glued with all wood grains parallel by using different adhesives. The products are commonly used in different application such as furniture, timber bridge and wall panels. In this study, the effect of different nanoparticles on physical and mechanical properties of laminated wood panels bonded with poly (vinyl acetate) adhesives was investigated. PVAc adhesive was prepared with adding of SiO_2, TiO_2 and nano clays for 0.5%, 1%, and 2% loadings. Spruce (*Picea orientalis* L) and Oak (*Quercus robur*) wood species were used to produce the panels. The prepared adhesive was applied to wood layers, and then all layers were combined to obtain laminated wood panels which have 5 layers. The panels having 12% moisture content (MC) were tested to determine the physical properties such as water absorption, thickness swelling; and the mechanical properties such as flexural strength and modulus of elasticity in flexure and compression strength. The results showed that the density and moisture content between wood species was found to be slightly different to each one, but water absorption and thickness swelling were determined to increase with increasing of loading rates. The mechanical properties were found to increase with nanoparticles and the maximum increasing of mechanical properties were determined to panels with nanoclays at 2%. Nanoclays at 2% can be advised to the production of laminated wood panels due to improving the mechanical properties more than 50%.

Key Words: Laminated wood, PVAc, Nanocomposites, Mechanical properties, Nano particles.

1 Introduction

Laminated wood panel, which is a one type of wood based composite are produced with layers of wood glued together. It is generally constructed with parallel gluing of wood layers in a variety of dimensions. The panels are usually used to load carrying elements such as beams, columns, arches, furniture, constructions. They have some advantages as compare with solid wood. In one of the important advantages, they can be produced as large members to passing of large blanks at construction applications. The other advantages are to have improved mechanical and physical properties and to be producing them without defects such as knots, cracking…etc. They can be produced with low costs due to using low quality wood materials as compare with solid wood (Gaborik, 2011; Tenorio et al., 2011).

In production of the laminated panels, adhesive used in the manufacture of the panels affects important properties such as physical, mechanical and structural properties of the product obtained. So, adhesive is one of basic elements in laminated panel industry. When adhesive changes the characterization properties, properties of the panels can be improved. There are many adhesive used industrially in the production of the panels (Sellers et al. 1988, Sellers 2001). The adhesives are urea (U) resin and melamine (M) resin glues, phenol resorcinol (PR) adhesives, polyurethane (PU) adhesives, and polyvinyl acetate (PVA) adhesives. PVA is one of such thermoplastic adhesive that is mostly used as an emulsion, and PVA is adhesive which is commonly used due to being a biodegradable polymer (Shukla and Kamdem, 2008).

Nanotechnology offers an important opportunity to build up alternations of adhesives with properties that cannot be provided by traditional methods. Nanoparticles incorporation into the adhesive is an economical and simple method to improve the physical and mechanical properties of adhesives. Nanoparticles has good properties such as increased aspect ratio and mechanical properties (Lopez-Suevos and Frazier 2006; Kim and Kim 2006). The addition of nano particles can provide an important increase in properties of various adhesives. The adhesives manufactured with reinforcement of nano particles have been proposed as a powerful tool for generating new multifunctional materials with improved mechanical, physical and chemical properties. Due to their small size and large surface area, nano reinforcements would be able to provide unique combination of properties, which are not possible to be reached for conventional fillers with sizes in the micrometre range. Of particular impor-

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tance, it is the requirement of achieving a good distribution of the nano particle in the polymer, in order to obtain the pursued increases in properties, without loss of other characteristics of the adhesive (i.e. process ability) because of the high tendency to particle aggregation. The potential of nano particle for adhesive formulations is promising, and their effects, most of them based on the chemical and physical interactions developed between the nanoparticle surfaces (Prolongo et al., 2010).

The aim of the study is to investigate of physical and mechanical properties of laminated oak and spruce wood manufactured with PVA adhesive with reinforcement of nano clay (NC), TiO_2 and SiO_2.

2 Material and Method

Polyvinyl acetate having a 1200 polymerization degree and a 90% hydrolysis level was purchased from Hafele Furniture and used to prepare the nanocomposites. Nano clay (NC), TiO_2 and SiO_2 was purchased from Nanocor Inc. and Mknano Inc., Canada. The nano particles were bought as ready for use directly into the resin system. Spruce (*Picea orientalis* L) and Oak (*Quercus robur*) woods conditioned at 12% MC was used to prepare the laminated panels. They were prepared with gluing the lamellas with 5 mm thickness together. The glue application was applied with a brush at an application rate of 220 g/m^2 according to related standards, and then the lamellas glued were compressed with helping of hydraulic presser under 1 MPa pressure and at room temperature. To obtained mechanical samples of the panels, they cut to 20 x 20 x 300 mm according to related standards and after pressing, samples were put into the conditioning chamber at 20ºC with relative humidity of 60% for a week. Nano particles were added to PVA matrix as 0.5% and 3% loadings. Water-nano clays solutions were firstly prepared. For this, 0.5% nano clays were added to 10 grams of water, and then the solution was firstly mixed mechanical blender at 1500 rpm for 20 min. The solution obtained was ultrasonicated at 60 kHz and 0.6 amplitude for 20 min. the same procedure was applied for other nano clays percentages. The each solution was added to PVA matrix, respectively. The temperature of the mixture of adhesive and particles was kept below 50^{o}C to provide decomposition of PVA matrix by doing all works under ethylene glycol bath inside a cooler. Percentage of nanoparticles in PVA matrix (ranging from 0.5% to 2%) was based on solid mass of PVA adhesive. Procedures and details of PVA matrix with nano particles were given in Tab. 1 and Tab.2, respectively.

Table 1. Procedure and details applied during the preparation of the adhesive

Formulation Preparation		
Step 1	Step 2	Step 3
Mechanical Mixing at 20 min, 1500 rpm	Ultrasonic Mixing at 20 min, 60 Force (W) 60 frequence (%)	Mechanical Mixing at 20 min, 1500 rpm

Table 2. Formulation of the adhesive prepared with nano particles.

Formulations	Content (%)			
	Matrix	NC	TiO_2	SiO_2
Pure PVA	100	-	-	-
PVA+0.5%NC	99.5	0.5	-	-
PVA+1%NC	99	1	-	-
PVA+2%NC	98	2	-	-
PVA+0.5% TiO_2	99.5	-	0.5	-
PVA+1% TiO_2	99	-	1	-
PVA+2% TiO_2	98	-	2	-
PVA+0.5% SiO_2	99.5	-	-	0.5
PVA+1% SiO_2	99	-	-	1
PVA+2% SiO_2	98	-	-	2

Density, swelling/water absorption for 24 hour tests were determined with Turkish Standards (TS) TS EN2472 and TS 4084. Flexural strength (FMOR), modulus of Elasticity in flexure (FMOE) and compression strength (CS) were determined according to related standards of TS 2474 and TS 2595, respectively. The mechanical test procedure was shown in Fig.1.

Figure 1. Test samples and Mechanical test procedure

Before test, laminated panels were held inside conditioning room at 20±2 °C and 65±3% relative humidity while the samples were reached to constant weight (12% MC). The conditioned samples were tested by using UTEST mechanical tester for determining the FMOE, FMOR and CS. All results were analysed in based on 95% significant level. ANOVA tests were used to determine differences between test groups, and then Duncan test was conducted.

3 Results and Discussion

Some physical and mechanical properties of laminated wood panels were investigated, as shown in Tables 1, 2 and Figures 1 and 2. Adding the nanoparticle to polymer matrix has not improved the thickness swelling and water absorption; however mechanical properties increased as significantly as compare with pure adhesive. Data obtained were analysed by performing one-way ANOVA, and then the differences among the group were determined by Duncan test. Some differences were found with statistical analysis ($P < 0.05$) for mechanical properties, thickness swelling/water absorption. According to the statistical values of density and MC, both values were found to be not significantly different. As seen Tab. 3, while particle ratio rises, density and MC values increased for small changes, but the differences were found to be not statistically significant. The highest density of the samples was found as 0.52 g/cm^3 (all samples with SiO_2) for Spruce wood and 0.80 g/cm^3 (samples with 2% TiO_2 and 2% NC) for Oak wood. After climatic condition, MC's of the samples were found to be similar for both wood species.

Table 3. Density and moisture content of laminated wood bonded by PVA prepared with different nanoparticles.

PVA Types	Laminated Spruce Wood		Laminated Oak Wood	
	Density (g/cm^3)	Moisture Content (%)	Density (g/cm^3)	Moisture Content (%)
Control	0.50 (±1.0)	10.2 (±6.7)	0.77 (±1.2)	11.2 (±1.1)
0.5% SiO_2	0.52 (±2.9)	10.3 (±6.4)	0.78 (±2.2)	12.1 (±1.2)
1% SiO_2	0.52 (±0.5)	10.2 (±6.6)	0.79 (±1.6)	11.8 (±1.0)
2% SiO_2	0.52 (±2.3)	10.7 (±4.9)	0.79 (±2.4)	11.9 (±3.0)
0.5% TiO_2	0.51 (±1.7)	10.6 (±8.8)	0.79 (±1.0)	12.5 (±1.8)
1% TiO_2	0.51 (±0.9)	10.3 (±6.0)	0.79 (±0.8)	12.7 (±1.0)
2% TiO_2	0.51 (±1.5)	10.2 (±6.6)	0.80 (±1.2)	12.3 (±0.4)
0.5% NC	0.50 (±4.9)	10.3 (±8.4)	0.78 (±0.7)	11.7 (±2.4)
1% NC	0.51 (±1.4)	10.2 (±0.7)	0.79 (±0.6)	11.9 (±2.7)
2% NC	0.51 (±2.4)	10.1 (±8.4)	0.80 (±1.4)	11.5 (±0.8)

Table 4. Thickness swelling and water absorption of the panels

PVA Types	Laminated Spruce Wood		Laminated Oak Wood	
	Thickness Swelling (%)	Water Absorption (%)	Thickness Swelling (%)	Water Absorption (%)
Control	4.7 (±0.6)	50.4 (±3.6)	2.7 (±0.3)	17 (±1.5)
0,5% SiO_2	5.7 (±0.4)	51.3 (±1.3)	5.7 (±0.67	21.2 (±6.5)
1% SiO_2	6.0 (±0.5)	51.4 (±5.2)	5.9 (±0.9)	23 (±3.2)
2% SiO_2	6.3 (±0.6)	53.1 (±2.3)	6.0 (±0.5)	24 (±1.2)
0,5% TiO_2	6.8 (±0.3)	53.0 (±4.2)	6.3 (±0.6)	18 (±2.0)
1% TiO_2	7.0 (±0.1)	55.1 (±1.0)	6.7 (±0.6)	24 (±2.0)
2% TiO_2	7.1 (±0.6)	55.8 (±1.5)	6.9 (±0.8)	26 (±2.6)
0,5% NC	6.7 (±0.5)	54.6 (±4.6)	6.2 (±1.0)	22 (±3.6)
1% NC	6.9 (±0.2)	55.4 (±1.4)	6.3 (±0.6)	24 (±6.4)
2% NC	7.4 (±0.2)	57.1 (±3.8)	6.6 (±0.6)	25 (±3.5)

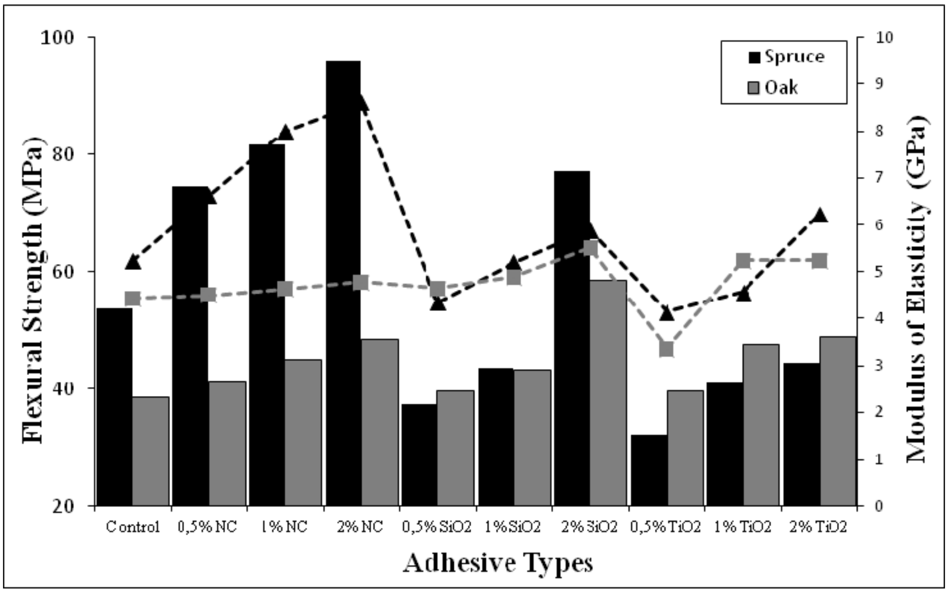

Figure 2. Flexural strength and modulus of elasticity of laminated wood bonded by PVA prepared with different nanoparticles

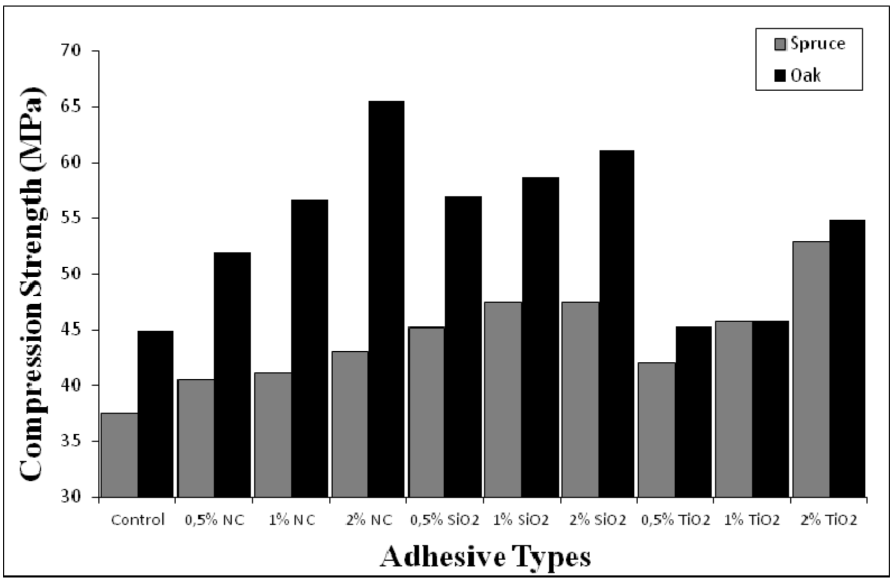

Figure 3. Compression strength of laminated wood bonded by PVA prepared with different nanoparticles

Thickness swelling and water absorption increased with waiting duration of the samples in water for both wood species. While percentage of nano particles was rising from 0.5% to 2%, both thickness swelling and water absorption were found to be increase. The highest thickness swelling and water absorption of the samples were found as 7.4% and 57.1% for 2% NC in spruce wood and as 6.9% and 26% for 2% TiO_2 in Oak wood as compare with control samples. As seen as Fig. 2 and Fig. 3, mechanical properties such as flexural strength, modulus of elasticity and compression strength of the samples were found to increase as rising the percentage of nano particles. As a result, it can be said that NC shows better performance than TiO_2 and SiO_2. The highest values of mechanical properties of the samples were found to be for 2%NC.

4 Conclusion

Laminated Spruce and Oak panels prepared with PVA filled by NC, TiO_2 and SiO_2 were produced and the effects of different nano particles on physical and mechanical properties of laminated panels were investigated. The effects of nano particles on physical and mechanical performance of laminated panels, density, MC, thickness swelling/water absorption and mechanical properties such as flexure strength, modulus of elasticity and compression strength were determined. The results showed that all of the nano particles negatively affected the thickness swelling/water absorption, and as filler content rose, the effect was found to be increase. However; mechanical properties of the all samples were determined to increase as particle content ranged from 0.5% to 2%. The highest mechanical properties for FMOE, FMOR and CS were obtained to the adhesive with 2% NC.

5 Acknowledgment

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Differences in design of stress-laminated-timber bridge decks in Europe and the USA

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Summary

Stress-laminated-timber (SLT) bridge decks are a suitable timber bridge type of simple bridges which are subjected to high concentrated loads as from heavy vehicles. The SLT bridge decks originate from North America where they were developed in the 1980's and 1990's. The American bridge code, AASHTO, has quite recently changed into a load and resistance factored design (LRFD). Several parts of the timber bridge design code in Eurocode can be recognized from older design codes from North America. This paper highlights some of the differences between the codes in terms of design load effects and design resistance of two simply supported SLT decks. Both a simplified approximate design method and orthotropic plate theory was used in design. The approximate design in AASHTO is overestimating the capacity of SLT decks compared to orthotropic plate theory. It is not possible to fulfil all the design requirements in Eurocode when using the approximate design method in Eurocode.

Key words: stress-laminated-timber decks, Eurocode, AASHTO, timber bridge design

1 Introduction

Nowadays there are many similarities between the timber bridge design codes in the US and in Europe. Especially since AASHTO recently was transformed into a LRFD format. However, there are still some fundamental differences such as: how the code is structured, traffic loads and how material values are established.

This paper can facilitate the knowledge transfer between European and North American bridge designers and researchers. There is much to be learned from comparing and sharing knowledge about how timber bridges are constructed around the globe. Gaining new perspective on bridge design methodologies ultimately supports the improvement of the national design procedures in different countries.

The main objective of this paper is to compare the AASHTO and Eurocode design process of stress-laminated-timber bridge (SLT) decks. Design of two SLT decks for vehicular traffic is exemplified for comparison. Similarities and differences between the codes for timber bridge design will be presented and briefly discussed.

2 Methodology

The main comparison in this paper will be on timber bridge deck constructed for highway traffic in the United States and in Europe. SLT decks are one of the more popular timber bridge types for vehicular traffic. This construction type will therefore be used when comparing the two design codes Eurocode and AASHTO. A more detailed comparison of the two design codes can be found in the paper by Ekholm et al. (2014) [1].

The two decks will be designed for two traffic lanes (one in each direction) and a simply supported condition. The first deck will be 12 meter long and 8 meter wide while the second deck will be 6 meter long and 8 meters wide. The loads acting on the deck will be dead-load (from the deck and a 100 mm uniformly thick layer of asphalt) and traffic load. Other loads such as: wind, snow, impact, breaking will not be covered in this paper. The decks will be designed in both the ultimate limit state (ULS) and the serviceability limit state (SLS). AASHTO calls ULS for Strength combination and SLS is called Service combination.

The deck will be designed using the current version of the AASHTO LRFD bridge design specifications [2]. The AASHTO design will be compared against the design obtained using the Eurocodes [3-6]. All amendments and corrigendum for the Eurocodes released up to 2012 were used. A service life of 80 years will be assumed for both decks. Substructures will not be included in the comparison or in the discussion of this study. All numerical

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analysis will be performed using the commercial FE package ABAQUS 6.13. Linear analysis will be performed of orthotropic decks modelled with shell elements. The simplified analyses will be calculated using Mathcad 14.

3 Design analysis methods

Both AASHTO [2] as well as Eurocode [6] states that the load effects in a stress-laminated deck should be obtained using any of these three methods:

- Orthotropic plate theory
- Equivalent grillage model
- Approximate or simplified method

Orthotropic plate theory and the approximate methods are the two most commonly used and will therefore be covered in this comparison. More information about the equivalent grillage model can be found in the literature.

Orthotropic plate theory

The behaviour of an orthotropic plate is governed by the material property orientation. SLT decks which consists of several beams (lumber or glulam) side by side has superior mechanical behaviour in the direction of the beams compared against the two directions perpendicular to the beam. The deck has stiffness in the direction perpendicular to the direction of the beams due to the prestressing. The stiffness of an SLT deck varies in reality with respect to several factors since the prestress magnitude varies with respect to time, moisture content and temperature. This is normally not considered when SLT decks are designed. Several studies have been conducted in order to correlate the transverse deck stiffness with the prestressing magnitude. [7-13]

There are only a handful of exact solutions to plate problems. Orthotropic plate problems normally approximately solved using the finite element method. Eurocode provides system properties for SLT decks that can be used when orthotropic plate analyses are conducted, see Table 1 [6].

Table 1 System properties for SLT decks made out of softwood species according to Eurocode

SLT deck material	$E_{90,mean}$ / $E_{0,mean}$	$G_{0,mean}$ / $E_{0,mean}$	$G_{90,mean}$ / $G_{0,mean}$
Sawn beams	0,015	0,060	0,080
Planed beams	0,020	0,060	0,100

There are no specific system properties specified for timber decks in the current version of AASHTO. For an approximate comparison the design values suggested in Timber Bridges: Design, Construction, Inspection and Maintenance were used [13]. The bending stiffness of the deck in the transverse direction is assumed to be 1,3% of the longitudinal stiffness. The shear modulus is assumed to be 3,0% of the longitudinal bending stiffness.

Approximate method

The approximate methods in both AASHTO and Eurocode are based on that the three-dimensional deck is replaced with a two-dimensional beam with the same length and the same depth as the deck. The width of the beam is calculated using equations that take some of the transverse load distribution into account.

In AASHTO, the longitudinal strips with the equivalent width E used to estimate moment and shear for bridges with more than one lane is calculated using equation 1. The AASHTO approximate method can only be used for bridges with a span longer than 4,57 m. The equivalent width in AASHTO is calculated per lane and not per wheel line as shown in Figure 1.

$$E = 2{,}134m + 0{,}037m\sqrt{\frac{L_{deck}}{0{,}305m}\frac{W_{deck}}{0{,}305m}} \tag{1}$$

Where L_{deck} is the length of the deck and W_{deck} is the width of the deck.

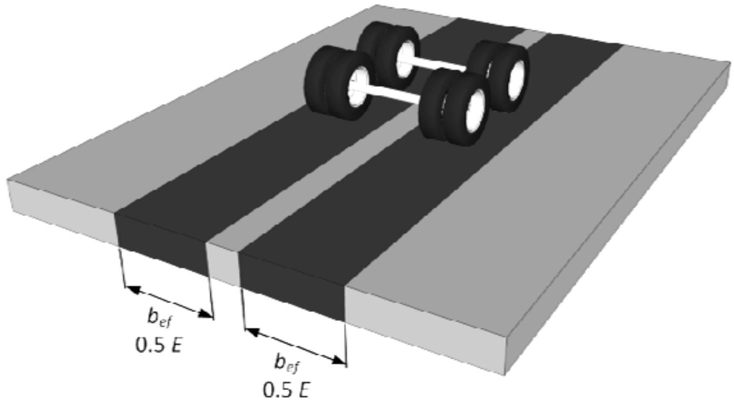

Figure 1: Width of equivalent strip (0,5 x E in AASHTO and b_{ef} in Eurocode) used by the approximate method.

In Eurocode, a stress-laminated-timber bridge deck may be replaced by several beams acting in the direction of the traffic. The load from a patch load is distributed in the transverse direction down to a reference plane in the middle of the deck using equation 2. The distribution in the longitudinal direction uses the same equation but with the angle set to 45º.

$$b_{w.middle} = b_w + 2t_{surface} + d_{deck} \tan(15°) \tag{2}$$

Where b_w is the width of the deck contact area, $t_{surface}$ is the thickness of the surfacing and d_{deck} is the depth of the deck, see Figure 2 .

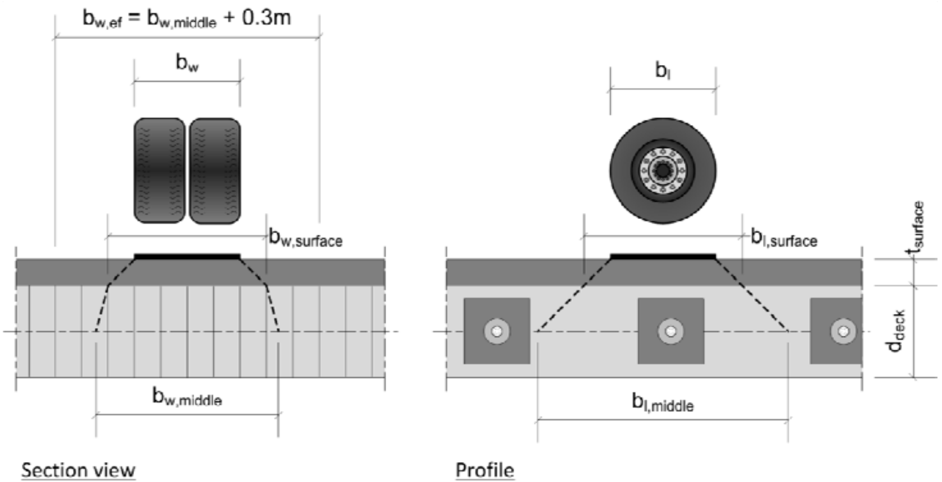

Figure 2: Distribution of patch load through the thickness of the deck according to Eurocode.

The final width under each wheel line, b_{ef}, is obtained by adding 0,3 m for SLT decks to the loaded width in the middle of the deck, $b_{w.middle}$, according to equation 3. In Eurocode, each traffic lane is replaced by two equivalent strips as shown in Figure 1.

$$b_{ef} = b_{w.middle} + 0,3m \tag{3}$$

Deck loads

There is a significant difference between AASHTO and Eurocode in terms of weight of timber. AASHTO states that a weight of 7,9 kN/m^3 should be used for softwood species. For similar softwood glulam the weight in Eurocode (EN 1995-1-1) ranges from 3,9-4,1 kN/m^3 for appropriate glulam grades. However, the main reason for the difference in weight is that AASHTO assumes the weight of wet pressure-treated timber (creosote or penta) while Eurocode assumes dry untreated timber. Bituminous wearing surface has a weight of 27,5 kN/m^3 in AASHTO. Equivalent asphaltic concrete has a density of 24-25 kN/m^3 in Eurocode.

The design vehicular load in AASHTO should be taken as the greater of the two load effects from either a design truck or a design tandem in combination with a design lane load. This particular load case is called HL-93, see Figure 3. The tire contact area over which the loads should be distributed is a rectangle 0,25 m long and 0,51 m wide (10 x 20 in). In the orthotropic analysis the tire load was distributed over an area which included the distribution through the surfacing. The area was 0,45 m long and 0,71 m wide. The design tandem with the design lane load was the governing case in the deck comparison conducted in this paper.

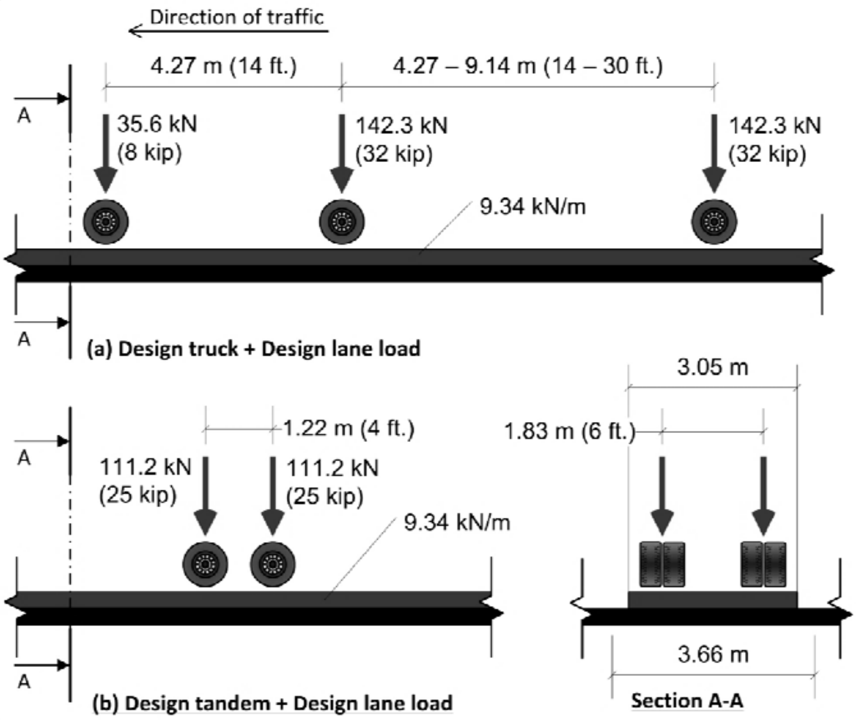

Figure 3: HL93 AASHTO LRFD [2] (a) design truck and (b) design tandem, both with design lane load.

In Eurocode 1 [4] there are basically four different load models which should be considered. These load models do not represent actual load cases but were developed to represent the load effects from traffic around Europe at year 2000. Load model 1 (LM1) is the most general load case. It contains both concentrated forces and distributed loads to represent most load effects from trucks and cars. Load model 2 (LM2) contains just a single vehicle axle. Load model 3 (LM3) is a set of axle and load combinations which can be used for special load cases, such as industrial transports. Load model 4 is a heavy pedestrian loading representing a crowd. Load model 1 and 2 are the two load cases normally used in design for this type of structure.

Each lane in LM1 is loaded by two vehicle axles with the magnitude $\alpha_{Qi}\ Q_{ik}$ and a distributed load of $\alpha_q\ q_{ik}$, as shown in Figure 4. Both α_{Qi} and α_q are modification factors which the values are specified in the national annex. The modification factors used in the design are shown in Table 2. The values of the modification factors are taken from the Swedish national annex. Only one tandem system has to be applied in each lane. Each load axle has two identical wheels with the foot print of a 0,4 m square. In the orthotropic analyses the vehicle load was distributed over an area which is equal to $b_{w.middle}$ and $b_{l.middle}$ in Figure 2. The distributed load should only be applied on the bridge where it has an unfavorable effect for the structure.

Table 2 Characteristic load values for Load model 1 in Eurocode (values for modification factors are taken from the Swedish national annex)

	Axle load	**Distributed load**	**Modification factors**	
Lane	Q_{ik} **(kN)**	q_{ik} **(kN/m²)**	α_{Qi}	α_{qi}
Lane 1	300	9,0	0.9	0.7
Lane 2	200	2,5	0.9	1.0
Lane 3	100	2,5	0	1.0
Lane 4 or more	0	2,5	0	1.0
Remaining surface, q_{rk}	0	2,5	-	1.0

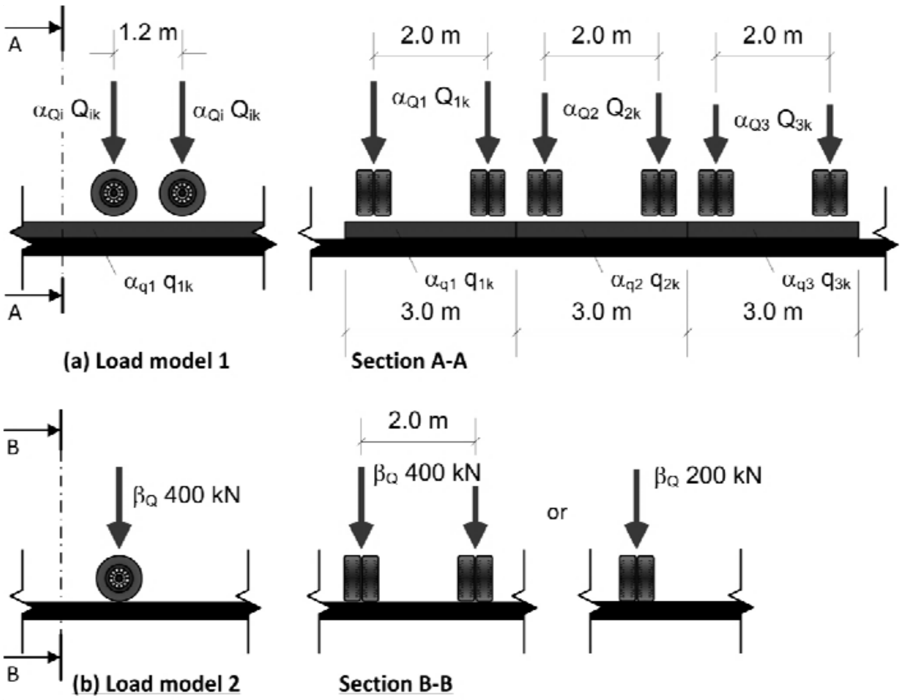

Figure 4: Eurocode load models [4] (a) Load model 1 and (b) Load model 2.

Both design codes have different load factor combinations of the various loads simulating several different kinds of events that might occur and to obtain a satisfactory safety envelope. There are several load combinations available to check the ultimate capacity of the bridge (Strength in AASTHO and ULS in Eurocode). There are also load combinations used to verify the condition of the bridge in service (Service in AASHTO and SLS in Eurocode). Table 3 shows the load combination factors used in the deck comparison. In Eurocode there are different load factors for the dominating variable load and the remaining variable loads. In the deck comparison it was obvious that the load effect from the axle load was greater than the load effect from the distributed load. Thus was a greater load combination values used for the axle load compared to the distributed load. A more thorough explanation of the different load combinations is given in [2].

Table 3 Load combination factors for both ultimate capacity (Strength and ULS) and service condition (Service and SLS) according to AASHTO and Eurocode. Values within brackets indicates factors used when the load has a favourable effect

		AASHTO			Eurocode	
Load	**Strength I**	**Strength V**	**Service I**	**ULS 6.10a**	**ULS 6.10b**	**SLS Freq. LC**
Self-weight	1,25 (0,9)	1,25 (0,9)	1,0 (1,0)	1,35 (1,0)	1,15 (1,0)	1,0 (1,0)
Wearing surface	1,5 (0,65)	1,5 (0,65)	1,0 (1,0)	1,35 (1,0)	1,15 (1,0)	1,0 (1,0)
Traffic, Axle load	1,75 (0)	1,35 (0)	1,0 (0)	1,01 (0)	1,35 (0)	0,75 (0)
Traffic, distributed load	1,75 (0)	1,35 (0)	1,0 (0)	0,54 (0)	0,54 (0)	0

Deck design

Glulam beams used for the deck comparison were chosen in order so that their adjusted design values for bending strength and stiffness were somewhat similar. For the Eurocode deck glulam of class GL 28 c was used. For the AASHTO deck a glulam made out of Douglas Fir called 24F-V4 was used.

4 Results and discussion

Design resistance

Material design values in AASHTO have to be adjusted in order to account for: volume effects, wet service (moisture content), time effect (load duration), code format (material values are still established for allowable stress design which has a different duration of load reference) and type of resistance (bending, shear, compression). The reference design values I AASHTO are low and then are the values increased by multiplying with the adjustment factors.

Eurocode use a slightly different format than AASHTO where the characteristic value of the material is reduced to a design value. The characteristic value is multiplied by the factor k_{mod} which takes the load duration and service class into account. The characteristic value is then divided by a partial factor factor γ_M that takes the uncertainty in the material property into account. The partial factor for glulam in ULS is 1,25 (can also be specified in the national annex) and 1,0 for SLS. Eurocode also uses a system factor which gives the possibility to increase the longitudinal bending and shear capacity for repetitive members. There is also a size effect which has to be considered to glulam in bending and in tension.

Table 4 shows the calculated deck resistance for the two different decks according to AASHTO [2] and Eurocode [5-6]. The thickness of the decks has been calculated to be as small as possible but still large enough that the resistance is larger than the load effects. The deck thickness for the AASHTO is a multiple of 1,5 inches (38,1 mm) while the Eurocode deck is a multiple of 45 mm. Neither AASHTO nor Eurocode provides any information for resistance against transversal bending moment even though this is generally considered one of the unique failure modes for SLT decks. AASHTO does not give any requirements for the longitudinal shear force. It should also be noted that AASHTO states that the shear capacity for SLT decks does not need to be considered. The transverse shear force capacity is calculated using equation 4 which is taken from [6].

$$V_{yz.Rd} = \mu_d \, \sigma_{p.min} \, h \tag{4}$$

Where μ_d is the coefficient of friction used for planed timber against planed timber perpendicular to grain at a specific moisture content, $\sigma_{p.min}$ is the minimum long-term residual compressive stress due to prestressing and h is the thickness of the deck. A long-term moisture content of 14% was assumed in the deck for the deck comparison. The long-term residual prestressing stress should be taken as 0.35 MPa in areas subjected to concentrate loads. This is a very low value and will most likely be much higher in reality.

The deflection limit for the AASHTO deck is span divided by 425 while Eurocode has a recommended limit of span divided by 400.

Table 4 Calculated deck resistance for both decks according to both AASHTO and Eurocode.

	Deck 1, 12m span		**Deck 2, 6m span**	
Design resistance	**AASHTO**	**Eurocode**	**AASHTO**	**Eurocode**
Deck thickness, d (mm)	*533*	*630*	*343*	*405*
Longitudinal bending moment $M_{y,Rd}$ (kNm/m)	857,1	1592,5	389,1	687,9
Transverse bending moment $M_{x,Rd}$ (kNm/m)	(13,1)	(23,6)	(5,4)	(9,6)
Longitudinal shear force $V_{xz,Rd}$ (kN/m)	818,6	851,0	526,3	547,0
Transverse shear force $V_{yz,Rd}$ (kN/m)	NA	66,2	NA	42,5
Support compression force $F_{z,Rd}$ (kN/m)	1675,2	648,0	1675,2	648,0
Deflection limit u_{limit} (mm)	28,2	30,0	14,1	15,0

The design resistance for the transverse bending moment is not included in either AASHTO or Eurocode. However, the resistance can easily be calculated by using equation 5. AASHTO uses minimum long-term residual pre-stress compressive stress of 0,28 MPa (40 psi).

$$M_{x.Rd} = \frac{\sigma_{p.min} * h^2}{6} \tag{5}$$

Where $\sigma_{p.min}$ is the minimum long-term residual compressive stress due to prestressing and h is the thickness of the deck.

Load effect

The load effects from dead loads and vehicle live loads have been calculated according to AASHTO and Eurocode. Values have been established for both decks and according to both the approximate method and orthotropic plate theory. The loads on the deck were of the magnitude as specified in the previous chapter. The loads were taken as the greatest of the load combinations (Strength I or V for AASHTO and ULS 6.10a or 6.10b for Eurocode).

The approximate method in AASHTO generally underestimates the load compared to the orthotropic plate theory. This was the case for both deck 1 and deck 2 as shown in Table 5. However, the difference between the design methods was in the range 10-12% for the governing deflection. The longitudinal bending moment in AASHTO merely differed 1-5% between the two methods.

The difference between the approximate method and the orthotropic plate theory was much greater in using Eurocode. This time the approximate method was clearly overestimating the load on the deck compared to the orthotropic plate theory, as shown in Table 5. This was at least valid for the longitudinal bending moment and the deflection. The difference between the two methods was in the range of 28-33% for the bending moment and 18-34% for the deflection. This indicates that the calculated width used in the approximate method is too small. The values for the longitudinal shear force and the support compression was much greater in the FE analysis compared to the approximate method. But this is most likely more due to numerical singularities in the FE analysis since the load is very close to the support and the support is prevented from translation in a single row of nodes.

Table 5 Calculated design load effects from dead and live loads. Load effects are calculated for both decks according to both AASHTO and Eurocode and using both the approximate method (AM) and orthotropic plate theory (OPT).

	Deck 1, 12m span				**Deck 2, 6m span**			
	AASHTO		**Eurocode**		**AASHTO**		**Eurocode**	
Design load effect	**AM**	**OPT**	**AM**	**OPT**	**AM**	**OPT**	**AM**	**OPT**
Deck thickness, d (mm)	*533*		*630*		*343*		*405*	
Longitudinal bending moment $M_{y.Ed}$ (kNm/m)	580,3	609,2	1091,2	817,8	219,3	220,5	476,4	371,4
Transverse bending moment $M_{x,Ed}$ (kNm/m)	NA	8,5	NA	10,8	NA	5,7	NA	9,3
Longitudinal shear force $V_{xz,Ed}$ (kN/m)	209,1	250,5	198,9	510,6	150,7	213,9	159,8	403,9
Transverse shear force $V_{yz,Ed}$ (kN/m)	NA	31,0	NA	52,3	NA	25,1	NA	41,9
Support compression force $F_{z,Ed}$ (kN/m)	216,7	286,5	214,2	352,3	158,2	217,0	185,5	323,7
Maximum deflection u_{max}(mm)	24,1	27,4	25,8	19,3	10,7	11,9	12,3	10,4

Material utilization

Table 6 shows the material utilization in terms of design load effects (shown in Table 5) divided by the design resistance of the deck (shown in Table 4). Both deck 1 and deck 2 designed according to Eurocode has the highest material utilization on the transverse shear force. However, the design resistance for transverse shear force is based on a long term residual prestress of 0,35 MPa which is fairly low. A more realistic value for decks (at least in Sweden) would be around 0,5 MPa which would reduce the utilization for deck 2 from 99% down to 69%.

Table 6 Utilization of the material in terms of design load effect divided by design resistance. Material utilization is calculated for both decks according to both AASHTO and Eurocode and using both the approximate method (AM) and orthotropic plate theory (OPT).

5	**Deck 1, 12m span**				**Deck 2, 6m span**			
	AASHTO		**Eurocode**		**AASHTO**		**Eurocode**	
Utilization	**AM**	**OPT**	**AM**	**OPT**	**AM.**	**OPT**	**AM**	**OPT**
Deck thickness, d (mm)	*533*		*630*		*343*		*405*	
Longitudinal bending moment $M_{y.Ed}$ (kNm/m)	68%	71%	69%	51%	56%	57%	69%	54%
Transverse bending moment $M_{x,Ed}$ (kNm/m)	NA	65%	NA	46%	NA	106%	NA	97%
Longitudinal shear force $V_{xz,Ed}$ (kN/m)	26%	31%	23%	60%	29%	41%	29%	74%
Transverse shear force $V_{yz,Ed}$ (kN/m)	NA	NA	NA	79%	NA	NA	NA	99%
Support compression force $F_{z,Ed}$ (kN/m)	13%	17%	33%	54%	9%	13%	29%	50%
Maximum deflection u_{max}(mm)	85%	97%	86%	64%	76%	84%	82%	69%

6 Conclusion and future work

The conclusions that can be draw from this comparison is that the load effects on SLT decks are greater when designing with Eurocode compared to when designing with AASHTO. For the design resistance the greatest discrepancies between Eurocode and AASHTO are found in the longitudinal bending moment and the support compression. The longitudinal bending moment using AASHTO is only 54-57% of the value obtained using Eurocode.

The transverse bending moment is not included in whether of the two design code today. The transverse load effects are certainly not negligible as shown in the calculations in this paper. Careful consideration of additional transverse load design criteria by each of the bridge code agencies is clearly warranted. Eurocode has a requirement to verify the design resistance for transverse shear force. However, there are no instructions on how this load effect should be calculated which makes the approximate method unusable. Designers are forced to use orthotropic plate analysis in order to fulfil the requirements in Eurocode as it is written today. A simplified equation for calculating the transverse shear force could also be incorporated in Eurocode without too much effort. The transverse shear force is only dependent on a handful of variables which are: load magnitude, size of tire footprint, span length and deck width. Future efforts will be focused on establishing a simplified equation for transverse shear forces which takes these variables into account.

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Timber-concrete composite bridges – sustainability assessment

João Rodrigues [1], Alfredo Dias, Paulo Providência

Content

A renewed interest on the use of Timber-Concrete Composite (TCC) structures in bridge decks is being noticed over the last few years. This seems to be related to their cost competitiveness and environmental friendliness.

Accordingly, this paper focuses on the sustainability assessment of TCC bridge decks, using a type of deck which is one of the most commonly found around the world. A life-cycle methodology, comprising environmental and economic assessment, is comparatively applied to the deck of an existing reinforced concrete (RC) case study bridge, representative of this structural solution, and to a TCC deck potential alternative.

The results obtained in this assessment show that TCC solutions cause less environmental impact and are also economically competitive.

1 Introduction

Nowadays, the sustainability of all human activities is a matter of major concern. This is particularly true within the construction industry, which is the economic activity sector with higher environmental impact. Indeed, the contribution of the construction activities to resources consumption and waste production is higher than that of any other industrial activity [1]. For instance, in the EU about half of the resources extracted from the earth is used in the construction industry and more than a quarter of the solid waste is produced in construction activities [2].

Accordingly, the construction sector must abide by the principles of sustainability. This urge for a sustainable construction is a possible reason for the development of new construction and structural solutions, such as Timber-Concrete Composite (TCC) structures for bridge decks. TCC bridge decks are commonly formed by three resisting components: a reinforced concrete slab on top of longitudinally positioned timber beams, with the slab and beams joined together by a connecting system. Rodrigues et al. [3] studied a representative sample of TCC bridges and concluded that, even though they first appeared in the 1930s, in the USA, this type of structure became much more common in the last two decades, with more than 85% of the bridges identified in that study built since 1990, and more than 50% from 2000 to 2013.

In an attempt to examine whether sustainability requirements represent a plausible explanation for the increase of TCC bridge deck construction observed in the last few years, a sustainability assessment of a standard short span simply supported girder-bridge was performed. This sustainability assessment is based on a life-cycle analysis, com-prising environmental and economic dimensions. Within this assessment, a typical reinforced concrete (RC) deck was compared with a common type of TCC bridge deck, frequently known as “T-beam deck”: it has a reinforced concrete slab, which forms the flange of a T-section, casted on equally spaced glulam timber girders which make the web of the T – they can be found in spans of up to 30 m and they are particularly suitable to overpasses crossing main roadways or railways in urban areas. A RC bridge was selected for this comparison because this structural material is the worldwide leading contender for short span bridges [4].

2 Life-cycle methodology

The environmental performance is measured through the Life-Cycle Assessment (LCA), guided by ISO standards 14040 [5] and 14044 [6]. According to the ISO methodology, LCA comprises four phases: (i) goal and scope definition, (ii) life-cycle inventory analysis (LCI), (iii) life-cycle impact assessment (LCIA) and (iv) interpretation.

LCI involves data collection and calculation procedures to quantify relevant inputs and outputs (i.e. which cause environmental damages) at the system boundary. Data can be collected from different sources. For this study, it was adopted the Ecoinvent v2.0 database [7], which was developed by the Swiss Centre for Life-Cycle Inventories and is commonly referred to as the most comprehensive and complete for European countries [8].

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LCIA evaluates the magnitude of the potential environmental impacts using the LCI data. This evaluation involves mandatory elements (classification and characterization) and also optional elements, e.g. normalization, grouping, weighting and data quality analysis [5; 6]. Firstly, LCI data must be associated to specific environmental impact categories. This study followed the CML 2001 Method, developed in the Institute of Environmental Sciences of University of Leiden, Netherlands [9], from which, the following impact categories were considered: Abiotic Depletion (AD), Acidification (AC), Eutrophication (EU), Global Warming (GW), Ozone-layer Depletion (OD) and Photochemical Oxidation (PO). The CML methodology includes four more impact categories: human toxicity, fresh water aquatic ecotoxicity, marine aquatic ecotoxicity and terrestrial ecotoxicity, which were omitted from this study because of unacceptably high uncertainty in the toxicity data, particularly for glulam [10].

In the interpretation phase of LCA the findings from the LCI and LCIA are considered together, in accordance with the goal and scope of the study [5; 6]. This requires a previous normalization of the results, where the different impacts are weighed. Subsequently, the results are compared using the normalized data also provided by the CML 2001 Method.

For this LCA study the software SimaPro® [11] was used, which includes the CML 2001 Method and is claimed to be the most tested and powerful LCA tool by several authors [8; 12; 13].

The economic performance is best considered using a Life-Cycle Cost Analysis (LCCA). This analysis can be based on the ISO standard 15686-5 [14] and it often follows the classification scheme developed by Ehlen and Marshall [15] which includes: (i) agency costs – "costs incurred by the project's owner over the study period"; (ii) user costs – "costs occurred to the direct users of the project, such as congestion and delays for traffic"; and (iii) thirdparty costs – "all costs incurred by entities who are neither the owners themselves nor direct users of the project". This study followed an adaptation of the Ehlen and Marshal methodology to bridge decks [16], in which only the agency costs were credited to the LCCA.

Agency costs are subdivided in three items: construction costs, operation costs and disposal costs. Construction costs include all the costs related to the construction of the bridge, such as material and equipment acquisition or manpower. Operation costs include all the maintenance and rehabilitation costs during the service life of the bridge. Disposal costs refer to demolition of the bridge and removal costs. Future costs like operation and disposal costs involve some uncertainty. In this study, a deterministic approach, based on scenarios which take into account current practice, was chosen. These future costs have to consider the fluctuation of money value over that period. ISO standard 15686-5 [14] uses the well-known expression for conversion of future costs to a fixed point in time,

$$PV = \sum_{t=0}^{t_L-1} \frac{C_t}{(1+d)^t} \qquad (1)$$

where PV is the present value of the total costs, C_t is the sum of all costs incurred in year t, t_L is the number of years in the study period, and d is the discount rate. The selection of a suitable discount rate is crucial in a LCCA, but there is no consensus on this issue. However, some authors [17; 18] and even ISO 15686-5 [14] claim that a real discount rate is more accurate than a nominal rate. ISO 15686-5 proposes the use of a discount rate between 0 % and 4 % [14]. The present study considered d = 4 %.

3 Characterization of the case study

The case study encloses a typical road bridge over a single-track railway, located in Figueira da Foz, near the Portuguese Atlantic coast. It is a two-lane 9.20 m wide and 14.80 m long single span bridge simply supported at the abutments. The existing deck is made of six precast prestressed I-beams connected to a reinforced concrete slab cast on-site (Figure 1), whereas the proposed deck is composed of eight glulam beams with rectangular cross-section 0.28 m x 0.96 m, fastened to a reinforced concrete slab also cast on-site (Figure 2).

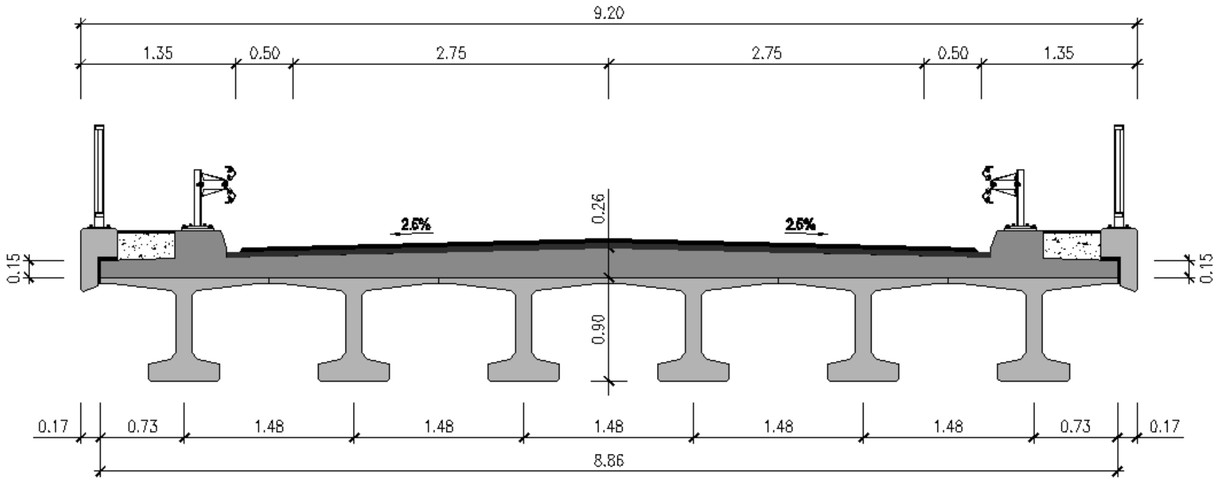


Figure 1: Cross-section of the existing deck (dimensions in meters)

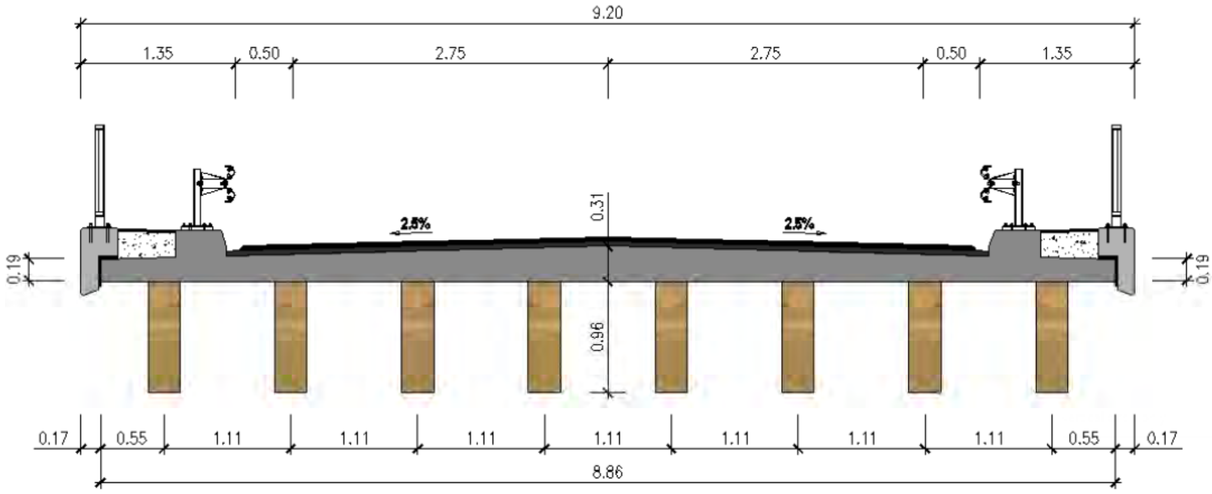


Figure 2: Cross-section of the proposed deck (dimensions in meters)

The design procedures of the decks comply with the requirements of EU standards – Eurocode 2 [19] and Eurocode 5 [20; 21]. The following nominal loads were used: (i) dead load – 25 kN/m^3 for reinforced concrete, 18 kN/m^3 for asphalt, 3.8 kN/m^3 for glulam and (ii) live loads according to section 4 of Eurocode 1 Part 2 [22].

The main objective of the present study was to compare the life-cycle of the RC existing deck with the TCC proposed deck. The functional unit for this comparative analysis is a bridge deck designed for a service life of 50 years, according to Eurocode 0 [23].

The system boundary includes environmental and economic impacts over four main stages: (i) the raw material production stage; (ii) the construction stage; (iii) the operation stage and (iv) the end-of-life stage. Further details about the system boundary are given below.

The guidelines of the impacts of the raw material production stage, construction stage and operation stage are presented in Table 1, Table 2 and Table 3, respectively. To perform the life-cycle analysis of the case study, a set of scenarios and assumptions has to be defined in order to establish the maintenance plan (Table 6). Activities with the same schedule should, when possible, be implemented simultaneously to reduce the number of interventions. These plans contain the required specifications to maintain the deck under proper conditions during its operation period; however, serious accidents that could cause severe damage or failure of the bridge are obviously excluded.

It was assumed that at the end-of-life of the bridge (year 50) the structure will be demolished. The impacts are considered according the guidelines presented in Table 4. It was assumed that concrete and other non-structural materials are placed in a landfill, steel is recycled and timber elements are reused in other applications, such as urban furniture (eventually, the glulam beams can be cut into several pieces).

Table 1: Details of the system boundary considered in the raw material production stage

Raw material production stage		
LCA	Raw materials acquisition and transformation	Only structural materials (concrete, steel and timber) were considered, according to Ecoinvent v2.0 database.
NOTE: This stage did not include the economic impact: the costs of acquisition and transformation of raw materials are used to quantify the economic impact of the construction stage.		

Table 2: Details of the system boundary considered in the construction stage

Construction stage		
LCA	Material transportation	Only structural materials transportation was considered: - The concrete was produced at a local mixing plant and transported 29 km by truck; - The precast concrete beams were transported 70 km by truck from the production plant to the bridge site; - The timber beams were transported 65 km to the bridge site by truck; - The reinforcement steel was transported 43 km to the bridge site by truck. NOTE: These distances, as well as the ones presented in Table 3 and Table 4, correspond to the reality verified on site.
LCCA	Resources needed	The amount of resources needed for the construction of the decks, and their unit costs, are shown in Table 5.

Table 3: Details of the system boundary considered in the operation stage

Operation stage							
		Activity A - periodic inspection.	Activity B - main inspection; - other cleanings.	Activity C - main inspection; - other cleanings; - surfacing.	Activity D - main inspection; - other cleanings; - protection of timber elements.	Activity E - main inspection; - other cleanings; - surfacing; - protection of timber elements.	Activity F - main inspection; - repairing of concrete structure and other elements.
LCA	Resources transportation	2 km by van	2 km by van	2 km by van and 35 km by truck	62 km by van	62 km by van and 35 km by truck	172 km by van
LCCA	Maintenance activities	The maintenance activities considered in the operation stage, and their unit costs, are shown in Table 6.					

Table 4: Details of the system boundary considered at the end-of-life stage

End-of-life stage		
LCA	Disposal treatment	Only structural materials (concrete, steel and timber) were considered, according to Ecoinvent v2.0 database.
	Waste construction transportation	- The concrete is placed at landfill, being transported 61 km by truck; - The timber beams are re-used, being transported 500 km by truck; - The reinforcement steel is recycled, being transported 154 km by truck.
LCCA	Disposal action	The disposal actions of the waste from deck demolition and their costs are shown in Table 7. NOTE: The important difference between the decks in removal cost is related to (i) the amount of material removed (approximately 78 m^3 for existing deck and 88 m^3 for proposed deck) and (ii) the transportation cost (13 €/m^3 for concrete, steel and other construction waste and 52 €/m^3 for timber). NOTE: In the case of steel recycling or timber re-use, the impact values are negative, representing benefits.

Table 5: Resources needed and their cost

Resource		Un	Unit cost (€/un)	Quantity Existing deck	Quantity Proposed deck
Slab	Concrete grade C30/37	m^3	105.00	(h=0,21 m) 27.84	(h=0,25 m) 33.96
	Reinforcement steel grade A500	kg	0.95	3,690.24	2,826.09
	Formwork (side surfaces)	m^2	30.00	8.23	10.26
	Formwork (bottom surface)	m^2	35.00	-	101.48
Beams	Precast concrete beams	un	5,500.00	6.00	-
	Glulam beams grade GL 28h	m^3	785.00	-	32.97
	Connection system	un	16.56	(included in beams)	82.00
	Waterproofing membrane	m^2	12.00	-	6.14
	Epoxi glue	l	20.21	-	42.00
Side-walks	Concrete grade C30/37	m^3	105.00	3.13	3.13
	Reinforcement steel grade A500	kg	0.95	481.18	481.18
	Formwork	m^2	35.00	14.87	14.87
	Light-weight concrete	m^3	44.80	44.80	44.80
	Cement coating	m^2	7.00	15.33	15.33
	PVC pipe Φ 110 mm	m	2.40	91.98	91.98
Others	Pavement – asphalt layer 4+4 cm	m^2	14.00	102.13	102.13
	Elastomeric bearing type C Φ 200 mm	un	250.00	12.00	-
	Elastomeric bearing type F 270x280x13 mm	un	75.00	-	16.00
	Bearings – galvanized steel plates	un	80.66	-	16.00
	Expansion joints	m	600.00	17.72	17.72
	Deflector rails	m	58.50	30.66	30.66
	Parapet rails	m	110.00	30.66	30.66
	Edge beams	m	85.00	30.66	30.66
	PEAD pipe Φ 110 mm	m	3.51	13.96	13.96
	Steel drainage system	un	39.50	4.00	4.00

Table 6: Scenario-based maintenance plan

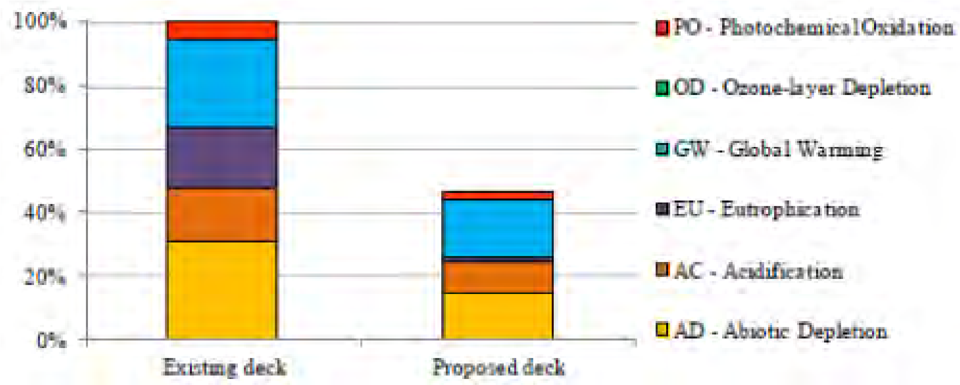
Maintenance activity		Interval (years)	Unit cost (€/Un)
Periodic inspection	Existing deck	2	96.00
	Proposed deck	1,25	96.00
Main inspection		5	320.00
Other cleanings (cleaning of expansion joints and bearings)		5	80.00
Surfacing (asphalt)		10	1,181.00
Protection of timber elements	Existing deck	-	-
	Proposed deck	15	1,079.00
Repairing of the bottom surface of concrete slab	Existing deck	-	-
	Proposed deck	25	4,059.00
Replacing of expansion joints		25	12,758.00
Replacing of elastomeric bearings	Existing deck	25	3,600.00
	Proposed deck	25	1,600.00
Repairing of concrete edge beams		25	1,303.00
Replacing deflector rails		25	1,901.00
Re-painting parapet rails		25	920.00

Table 7: Scenario-based end-of-life plan

Action	Cost (€) Existing deck	Cost (€) Proposed deck
Demolition	10,893.00	10,893.00
Removal	1,015.00	2,421.00
Landfill treatment	2,935.00	2,456.00
Recycling or reuse treatment	-5,849.00	-10,351.00

4 Results

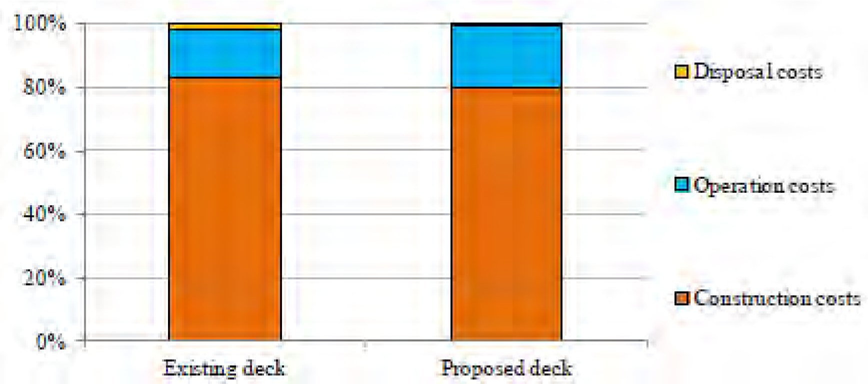
The overall environmental performance or, more specifically, the normalized results for the environmental impact categories considered, is shown in Figure 3.



The most significant result in this chart is that the environmental impact of the existing RC deck is more than twice that of the proposed TCC deck. Bearing in mind that the resources needed for the existing and proposed decks are similar except for the girders, it can be concluded that the environmental potential of timber is the main reason be-hind the obtained differences – timber is a natural, renewable resource and a carbon store, whose production re-quires small amounts of energy [24; 25].

The analysis of Figure 3 also shows that two environmental impact categories turn out to be much more relevant than the others: decreasing availability of natural resources (AD) and global warming (GW). There are also some significant impacts related to the conversion of air pollution into acid substances (AC) and to the excessively high environmental levels of nutrients (EU), whilst the impacts associated to the formation of reactive chemical com-pounds such as ozone by the action of ultraviolet (PO) and to the ozone-layer depletion (OD) prove to be insignificant. These results are in line with those obtained in similar studies on bridges, e.g. Hammervold et al. [10] and Gervásio [16].

Figure 4 depicts the economic performance for the case study, considering d=4%, and the individual costs of each bridge life-cycle stage. This figure reveals that, in practical terms, the cost of the proposed TCC deck solution (77,973.00 €) is equal to that of the existing RC deck solution (77,870.00 €). Figure 4 also shows that operation costs represent about 20% of the deck cost, while construction costs are about 80% of the deck cost.



5 Conclusions

The usage of TCC bridge decks is particularly competitive for single or multi-span bridge with individual short spans, which justifies the choice of the bridge used in the sustainability comparative assessment performed in the present study. This assessment compared the deck of a RC bridge (the most common typology used in short span bridges) with an alternative TCC deck solution.

The environmental life-cycle analysis showed that the proposed TCC deck has a much better performance than the existing RC one. This fact is closely related with the environmental advantage of using timber – a sustainable building material. This analysis also indicated that the environmental impact categories related to abiotic depletion (AD), and global warming (GW) are the most important, as far as the studied bridge is concerned.

From the economic life-cycle analysis, it can be concluded that (i) the proposed TCC deck is a rather competitive solution (the cost of the proposed TCC deck is similar to the cost of the existing RC deck) and (ii) the construction stage is responsible for the majority of the costs.

From this study, it is concluded that, for short span bridges, TCC decks are a competitive and sustainable solution when compared with RC decks. However, it is important to note that these results cannot be generalized for all types of bridges. Moreover, a bridge deck is a complex system that consists of numerous components and various scenarios through its long life span. Hence, some level of uncertainty is always inevitably involved in any sustainability assessment of such structures.

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Timber-Concrete Composite Bridges with Reinforced Timber Beams and Stud Connectors

Markus Jahreis[1], Wolfram Hädicke[2], Martin Kästner[3], Karl Rautenstrauch[4]

Summary

Timber-Concrete Composite (TCC) is a convenient technique to construct T-beams for infrastructur constructions with economical and ecological benefit. This paper presents the results of several ongoing research projects about TCC at the Department of Timber and Masonry Engineering of the Bauhaus University Weimar and also some realized pilot projects of TCC-road bridges. Presented are different possibilities for the design of the composite joint between the timber beam and concrete slab by using stud connectors. In addition, several reinforcements of the wooden girder for the use at hybrid road bridges are presented.

Keywords: TCC, Reinforced Timber Beam, FRP, Polymer Concrete

1 Introduction

The importance of environmental awareness in structural design increased during the last decade. The rate of timber as the most sustainable material for construction has risen considerably. However, infrastructural buildings are mostly made of energy intensive materials like concrete and steel and it seems that there is only a low demand for structural elements made of timber. The reasons for this may be on the one hand the strong stakeholders of steel and concrete industries and on the other hand the variance of the properties of timber and the lower stiffness of the material. Therefore, it is necessary to spark interest of the authorities for road construction for infrastructural buildings in timber constructions also with innovative solutions in construction and design. Furthermore, the acceptance of the natural material can be raised by reducing the variance of properties with reinforcements. With an expedient use of common and new materials in combination for a hybrid structure, it is possible to design a construction with high efficiency and good life cycle assessment. Two main issues are persecuted at the Department of Timber and Masonry Engineering with the object of TCC bridges: the optimization of the connecting joint between the concrete slab and wooden girder and the development of reinforcements of the timber beam.

Figure 1: TCC Bridge „Birkbergbrücke“ Germany

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2 Connection and Joint

For the design of hybrid structural elements the connecting joints between the materials or elements are important. One factor for successful design is the load bearing capacity, the serviceability and also the safety can be influenced by the stiffness. There are two main options for transmitting the forces. The first one is the form closure, as it has been used in traditional timber works with a kind of step joint and can be used with filled groovings. The other way is the adhesive bonding for stiff connection. This is already been used with glued in rods or glued in metal sheets. Ongoing research investigates the direct bonding of concrete slabs or steel connectors to the wooden girder.

Based on the research results of the Department of Timber and Masonry Engineering [1], [2], [3], [4], two road bridges were realized as TCC construction with stud connectors used as connecting elements. This is an accepted method used in steel-concrete composite constructions. The connecting elements, which are embedded in the concrete, are welded to a 30 mm strong plate of steel, witch are fitted into notches in the timber. The principal of form closure is used to transfer shear forces directly per contact between steel and timber. The ductility of the connection can be influenced by the dimensioning and allocation of the stud connectors.

The connecting system was tested in short time tests as well as in long term and fatigue tests. More detailed information are published by Mueller in [5], [6], [7]. The experimental set up was conducted according to DIN EN 26891:1991 [8] as symmetric push-out test. The failure mechanism is double-staged. Firstly, there is a local deformation of the timber in consequence of the compression load at the bearing area in contact to the steel connector. This provokes a rising slope but also a hardening of the compression zone. Finally, the collapse is caused by shear failure of the timber in front of the step joint. These failings can be observed almost at all kinds of tests with the pure connecting system made of stud-connector in a carving. In later experiments the joint was modified with a filling of polymer concrete. To improve the form closure and prevent the deformation of the timber in the area of contact, a gap of 15 mm between steel plate and timber was casted with a polymer concrete (PC), see chapter 4. Hereinafter, this configuration is referred as SC-PC. The PC, consisting of a mineral grit bonded with epoxy resin, fills the gap completely and equalizes tolerances or little wood defects.

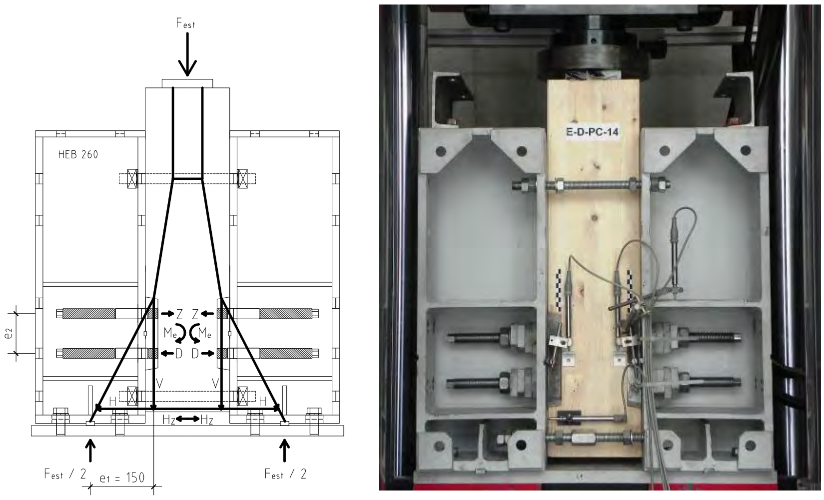

Figure 2: Test setting and specimen for short term push out test

Figure 3: Long term test

The results of the short term test with multiple variation of design are a favourite depth of the carving with 30 mm and a cutting angle of 10°. To evaluate the load bearing capacity, the length in front of the step joint can be impute up to the factor eleven of the depth of the carving. According to DIN EN 1995-1-1/NA (Germany) [9] the value of the shear resistance with $f_{v,k}$ = 3.5 N/mm² can be used. In practice, the concrete slab of the bridge enforces a vertical load, which has a positive influence to shear behaviour.

Additional to the short term push out tests, three series of connectors were tested under long term load. Beside the system of stud connectors (series S), there was a connection made by a notch, filled with concrete (series K) and with X-connectors (series X), made of glued in steel rods. The specimens were designed as symmetric push out tests with glulam and two blocks of concrete. They were placed under a simple roof with outdoor climate. Hence, the mean humidity of the timber was at 14 % by weight. This can be estimated for TCC bridges in a similar way, because of the rain protecting effect of the concrete slab. After a preloading with 40 % of the estimated load capacity, determined by short term experiments, a constant load was applied by a steel spring with a value of 32 % for grooved connection (series K), 26 % for stud connector (series S) and X-connector (series X). The deflection between timber and concrete was observed over a period of little more than five years. The deformation rate is the comparison of elastic start deformation (v_0) and deformation while load period (v_t) as shown in equation (1).

$$k_{def,con} = \frac{v_t - v_0}{v_0} = \frac{v_t}{v_0} - 1 \tag{1}$$

The deformation factor under load at the concrete filled grooved connection (series K) was $k_{def,con} = 1.97$, at the stud connector it was $k_{def,con} = 1.33$ and at the X-connector it was $k_{def,con} = 0.46$. As an approach for creep an extrapolation by exponential function can be used. For approximately 55 years the value for the concrete filled notch can be estimated at $k_{def,con,est} = 2.45$ and for the stud connector at $k_{def,con,est} = 1.5$. In standard DIN EN 1995-1-1/NA [9] the factor for wood is proposed with $k_{def} = 1.65$. For the stud connector as well as for the X-connectors it is recommended to use this value, even the experimental value is obviously lower. In annexation to the long term test and after temporary release from load, the specimens were tested in short term tests again. As expected, the deflection was higher compared to the unstressed connections. However, the load capacity was raised too, maybe due to compression of the contact zone.

Furthermore, investigations on fatigue behaviour of the connecting system "stud connector" were done at nine specimen with stud connectors placed directly into a grooving in the timber (series SC) and 15 specimen with a PC-filled gap between the steel plate of the stud connector and the timber (series SC-PC). The 24 specimens were tested under cyclic load in a frequency of 3 Hz. A specimen had to achieve the minimum number of 2 282 500 load cycles to pass the test completely. A failure at a lower number of load circles generates a point at the fatigue curve. For the upper load level 40 %, 50 % or 60 % of the short term load capacity was chosen. The relation of the upper and the lower level was R = 0.1 and represents the highest amplitude, which is possible at buildings in current practice. One experiment needs about ten days due to the more than two million load cycles. During the test run, the stiffness decreases. With help of the disappeared energy, the reduction ratio of the connecting stiffness k_{dyn} can be evaluated as the ratio of the moduli of initial deflection and reduced stiffness by the cycling load with equation (2).

$$k_{dyn} = \frac{k_{ser}}{k_{ser,red}} \tag{2}$$

For the non modified system the values at load level 50 % was at $k_{dyn} = 3.82$, for the modified specimens of series SC-PC it was at $k_{dyn} = 2.98$. All specimens with a load level up to 50 % passed the whole tests, at load level 60 % two specimen of series SC and five specimen of series SC-PC failed while testing.

As well as at short term tests, the mode of failure always was by shear failure of the timber in front of the joint. At the unmodified system a local compressive failure was detected too. Therefore, the fatigue curve can be established with the fatigue tests and the results of the short term tests. In Figure 4 a comparison of actual research results are shown as 5 %-fractile value of the non modified stud connectors (SC_5%), modified stud connectors (SC-PC_5%), concrete filled grooved connection (K_5%) in accordance to Kuhlmann and Aldi [11], glued-in metal sheet (HBV-Verbinder) [13], a trilinear equation in accordance to Mohr as well as the fatigue curve in accordance to DIN EN 1995-2 [10]. The break down mode of brittle shear failure in front of the connector for both, stud connector as well as for concrete filled grooved connection belongs to fatigue class III in accordance to Mohr [12].

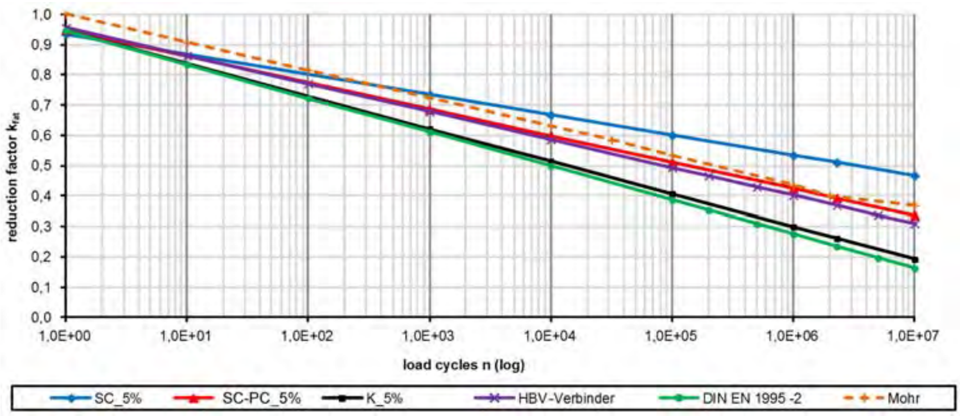

Figure 4: Fatigue curves of several TCC connections [7]

Further investigations for the modification of the shear transmitting area are in progress [14]. Therefore, the timber in front of the stud connector is replaced by PC completely (series SC-PC-I). In a further variation was complete embedding of the steel plate at the front and bottom side into the PC (series SC-PC-II). In both configurations the PC is in a stiff connection to the timber due to the bonding resin. The stiffness of series SC-PC-I is comparable to the modified system of stud connector with the PC filled gap (series SC-PC), which is about twice as high as the direct steel-to-timber connection (series SC). With configuration SC-PC-II the stiffness can be enhanced significantly. While push-out tests the specimen of series SC-PC-I and SC-PC-II showed significant higher load capacities compared to the specimen of series SC and SC-PC.

Figure 5: Stud connector placed into the timber with direct contact (SC), filled gap (SC-PC), embedded into PC (SC-PC-II)

Further research about direct connection by PC casting between timber and concrete is in progress [14], [17]. This enables a very rigid and stable connection. Some special problems caused by stress peaks at the end of the joint, for instance at curved girders and by changing temperature and moisture because of the differing expansion coefficients, are under investigation.

3 Reinforcement - High-Tech Timber Beam®

The load bearing capacity, the span or slenderness, respectively, of a TCC-bridge can be increased, if the timber beam itself has a high performance. Therefore, the multiple reinforced joist was developed in several research projects at the Department of Timber and Masonry Engineering with the name "High-Tech Timber Beam®" (HTB) [15], [16].

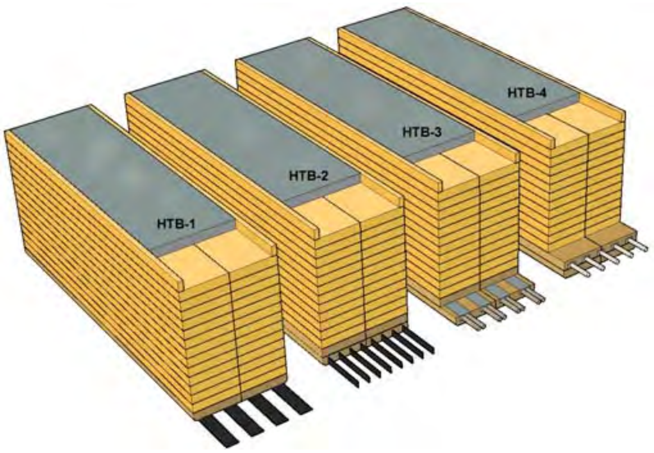

Figure 6: Prototypes of High-Tech Timber Beam® with polymer concrete in the compression zone and reinforcement of the tension zone by CFRP-lamellas (HTB-1, HTB-2), reinforcement steel (HTB-3) and GFRP-bars (HTB-4)

An effective upgrade of load bearing capacity of the joist, made of glulam, is enabled by reinforcement with high-performance materials. The material properties of wood are also depending on size effects. The bending and tension strength decreases at beams with larger height and longer span (effect at higher volume). In order to DIN EN 1995-1-1/NA, the bending strength for design has to be reduced at larger height. To counteract these effects, the tension as well as the compression zone should be strengthened. For high duty, also the areas of high load impact, like connecting points or at the bearings, as well as areas of high shear stress at the end of the beam, are in need of reinforcement. After several investigation and simulation at details and small size specimen, four prototypes of HTB were produced and tested with a size of 40 x 60 cm and a length of 800 cm.

The following steps of reinforcement were done to develop the HTB. For upgrading the compression zone of the bending beam, the upper lamella of the glulam was replaced by a decking of polymer concrete (PC) as descripted in chapter 4. At the bottom side of the glulam, one ore two boards of laminated veneer lumber (LVL) allowed a first homogenization of the timber beams properties. The influence of inconstancies of the wood, as knots, finger joints and defects, are reduced. Additionally, materials with high tension strength and high stiffness like CFRP, GFRP or steel are integrated with casted polymer mortar into the LVL, to enhance the performance of the tension zone.

The reinforcing materials are Carbon-FRP with a high stiffness (E = 210 000 MPa) and very high tension strength ($f_{t,k}$ = 2 500 MPa) as lamellas with a thickness of 1.4 mm and a width of 25 mm or 50 mm (S&P reinforcement). The CFRP-lamellas are mounted to the surface or into precut saw grooves with glue made of epoxy resin and a filling of sand and mineral flour. Reinforcement bars of Glass-FRP ($f_{t,k}$ = 580 MPa, E_{mean} = 60 000 MPa) with a diameter of 16 mm (Schöck Combar) and reinforcement steel ($f_{y,k}$ = 500 MPa) with a diameter of 16 mm are casted into trenches in an additional board of LVL. Therefore, a polymer mortar was used, comparable to the PC but with different grading curve for more flexibility in handling with smaller moulding-size. Due to the significant higher strength and stiffness of the added materials compared with timber, the load capacity increases, the deflection is reduced. Furthermore, the behaviour of collapse is more convenient because the reinforcement carries the beam after rupture of timber in the tension zone. Due to the direct and rigid connection between the materials and layers effected by the adherence of the resin, it is acceptable to calculate with a linear distribution of elongation over the whole cross section. Hereby the different materials can be assessed with their stiffness inside the composite beam. For the combination the coefficient of the materials will be found by the comparison of the MOE's of the main material and the additional materials.

$$I_{eff} = (I_t + A_t \cdot e_t^2) + \sum_{i=1}^{n} n_{re}(I_{re} + A_{re} \cdot e_{re}^2) \tag{3}$$

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

In case of TCC construction with direct mounting of the concrete slab to the HTB, the assumption can be used in the same way. At constructions with the stud connectors, the resilience of the connection toward the concrete has to be considered.

With the higher load capacity, the bearings and areas for load impact receive higher compression stress, mainly perpendicular to the grain. Therefore, the bearings under a high loaded timber joist need a large supporting area. In case of bridges with elastomeric bearings, it is expedient to reduce the size of the bearing. At the HTB-prototypes, the bearings consists of a block of PC with a thickness of a LVL layer and a length of 18 cm, which is a small size for an eight meter long beam with high load capacity. For better stress distribution the bearings are completed with additional PC-filled drill holes with optimized angles to the gradient of strain, see Figure 7, left hand side. Figure 7 right hand side shows the indentation at the bearing compared to an unreinforced support design. In the linear elastic range a strengthening effect of nearly 200 % could be attained. This is an effective and comparatively easy way to enforce the load capacity and reduces the deformation of the timber perpendicular to the grain. The short length of bearing is expedient for girders with large span wide on elastic bearings, because it allows smaller height of the elastic bloc. More detailed information about reinforcement of load introduction areas are published by Haedicke [18].

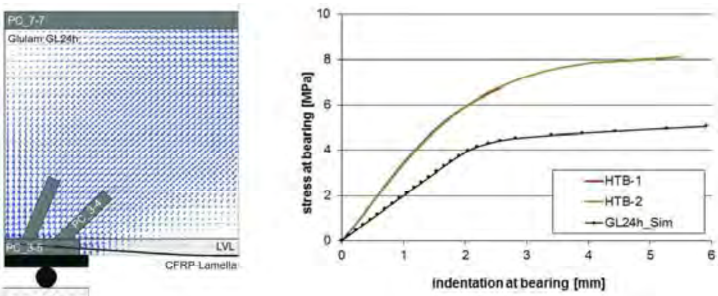

Figure 7: Bearing with PC and stress distributers (left); Effect of reinforcement (right)

Furthermore, the end anchorage of the reinforcement of the tension zone is integrated into the bearings. Beside high compression forces, there are peaks of shear stress at the end of the beam and at the anchorage zone of the reinforcement. 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 at the ending area are unavoidable, so an effective end-anchorage of the lamellas is important. The bearings serve as end anchorage zones for the reinforcements by embedding the CFRP-lamellas in the PC-blocks.

To counter the shear stress in the beam's end-sections, glued-in steel rods are integrated into the timber. The mounting angle was 45° to the grain in tensions direction. During the bending tests the strains on the surface of the reinforced areas were measured with close-range photogrammetry (CRP) (Fig. 8, left side). The strengthening effect can be derived by comparison with the results of an FE-simulation without shear- and bearing-reinforcements (Fig. 8, right side).

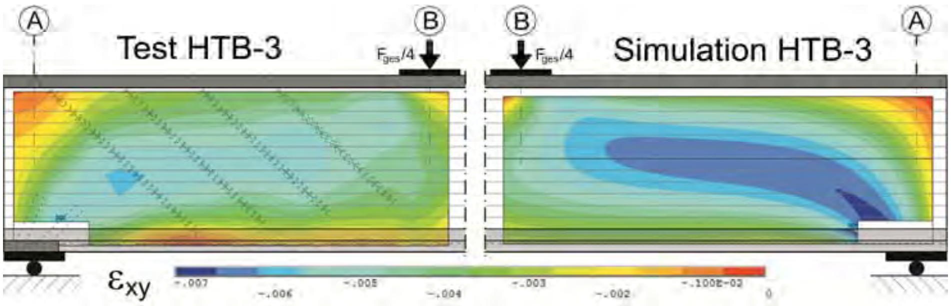

Figure 8: Strain distribution as test result of reinforced HTB (left) and as simulation without shear and bearing reinforcement (right)

4 Material of Polymer Concrete

For the reinforcement of the areas with compression stress and to mount the reinforcement materials to the timber or cast it into holes, an epoxy based resin with several fillings of minerals was used as polymer concrete (PC). The PC has high compression strength and also high stiffness because of its high rate filling with mineral grits. A two component resin reveals a high adhesive potential for many materials and especially for wood, minerals, glass and steel. Hence, it is highly convenient to provide rigid bonds between those materials. For the experiments a pure epoxy-system (Bennert COMPONO®) was used with different mineral fillings from rock flour to gravel with a special grain-size curve to 7 mm. For diverse applications at the reinforced timber beam or for the connection joint at TCC, special recipes were created. One of the compositions, called PC 3-5 in the following table, has a national technical approval [19] for rehabilitation of historic timber beams.

Further investigations were done on the different kinds of polymer concrete. Tests to determine the properties of PC were done as well as several tests for bonding behaviour towards wood, steel, CFRP or GFRP and for casting technologies. Contact free measurement by industrial CRP was used in addition to traditional measurement equipment. The CRP-technique enables the measuring of the progression of deformations, cracks and deteriorations during loading and unloading of specimen by computer controlled high resolution cameras. Thus, it is possible to setup and calibrate a numeric simulation model. Beside usual tests of tension and compression strength, there are extended investigations for deformation, deterioration, shear and long term behaviour.

The next table displays the material properties for selected variations of polymer concrete, used at the different applications for instance in upgrading the High-Tech Timber Beam. With an increasing rate of mineral filling, at the table from left to right, the Young's-Modulus is rising. Because of the high tension capacity of the resin, the tension but also slightly the compression strengths decrease with a higher percentage of minerals. [20]

Table 1. Material Properties of Polymer Concrete

Polymer Composite	**PC 1-2**	**PC 3-4**	**PC 3-5**	**PC 7-7**	**PC 3-7.5**
Application	Glue	Grouting	Bearings	Compression Zone	In Test
Max. grain size	Sand / Flour	3 mm	3 mm	7 mm	3 mm
Resin : Aggregate	1 : 1.75	1 : 4	1 : 5	1 : 7	~ 1 : 7.5
Compression Strength [MPa]	102	97	97	90	142
Tension Strength [MPa]	28	19	19	16	36
Shear Strength [MPa]	35	34	26	25	-
MOE (compression) [MPa]	10 200	16 800	20 700	22 800	> 42 000
Radical strain coefficient (under compression)	0.30	0.26	0.24	0.24	-
Bulk density [g/cm³]	1.70	1.90	2.00	2.05	2.55

5 Conclusion and future work

For more applicability of wood at infrastructural buildings, it is possible to use TCC-bridges with special performed joints and by the use of girders made from reinforced timber.

At TCC systems, the stud connectors are very convenient for the joint between concrete slab and wooden girder. With the modification of casted polymer concrete (PC) in the load-transmission area, the stiffness can be enhanced. Because of the equalization effect to the tolerances due to manufacturing, it 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. Ongoing experiments with larger areas of PC show higher load bearing capacity. Therefore, the investigation and optimization of the joint and the use of PC will be the content of research for TCC with stud connectors.

With reinforcement of timber beams by high-tech materials, it is possible to raise the load bearing capacity and the safety of the girders. By modifying the areas of high load impact, especially at the bearings, it is possible to improve the efficiency. The further activities are targeted on the technology and dimensioning of reinforcement of those areas.

The wooden structural elements can be applied as ready to use elements. They are easy to transport and assemble at the site and serve as scaffold for concrete works. Therefore, the construction time, and with this the period of road blocking, can be reduced significantly. This reduces the expense for the construction. The HTB can be produced with more than 95 % of renewable products if the petrochemical resins be replaced with epoxy resins on base of phytogenetic oils. Hence, infrastructural buildings can be built more sustainably and economically in future.

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Timber enforces concrete – the first hybrid timber arch bridge for wildlife

Antje Simon[1], Karl-Heinz Lorbach[2], Gerhard Setzpfandt[3]

Summary

In 2012, the new green bridge “Heinzenberg” was built near Nettersheim in North-Rhine Westphalia. This bridge is intended to enable wild animals to cross the busy highway A1. Considering the development of timber-concrete-composites in the last few years, the new bridge should be designed as the first hybrid arch bridge in Germany. In the paper the investigations about curved timber-concrete composites and the design process are described. It is explained that a stiff connection of timber and concrete in curved profiles causes unacceptably high constraint forces resulting from the different time- and climatic dependent behaviour of both materials. A newly developed flexible connector type permitting a displacement between concrete and timber is shown. Construction details and the building process are presented as well as the measures for wood protection. The first hybrid timber-concrete arch bridge is an ecological alternative to pure concrete vaults and a further development of pure timber arch bridges.

Key words: green bridge, hybrid arch bridge, timber-concrete-composite, flexible joint, wood protection

1 Introduction

A new green bridge or wildlife crossing for wild cats was built across the highway A1 between Cologne and Trier (North-Rhine Westphalia) in 2012. The bridge “Heinzenberg” is situated near Nettersheim in the nature reserve “Hohes Venn – Eifel”. It has been erected to reconnect the wildlife habitat of the wild cats which had been cut by the highway. By that time wildlife crossings consisted of concrete or timber in Germany [1], [2], [3]. But the use of the natural and ecological building material timber is deeply rooted in the region around Nettersheim. The wood competence centre Rheinland, working since 1998 in Nettersheim, is well-known throughout Europe. With a new green bridge meanly consisting of timber should be set another symbolic landmark for ecological innovation and sustainability. Considering the actual development of heavy timber-concrete-composite bridges [4], [5] enhancements of the existing timber bridges were intended.

The building site is situated in a cutting area of the landscape, where the foundation soil consists of solid rock. These boundary conditions were favourable for building an arch bridge.

For those reasons, a hybrid timber arch bridge has been built for the first time in Germany respectively in Europe.

2 Curved timber-concrete composites

A timber-concrete-composite cross section usually consists of concrete as the upper layer and timber as the lower layer. Connectors in the contact zone support the layers acting together as a composite material. The most important benefit of these hybrid bridge types is the ideal constructive wood protection by the concrete deck. It has been investigated, that the life time of a protected timber bridge is twice to thrice as high as the life time of a non-protected one. Furthermore, a lot of construction details well proofed in concrete bridges could be used without any additional design effort. In comparison to concrete bridges a hybrid timber bridge is much lighter, more ecological and sustainable.

At the beginning of the planning process, the arch has been designed along its pressure line. Thus the materials should be under compression in most load cases. That design is favourable because concrete and timber support compression loads easily. Static analysis has been done using a lot of different systems varying stiffness and material properties. It has been investigated that the different time- and climatic depending material behaviour of concrete and timber causes excessive forces. Due to concrete shrinkage or by cooling down the complete cross

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section, the concrete part wants to contract. But the very stiff timber cross section prevents the concrete's shortening. High tension stresses arise in concrete and it cracks. The same effect is caused by timber swelling. Thus the original idea to use the arches expedient load carrying capacity as a moment-free compression system was not applicable.

Also a second special feature of curved hybrid structures was detected. Due to warming the complete cross section or shrinkage of timber, the concrete shell lifts up. High tension forces are caused in the connection zone additionally to the regularly acting shear forces. Connectors carrying both, high shear and tension forces, do not exist up to now.

Finally it was determined, that an arch bridge as a timber-concrete-**composite** structure could not be realised. It is necessary to eliminate the high constraint forces by tolerating a displacement between concrete and timber.

3 Analysis of design variants

In the further planning two different variants have been investigated:

1 a timber arch with a concrete shell (figure 1)
2 a concrete arch with timber ribs (figure 2)

In the first variant massive log-glued laminated timber ribs carry the vertical and horizontal loads in the arches plane. Prefabricated reinforced concrete elements span perpendicular to the timber ribs to distribute the loads and to brace the bridge in cross direction. Respectively two single timber ribs are connected in the field to one bow with restraint abutments. To reduce the constraint forces caused by concrete shrinkage the bridges cross section is divided into five 8 – 12 m long segments. Furthermore five expansion joints are arranged in the longitudinal direction to enable small displacements of the concrete elements along the timber surface.

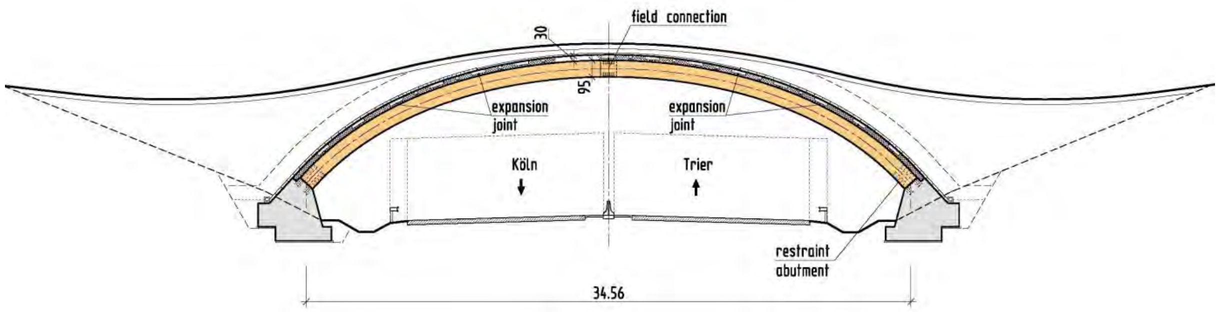


Figure 1: Longitudinal section of variant 1: a timber arch with a concrete shell

The bridge of the second variant consists of a very slender reinforced concrete arch enforced by massive log-glued laminated timber ribs. In contrast to variant 1 the concrete arch is erected with restraint abutments and the timber ribs hang only below without fixing at the bases and at the top of the arch. There is only one fixed connection per timber element nearly the quarter point. All other connectors between timber and concrete are flexible, so that the constraint forces result from the different hygrothermal and long-time material behaviour are minimized. The concrete arch carries the normal forces and braces the bridge in the cross direction. The timber ribs carry the major part of the bending moments and brace the slender concrete arch in its plane. Due to the varying thickness of the timber ribs and the lack of their supports, the design of the composite bridge seems like a three-hinged arch.

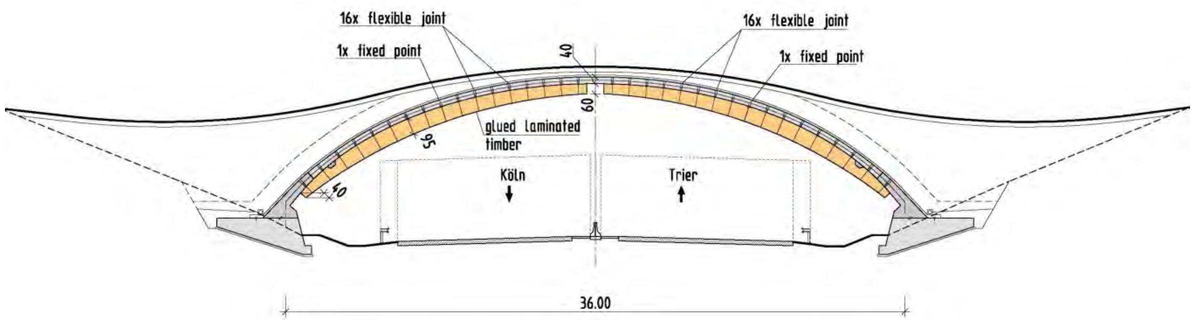


Figure 2: Longitudinal section of variant 2: a concrete arch with timber ribs

Both variants showed advantages and disadvantages. In both variants the concrete realises a perfect constructive wood preservation for the timber ribs. Because of the reduction of the expansion joints in the concrete deck and the minimized constructive junctions in the timber the building owner decided to build the second variant where stiff timber ribs enforce a very slender concrete arch.

4 Technical details of the bridge construction

Due to the topography and the route data of the highway, the following parameters result for the bridge design:

Span:	36,00 m
Clear width:	34,05 m
Smallest clear height:	5,304 m
Crossing angle:	100 gon
Usable width between the protective walls:	50,00 m
Construction height:	0,40 – 1,35 m
Total bridge area:	2112 m²

The bridge should be used regularly as a green bridge and wildlife crossing. Therefore it is dimensioned considering a regular live load of 10 kN/m². That load includes the planting. In accidental situations fire engines or forestry vehicles should cross the bridge. For that reason the load of a tandem system according to DIN Technical Report 101 is additionally considered. Beside the live loads the different temperature load cases, the shrinkage of the concrete, the swelling and shrinking of the timber and different settlements of the abutments are included in the calculation.

According to the assembly process, the structural analysis of two different systems was necessary:

- A Temporary construction stage (single timber ribs carrying the dead loads and the loads during the concreting)
- B Final stage (hybrid construction with concrete and timber acting together for load carrying)

In comparison to the calculation of a simple arch bridge the static analysis of such a hybrid structure is much more complicated. A lot of different systems have been evaluated considering different stiffness approaches, partial safety factors and modification parameters due to the different calculation situations in ultimate and serviceability limit state.

The hybrid bridge is designed along its pressure line. As a result, dead loads and earth pressure initiate mostly compression forces in the concrete arch. The thickness of the reinforced concrete shell varies from 40 cm at the top to 65 cm at the base. This slender concrete shell is enforced by 80 cm wide log-glued laminated timber ribs being arranged at a centre distance of 2,0 m. The thickness of the sickle-shaped timber ribs varies from 40 cm to 95 cm according to the distribution of the bending moments.

The cross section parts of concrete and timber are combined flexibly by steel tension ties (figure 3). Holes in the timber ribs and pin joints at the steel ties allow different displacements of concrete and timber and minimize the constraint stresses.

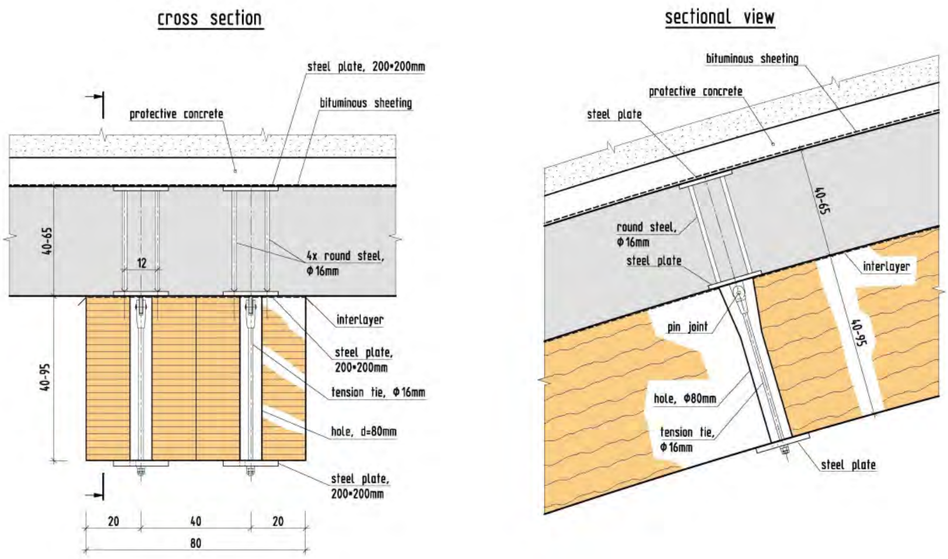


Figure 3: Flexible joint

At the quarter point of the arch concrete shell and timber ribs are connected by a fixed joint (figure 4). This connector consists of a 5 cm thick steel plate with studs on the concrete side and four tension ties on the timber side. The steel plate is set into a timber groove. To protect any slip the groove is filled with polymer concrete.

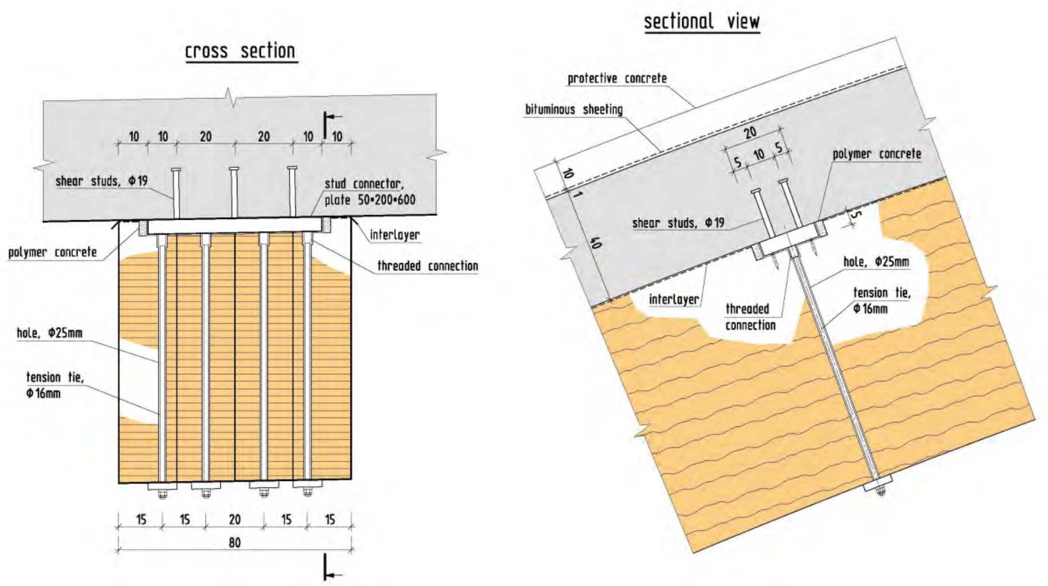


Figure 4: Fixed joint

All steel joints, plates and ties are made of stainless steel because they are situated in the spray zone of the highway with high exposure to chloride.

5 Measures for wood protection

A lot of measures for wood protection have been used to increase the durability of the construction:

- The concrete shell with its bituminous sheeting protects the timber ribs from weathering. The measuring of wood moisture content in similar bridges showed values from 12 to 18 %. In this value range a fungal growth is excluded.
- A double interlayer between timber and concrete realises an additional protection.
- The massive log-glued laminated timber ribs show a better area/perimeter ratio in comparison to normal glued laminated timber girders. The outside climatic variation causes a modification of the wood moisture content only in the outer zones but not in the centre of the massive ribs.
- The high area/perimeter ratio is also favourable for the fire resistance.
- The timber is manufactured and assembled with a moisture content of 15 %. The assembly of technical dried and glued timber reduces the risk of insect attacks.
- Due to the design of the timber ribs, moisture accumulation is unlikely. The ribs could dry easily and fast, especially in the end grain areas.
- All timber ribs are visual testable.
- All flexible and fixed joints between concrete and timber are detachable. So it would be possible to exchange a single timber rib if it is damaged.
- Additionally the timber ribs have got a chemical treatment and a moisture protection coating.

Regarding all above mentioned constructive measures the green bridge could be considered as a protected timber bridge according to DIN EN 1995-2/NA:2011-08. A monitoring system was applied to observe the moisture content and the climatic variation permanently over a period of at least two years.

6 Assembly of the bridge

An optimized assembly was strictly required by the highway administration to minimize the traffic obstruction. A total blocking of the highway was only allowed during the concreting. During the other phases it was permitted to reduce the normal four lanes to temporary two lanes, one lane in each direction. Figure 5 shows the construction progress which was planned in six phases.

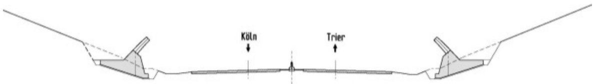
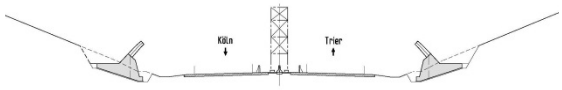
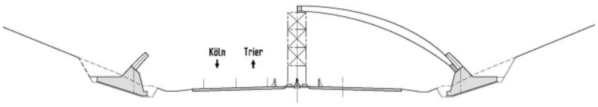
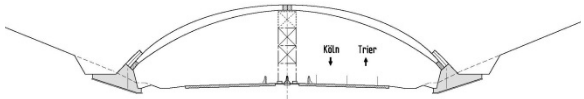
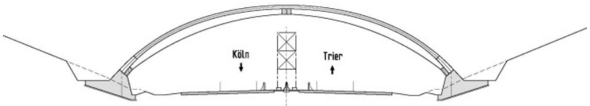
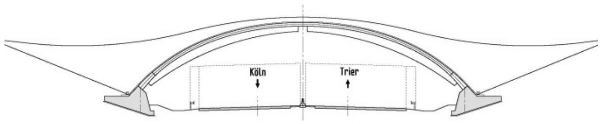
Phase 1  Production of the foundation	Phase 2  Erecting of the temporary support in the middle of the highway
Phase 3  Lifting and adjustment of the timber ribs and installation of the formwork above the Trier lane	Phase 4  Lifting and adjustment of the timber ribs and installation of the formwork above the Köln lane, assembly of temporary three-hinged arches using the timber ribs
Phase 5  Lowering of the middle support, Concreting	Phase 6  Completion

Figure 5: Assembly of the green bridge

The green bridge consists of the following main volumes:

Concrete (foundation, shell and bridge caps):	2030 m³
Timber:	774 m³
Reinforcement steel:	283 t
Steel joints:	1300 units

Four concrete pumps and four groups, each with 7 - 8 concrete workers, acted continuously for concreting the bridge over a period of 14 hours (figure 6). The concreting regime was planned with a maximum difference of 50 cm between the concretes heights at both sides of the arch. During the concreting the displacements of the timber ribs were permanently controlled and compared with the calculated values [6].


Figure 6: Building progress: adjustment of the timber ribs (left side) and concreting (right side) (photos: K.-H. Lorbach)

7 Conclusion

As shown in figure 7 the arch bridge “Heinzenberg” is an innovative construction building using a very high quantity of timber. It could be a pilot project for further ecological wildlife crossings.

Applied research was necessary to design the wildlife crossing as an arch bridge consisting of concrete and timber. It was investigated that it is not possible to create a rigid connected timber-concrete-composite bridge with a curved longitudinal section. Concerning hybrid arch bridges it is essential to allow the displacement between concrete and timber to reduce the constraint forces.

The building process should be further developed according to an optimization of the assembly technology. Using high-performance concrete the thickness of the concrete shell could be reduced to increase the efficiency of the construction.

Figure 7: Green bridge „Heinzenberg“ near Nettersheim *(photos: K.-H. Lorbach)*

Hybrid timber-concrete arch bridges could be an ecological alternative to pure concrete vaults and a further development of pure timber arch bridges due to their higher constructive wood protection and fire resistance.

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Asphalt surfacing on timber bridges

Florian Scharmacher[1], Andreas Müller[2], Maurice Brunner[3]

Summary

It is difficult to get adequate information about the load-bearing properties of the different asphalt systems which are used for the surfacing of modern timber road bridges. The authors have participated in a research project to investigate the load-bearing behaviour of different, asphalt-based road surfacing systems under service loads. First, suitable material combinations and layer compositions were selected for detailed research. The transfer of horizontal forces through the composite construction was a special research interest. A number of test series was carried out to investigate the adhesion between the surfacing and the timber deck. The test results were comparable to those obtained for surfacing placed on concrete and steel bridge decks. The tests confirm that the requisite bonding strength can be achieved with similar systems such as those used in steel and concrete bridges. Structural recommendations for the practical application are also presented in the paper.

Keywords: asphalt surfacing systems, shear resistance, adhesion tensile strength, blistering, monitoring

1 Introduction

Nowadays, mastic asphalt and rolled asphalt are both used for the surfacing of timber road bridges. A durable sealant between the asphalt layer and the timber deck is of enormous importance for both systems [1], [2]. It protects the timber deck from direct contact with the molten asphalt during the pouring phase. Later on, it prevents the entry of water. In systems without shear connection between asphalt structure and bridge deck, there is the risk of the development of "surface waves" caused by high braking and acceleration forces. In Germany for instance, only systems with shear connection are permitted for traffic road bridges.

In surfacing systems without a shear connection, a separation layer, e.g. glass-fleece and oil-impregnated paper, lies between the timber bridge deck and the bottom from the sealant, which is attached to the asphalt structure on top (Figure 1). In a typical system with shear connection, the surface coating replaces the separation layer (Figure 2). The surface coating is the “glue” which holds the timber bridge deck and the sealant together.

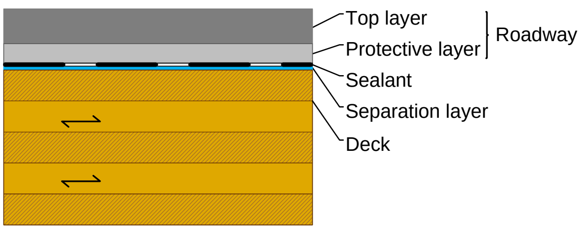

Figure 1: Sketch of a system without a shear connection between asphalt and bridge deck

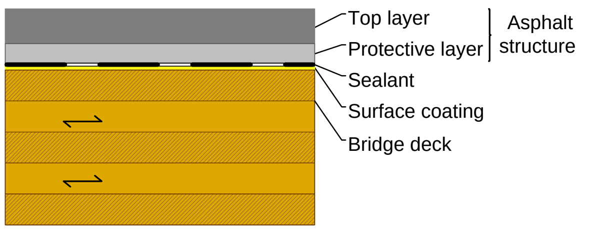

Figure 2: Sketch of a system with a bonded shear connection between asphalt and bridge deck

In comparison to concrete and steel bridges, research work on the surfacing of timber bridges has been rather modest. The authors have participated in a research project to investigate the properties of different, asphalt-based road surfacing layers under service loads. The research project was concerned with the shear resistance of the surfacing, and with the problem of “blistering” which may occur when hot asphalt is poured on a timber deck. The research work included the scientific observation and monitoring during the renovation of the surfacing of the Bubenei Bridge in Canton Berne, Switzerland [3].

The paper will give an overview of the test set-ups and the results obtained. The monitoring of the resurfacing of the Bubenei Bridge gave useful inputs which also helped in the formulation of recommendations for the practical application.

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2 Materials and methods

2.1 Shear tests

Systems without a shear connection and systems with a shear connection between asphalt structure and deck material are both used for road bridges. The tests performed during the research work were limited to systems with a shear connection. All test specimens included a layer of temperature modified mastic asphalt (pouring temperature 200 °C) because they are more favoured in systems with shear connection. No rolled asphalt was used. Figure 3 shows the principal cross section of the test specimens.

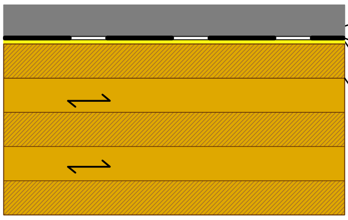

Figure 3: Principal cross section of the shear test specimens

The following parameters were also investigated:

a) The deck material, such as steel, concrete, cross laminated timber (CLT) and laminated veneer lumber (LVL).

b) The sealant system, such as polymer bitumen membranes and liquid synthetic sealants (based on polymethyl methacrylate, PMMA) together with various surface coatings.

Table 1 explains the parameters of the test specimens for the shear tests. Three specimens were manufactured for each of the 12 layer compositions shown.

Table 1: Layer composition of the test specimens for the shear tests

Deck material	**Surface coating**	**Sealant**
Concrete	Sanded epoxide	PBM
Concrete	LS primer	LSS
Steel	Primer	PBM
Steel	LS primer	LSS
CLT	Sanded epoxide	PBM
CLT	LS primer	PBM
CLT	LS primer	LSS
CLT	Epoxide primer	LSS
LVL	Sanded epoxide	PBM
LVL	LS primer	PBM
LVL	LS primer	LSS
LVL	Epoxide primer	LSS

PBM: polymer bitumen membrane; LSS: liquid synthetic sealant

The selected material combinations and layer compositions were subjected to shear and tensile bonding (adhesion) tests. Figure 4 and Figure 5 show the set-up for the shear tests.

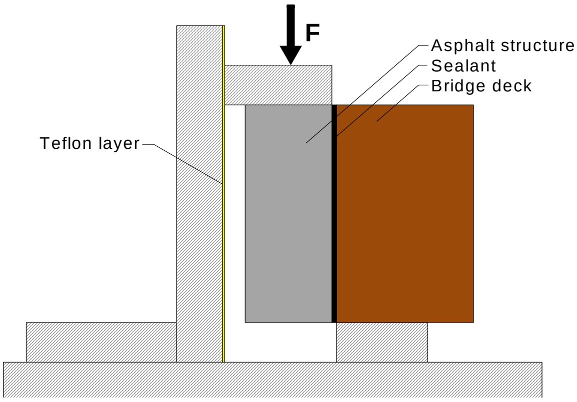

Figure 4: Test set-up for the shear tests

Figure 5: Detail of the test machine readied for a shear test

2.2 Adhesion tensile tests

Adhesion tensile tests are the most common in-situ testing method. Material tests can be carried out on site with a mobile testing machine. The test arrangement is regulated in the Swiss Standard SN 640450a.

The layer compositions of the test specimens are listed in Table 2 below. Three specimens were tested of each layer composition. Only a relatively small number was selected for the adhesion tensile tests: these preliminary tests were intended to give a general idea of the adhesion properties of the surfacing on the wooden base. A larger number of samples will be tested at a later stage in order to make a statistical analysis of adhesion tensile tests.

Table 2: Layer composition of the test specimens for the adhesion tensile tests

Deck material	Surface coating	Sealant
LVL	Sanded epoxide	PBM
LVL	LS primer	LSS

PBM: polymer bitumen membrane; LSS: liquid synthetic sealant

The researchers were primarily interest in the adhesion tensile strength between the sealant and the wood base, so the specimens were prepared without any asphalt layer (Figure 6).

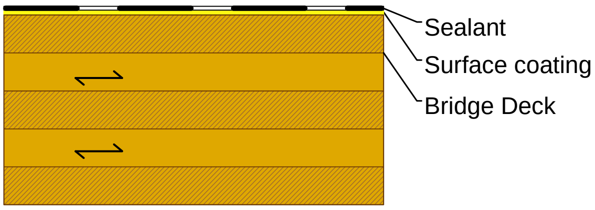

Figure 6: Principal cross section of the tes specimens for the adhesion tensile tests

The mobile testing machine used has a stamp-like head called “indenter” which pulls the test specimen from the wooden base. The surface of the test specimens needed to be properly prepared so that they could be glued to the indenter of the testing machine. The PBM polymer bitumen membrane was heated with a Bunsen burner so that the top-most layer of sanded epoxide could be removed. The primer of the LSS liquid synthetic sealant was removed with alcohol.

A layer of cyanoacrylate adhesive was used to attach the test specimen onto the indenter of the testing machine. The testing area of the surfacing was cut free from the surrounding surfacing area, so that only this area was subjected to the pulling force (Figure 7, Figure 8). The tests were carried out in the laboratories of the Bern University of Applied Sciences in Biel by the company BTS Baucontrol. The tests were force-regulated at a rate of 300 N/s.

Figure 7: Mobile adhesion tensile testing machine of the company BTS Baucontrol

Figure 8: The indenter is pulling at the barely visible PBM sealant underneath it

2.3 Resurfacing Bubenei Bridge

The surfacing of the Bubenei Bridge needed to be redone because of numerous cracks in the asphalt. No sealant was used in the old surfacing. The timber deck had a very high moisture content of 18 – 20 %. The distribution was very uneven: in some places the moisture content was measured to be over 100 %.

For cost reasons, the owners and the project engineer decided to leave the timber deck in position despite the extraordinary moisture content. Their reasoning was that the new sealant would prevent more water from getting to the timber deck. The fact that the new sealant would also prevent the timber from drying upwards was an accepted risk: the engineer estimated that the drying downwards away from the sealant would be slow but adequate.

Because of the high moisture content of the timber deck, there was a risk of severe blistering when the mastic asphalt would be poured. Despite the risk of “surface waves” caused by braking and acceleration forces, the project engineer decided to use a surfacing system without a shear connection to the bridge deck. The selected solution is shown in Figure 9 below: it had two important advantages to mitigate the risk of blistering. First, a separation layer of glass-fleece and oil-impregnated paper was combined with closely drilled release openings for the controlled discharge of any water vapour which might form during the pouring of the mastic asphalt. The second measure was the massive reduction of energy input by using temperature-modified mastic asphalt with a relatively low pouring temperature of 200 °C. The thickness of the lowest asphalt protective layer was reduced to 25 mm and it was placed carefully by hand.

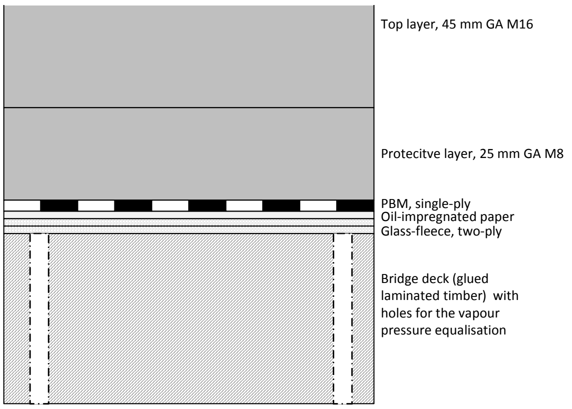

Figure 9: Layer composition of the new surfacing of the Bubenei Bridge

The researchers were given two monitoring assignments on the Bubenei Bridge. Before the new surfacing was poured, they mounted temperature gauges at different depths of the timber deck to clarify if the temperatures would rise high enough to cause the moisture in the timber to vaporise. Moisture measuring instruments were mounted in several places to monitor the expected, long-term drying of the timber downwards, away from the newly placed sealant.

3 Results

3.1 Shear tests

The test results showed different load-bearing behaviours for the two sealants used. Layer compositions with polymer bitumen membranes (PBM) exhibited very ductile behaviour: the yield shear stress of 0.2 - 0.6 N/mm^2 was attained at an elastic deformation of 1 - 2mm. The plastic deformation after the yielding was considerable: the tests were stopped after a deformation of 10 mm was attained (Figure 10). After the tests, the elastic deformation of the specimens was slowly but fully recovered after a few days.

Figure 10: Plastic deformation of a test specimen with PBM sealant on CLT (left) and on concrete (right)

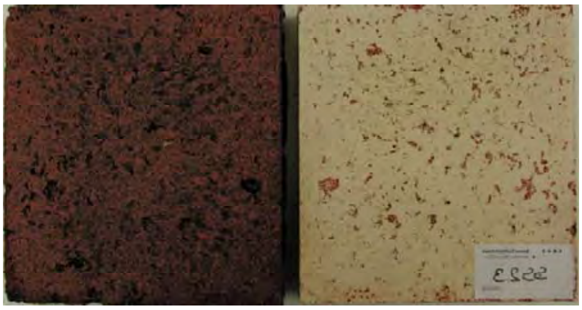

Figure 11: Brittle failure of the shear connection of the LS sealant on a steel deck

On the other hand, layer compositions with liquid synthetic sealants (LSS) exhibited very brittle load-bearing behaviour. The failure shear stress was 2.5 - 5 times higher than the values obtained for the specimens with PBM. In all the tests, failure always occurred at the interface between the LSS and the mastic asphalt (Figure 11).

The shear force / deformation diagrams below confirm that the results of the shear tests depended mainly on the type of sealant used: the test specimens with PBM all exhibited ductile behaviour, whilst the LSS all exhibited brittle failure modes. The type of bridge deck did not seem to be of any importance: there were no significant differences when timber, steel or concrete bridge decks were used (Figure 12, Figure 13, Figure 14, Figure 15).

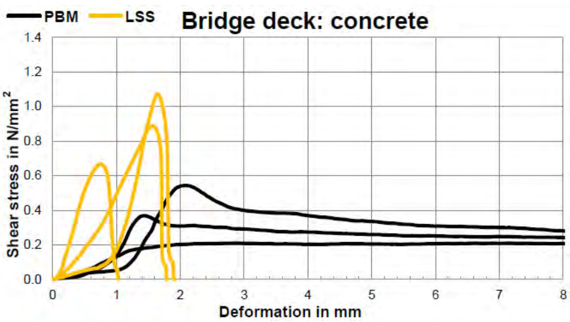

Figure 12: Shear stress and deformation diagrams for different surface compositions on a concrete deck

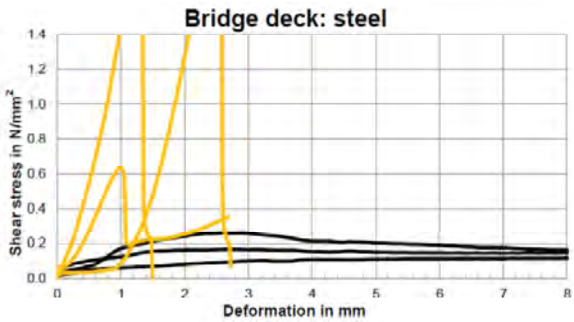

Figure 13: Shear stress and deformation diagrams for different surface compositions on a steel deck

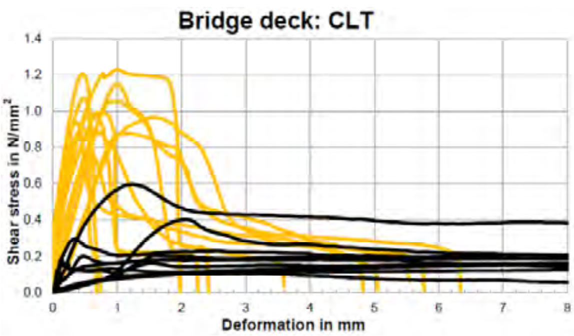

Figure 14: Shear stress and deformation diagrams for different surface compositions on a CLT-deck

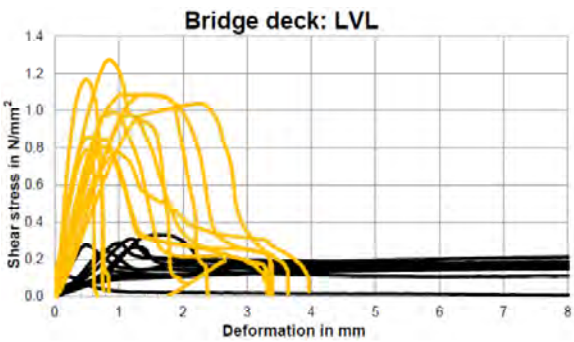

Figure 15: Shear stress and deformation diagrams for different surface compositions on a CLT-deck

3.2 Adhesion tensile tests

All test specimens fulfilled the adhesion tensile strength requirements according to the standard SN 640450a:2009. Figure 16 and Figure 17 show two test specimens after the adhesion tensile tests. The evaluation of the adhesion tensile tests for the PBM sealant is shown in Figure 18. The required strength values (red line) were surpassed by all test specimens.

In the case of the test specimens with LSS liquid synthetic sealant, the first 3 tests had to be repeated because the adhesion of the test specimen to the indenter of the testing machine yielded prematurely. The test specimens 4 – 6 fulfilled all the requirements of the standard with regard to the individual strength value and the average values (Figure 19).

Figure 16: Test specimen with PBM sealant after successful adhesion tensile test [5]

Figure 17: Test specimen with LSS sealant after successful adhesion tensile test [5]

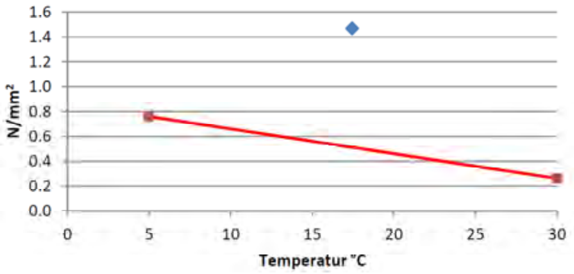

Figure 18: The average value of the PBM specimens (blue dot) lies well above the red line of SN 640450a:2009 [5]

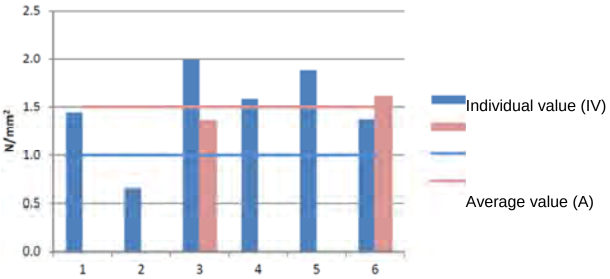

Figure 19: The LSS specimens fulfil the requirements of SN 640450a:2009 for the adhesion tensile test [5]

3.3 Observed blistering during the manufacture of the test specimens

During the manufacture of the test specimens for the shear tests, in particular during the pouring of the hot asphalt onto the bridge deck, some remarkable observations of blistering were made. Many of the specimens with an epoxide primer suffered some clear blistering. The epoxide primer is known to be open to water vapour diffusion. The heat of the asphalt apparently caused water vapour to rise from the timber to accumulate directly at the bottom face of the sealant. The water vapour caused a partial separation of the sealant from the timber deck: in some places it penetrated the sealant and collected as “blisters” in the asphalt (Figure 20, Figure 21, Figure 22).

Figure 20: No blistering observed in this specimen of asphalt surfacing on CLT

Figure 21: Clear blistering in the surfacing of this specimen of asphalt surfacing on CLT

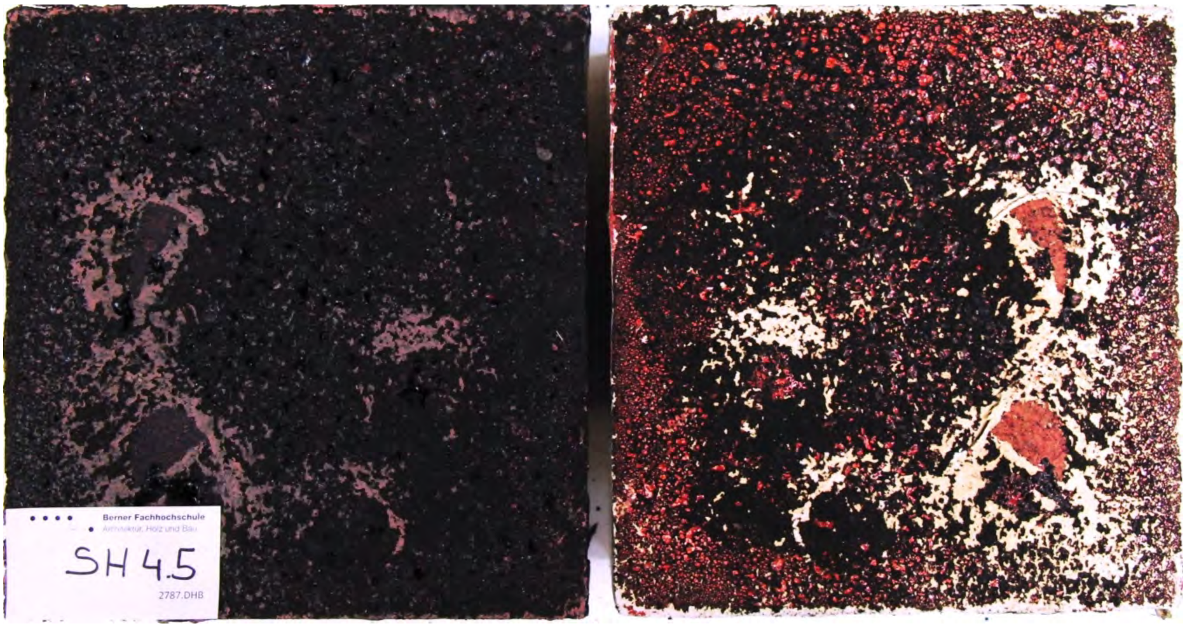

Figure 22: This test specimen has been cut open to display the clear blistering

The effect of the blistering which occurred in some specimens was evident during the later shear tests. The shear strength of samples with blisters was reduced by approximately 10 - 15% as compared to an undisturbed sample. The obvious reason was the reduced contact area for the shear force. It was also observed that the test specimens which exhibited blistering were more ductile in their load-bearing behaviour than the specimens which did not suffer blistering. A plausible explanation might be that the weakened material around the blisters were deformed more readily but then got caught in the indentations on the wood surface (Figure 23, Figure 24).

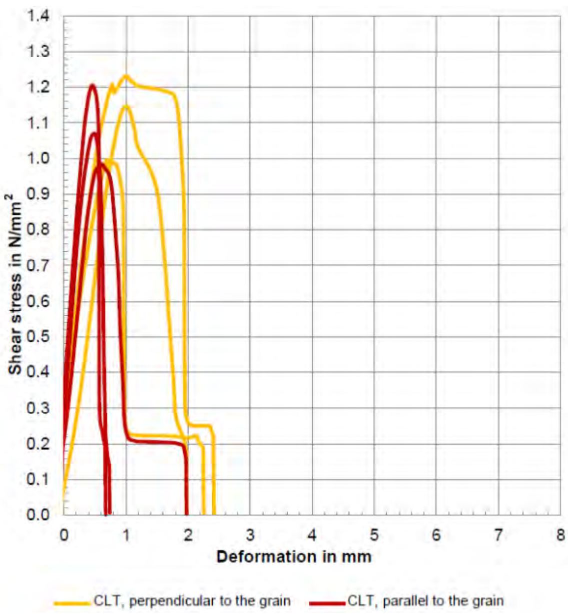

Figure 23: Shear stress / deformation diagram of a test specimen without blistering

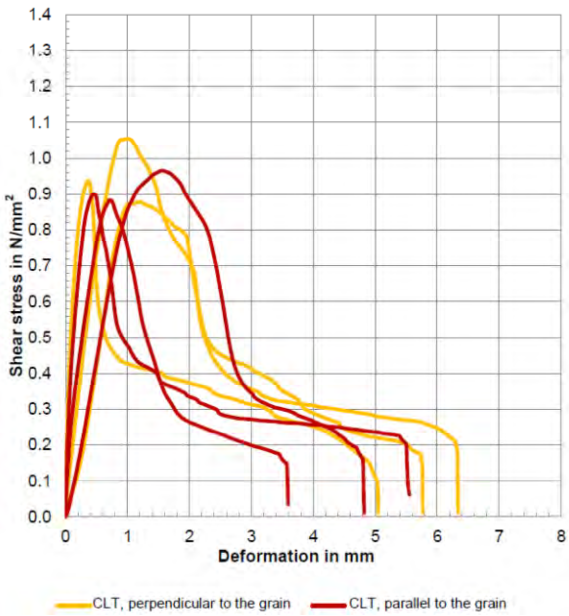

Figure 24: Shear stress / deformation diagrams of test specimens with blistering

3.4 Resurfacing of the Bubenei Bridge

The research team was allowed to scientifically observe the renovation of the surfacing of the Bubenei Bridge (Canton of Berne, Switzerland). The massive timber deck was surfaced with a 25 mm thick asphalt structure [4] supplied with vent holes but without a shear connection (Figure 25). Despite the high wood moisture content, no increased blistering was observed. The temperature in the wooden deck was observed to rise very slowly during the application of the temperature-modified asphalt: a sudden evaporation of water could not occur according to the temperature measurements (Figure 26).

Figure 25: Picture of the Bubenei Bridge during the asphalt coating

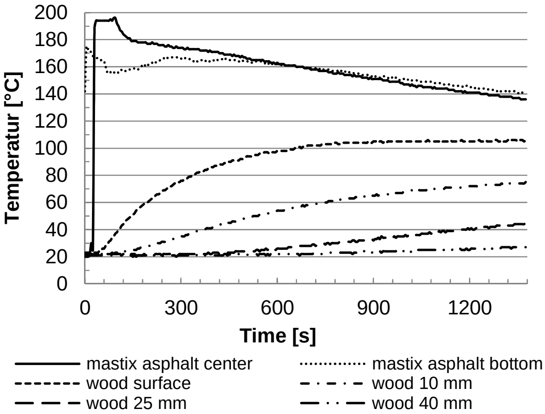

Figure 26: Temperature profile for a timber deck during asphalt coating

The second task of the researchers was the long-term monitoring of the moisture content of the timber beams of the bridge deck. These had suffered considerable wetting because no sealant had been foreseen in the old surfacing. Because of cost reasons, the engineers had decided to reuse the beams. The new sealant prevented a drying upwards through the new surfacing. There was a risk that the drying of the beams downwards might be too slow to prevent fungus attack. The monitoring of the slow drying of the wood is still on-going.

4 Conclusions

The research work largely confirms earlier research work that asphalt surfacing types which are typically used for steel and concrete bridges can – with some appropriate modifications - be safely and reliably used for timber bridges as well. The shear tests performed confirm that the different layer compositions perform equally well on timber, steel or concrete decks.

A durable sealant between the asphalt layer and the timber deck is an important water protection for the timber material. For timber bridges with a shear connection between the asphalt structure and the timber deck, a sealing with a vapour proof surface coating prior to the installation of the sealant or the pouring of the hot asphalt is essential.

Another important need is to prevent blistering, because timber decks typically contain much more moisture than steel and concrete decks. Surfacing types which use an epoxy primer to help activate the adhesion between the sealant and the timber deck are particularly at risk with regard to blistering hazards. This hazard can be mitigated by reducing the energy of the poured asphalt with three measures: first, temperature-modified asphalt with a pouring temperature under 200 °C should be used. Secondly, the protective asphalt layer lying directly on the sealant should not exceed 25 mm. Finally, the hot asphalt should be placed carefully by hand and not with a road finishing machine.

The research work showed that the load-bearing behaviour of the bridge surfacing under shear forces was largely determined by the type of sealant used. Two important sealants types were thoroughly investigated. Although the sealants of the type polymer bitumen membrane (PBM) had a much lower yielding stress that the brittle shear strength of the liquid synthetic sealants (LSS), the former - PBM - is probably more suitable for timber bridges because it can better accommodate the large deformations which may occur between the surfacing and the bridge deck of timber bridges.

Finally, in the adhesion tensile tests, all the test specimens fulfilled the strength requirements of the standard SN 640450a:2009.

5 Acknowledgement

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Timber Trusses: State of Art and Challenges

Jose L. Fernandez-Cabo [1], Robert Widmann,[2], Miguel C. Fernandez-Cabo [3]

Summary

The truss is a structural type which is often used for timber structures, either for bridges or buildings. This is due to the fact that trusses, when they are properly designed, are simple and efficient structures. Nevertheless, timber trusses present specific and important difficulties in attaining an efficient connection system. The design of timber trusses depends completely on their joints. Due to the anisotropy of the material, the incorporation of other materials (e.g. steel, concrete and others) is absolutely needed in long spans. This can lead to additional problems, e.g., such as those derived from the hygroscopicity of wood. Connections are therefore the bottle neck. This paper compiles a state of the art report on the conceptual design of the connections of timber trusses. After some reflexions about the problems which arise when designing a timber truss, a review of the different connection systems is presented.

Key words: timber trusses, state of art.

1 Introduction

This article limits the term truss to parallel-chord truss. Small variations of parallel-chord type are of course allowed, as a small slope at one or the two chords.

As it is implicitly assumed in the title of the paper, its core is the case study. But before that, it was considered necessary to compile firstly any basic principles for the layout optimization of trusses (for any material), and secondly to describe additional constraints for the case of timber trusses. The case study is presented afterwards, using a chronological order, and with special emphasis in the building examples of the last decades. Discussion and conclusions are finally presented.

2 Conceptual basis

2.1 General principles for tracing an efficient layout

Layout optimization is a mature and wide field. The approach here summarized follows the path laid down by Maxwell (1890) [98] and Michell (1904) [100] and widened by Prof. Ricardo Aroca [5][6][7]).

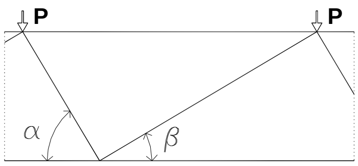

Figure 1: Layout definition of a (parallel-chord) truss.

Assuming that depth is fixed (notice that the strength to shear and bending moments is uncoupled between diagonals and chords), the condition for an optimum layout of the web is obtained when (buckling is not considered):

$$\left(1/\tan^2\alpha + 1/\tan^2\beta\right) = 2 \quad (1)$$

The particular cases with $\alpha = \beta$ and $\alpha = 90°$ lead to a values of $\alpha = \beta = 45°$ and $\alpha = 90°$ and $\beta = 35,26°$ respectively. The case of $\alpha = \beta = 60°$, a Warren type truss, is almost so efficient as $\alpha = \beta = 45°$; and better when buckling is considered, and it is the simplest layout.

The key parameter to control the quantity of material and stiffness of the truss is the ratio between the span L and the dept; d, termed here slenderness: $\lambda = L / d$. The slenderness λ of the truss is usually determined for getting and adequate stiffness. This is typical for the roofs, and it is also a fundamental parameter for the case of

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bridges, where dynamical behaviour would be the critical requirement [68]. A complete treatment about λ is beyond of the scope of this paper.

2.2 Additional constraints when using timber

The first important question is the influence of the slip at the connection. The typical way for including this variable, in a wire frame analysis model, is modifying the stiffness of the web members for including the slip at the connections; i.e. to use an equivalent stiffness:

$$(1/k_{eq}) = (1/k_{con,i} + 1/k_m + 1/k_{con,f}) . \tag{2}$$

where k_{eq} is the equivalent stiffness of the member, including the slip at the connection, $k_{con,i}$ and $k_{con,f}$ are the stiffness of the connection at both ends of the member, using the subindex (i) for initial and (f) for final; and k_m is the stiffness of the timber member itself.

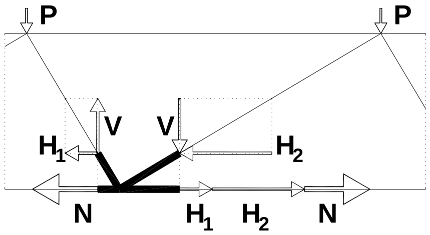

Figure 2: Equilibruim of internal forces at a type node of a (parallel-chord) truss.

The second question is derived from how the shear force at the diagonals is transferred. Figure 2 shows the equilibrium of internal forces at a type node. A node without load has been chosen for the sake of clarity. As it is well known, a key problem when designing a timber trusses arises from the way of equilibrating the two vertical shear forces V in the physical structure. When those forces are equilibrated through the chord, splitting at that chord can occur due to tension perpendicular to the grain. Additionally, or alternatively, a decrease in strength for compression perpendicular to the grain would also exist.

The third important question, as it was commented, is linked with geometrical conditions derived from the connections itself.

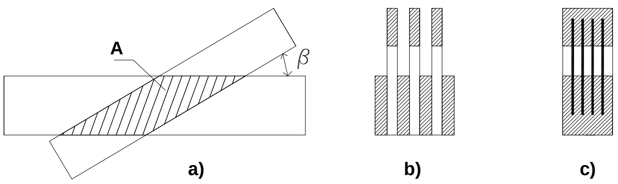

Figure 3: a) Typical connection between a web element and a chord; b) transversal set up with multiple pieces, c) transversal set up with just one piece for each element but adding slotted-in plates or gusset plates.

The strength of the connection is bound by the total contact area A (Figure 3) between the web and the chords. Once the depth of the web and chords are fixed, the only way to increase the strength is to increase the number on contact planes via adding new diagonals (Figure 3b) or adding gusset or slotted in plates (Figure 3c). (see [45]for additional information).

Last but not least, there is another geometrical question of vital importance in the conceptual design of the connection. The maximum strength supplied by each different fastener is function of the material properties of the timber and the fastener itself and a set of geometrical properties.

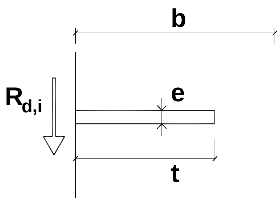

Figure 4: Shear capacity $R_{d,i}$ supplied by a dowel function of its diameter e and the anchor length t.

Material properties and boundary conditions can be considered fixed variables in this argument, which focuses the problem in terms just of length of the dowel t and its diameter e (Figure 4). Current German standard DIN 1052 (2008) uses the concept of required anchor length, t_{req}. (see DIN 1052, 2008, Section 12 [41]; see also [45] and [111], p. 232). Beyond this length, no profit is obtained. And therefore, for an optimum design, the width of timber b is linked with the specific diameter e.

3 Case study

The different cases are collected in several groups according their technology. It will be clear how all the focus has to be placed at the connection system. This state of art report would be completed adding other sources present in other works: [49][53][54][57][24][59][106][107][22][32][60][104][119][128][71][9][103][138].

3.1 Traditional carpentry (from Romans to the end of XVIII)

The whole European timber culture has a Roman root [96]. The arch was the key type used by Romans and it is the basis for the majority of the traditional structures. The truss was also known, but in the case of timber brigdes was basically used as secondary scheme, e.g. for horizontal bracing (a good example is the well-known

Grubenmann´s work at Schaffhausen, built in 1757). See [10][34][30][73][74][81][92][110] for additional information about that period.

3.2 Introduction of machine-sawn timber, manufactured nails and bolts, cast iron shoes and wrought iron rods: novelties of the XIX century.

The use of metal tie-bars in construction is extremely ancient. But due to the high cost, the use of iron can be dated from the XVIII century. Cast iron, used at detailing, was able to bear compressions; and after wrought iron was able to bear also tension. Bessemer´s converters made possible the production of cheap steel at 1850´s, which relegate the use of timber for bridges except in areas where timber was cheaper. Machine-sawn timber was able at the end of XVIII in Europe; and it was extended mainly during the XIX century. Three types of trusses exemplify this period: Town, Howe and Pratt.

Town truss [133] had the novelty of using small sections, easy to handle; and the simplicity of the joints: oaken treenails (call it also “trunnels”). Fracture perpendicular to the grain at the treenails is documented, and clearly is a weak point for this system [34][73][74]. Even though USA was pioneer in the use of timber-lattice trusses, the recent work of Bill [19][20] has shown that they were also used in UK for railways between 1835 to 1870 with spans usually between 15 and 40m.

Howe truss was an important breakthrough for the timber truss. With timber at the chords and diagonals, wrought-iron rods were used for the verticals, which could readily be tightened us as necessary. Joint system is stronger, and the prestress of iron rods reduce the gaps and induces a camber. Nevertheless, authors closer that time were not so enthusiastic, and problems with deflection are an unquestionable fact [56][34][55]. The second and improved Howe´s patent dates from 1846 [74]. His original design is now improved adding and arch, which is basically the same idea present at the Burr´s arches [80]. That proposal recognizes the problems presented at originally Howe trusses. Other possibility is the combination with the strutted arch, as in the Ruseintobel bridge, Switzerland.

Pratt design (patented in 1844 and extended in 1850s), reveres Howe´s principle, using vertical wrought-iron rods and diagonal compresion struts (set originally in St. Andrews cross). It was considered cheaper that the Howe truss but less rigid ([74], pg. 88). Its use in New South Wales is really interesting, where this type was used from 1861 to 1903 [50], see also [27][4].

The Colossus bridge, designed by Lewis Wernwag [110], an early use of wrought iron rods in a long span arch-truss, deserve special attention. It remember recent Cruciani´s trussed arches built as scaffoldings of concrete arches [2][3][26].

See also [61][73][48][99][115] for additional information.

3.3 First timber engineering materials: glulam and plywood.

Glulam technology was developed in 1860´s in Germany, but it was mainly used at the beginning of XIX century (the patent of Otto Hetzer dates from 1906). First patents related with a production of glued veneers date from the end of XVIII. The first industrial production of plywood is also from the 1860´s. It is application as local reinforcement in trusses is nevertheless made at the beginning of the XX century, as it is reflected in Kersten´s manual of 1926 [77].

3.4 First steel mechanical connections and local steel reinforcements: end of XIX century to 20´s of the XX century.

First steel mechanical connections and local steel reinforcements began its development at the end of XIX. But it was mainly after the First World War where they were quickly developed. Germany, USA and Nordic Countries were leading that technology. It was a fundamental breakthrough. Timber technology of the 20´s excels. Kersten´s manual [77] is an amazing prove of that.

The use of gusset steel plates for local reinforcement of trusses is also first used then; e.g. the Maier-Leibnitz system developed in 1918. This technology will be nevertheless really used from the 40´s. By other hand, traditional schemes of trusses are revisited using new mechanical connections, and those improved solutions are present until aprox. the 80s of the XX century. Trusses appear also at secondary elements, as in the Sthephan´s system; e.g. at the arches of the Copenhagen train station (1913), of 19.15m span.

Germany had a crucial role in the development of the timber technology after the First World War. The first German timber Standard related with timber is from 1926, for train stations. The first German Standard for timber bridges if from 1930 (DIN 1074), and the first German Standard in the field of building structures is from 1933 (DIN 1052)[128]. See also [21][127].

3.5 From 30´s to 60´s: basically in standby

The technology of the 20´s was basically the same used until the 60´s or even the 70´s. See [70] for the state of art at connections in the 1970´s. The use of gusset steel plates becomes now a more common practice from the end of 40s. See also [36][37][38][39][131].

3.6 Lateral gusset steel plates with large diameter pins (from end of 70´s)

As it was commented, the use of gusset steel plates dates from the beginning of the XX century. Nevertheless, its use in long span trusses, with new technology, dates from aprox. the end of 70´s.

Those first long span trusses placed the gusset plates at the timber faces, laterally, not into slits. The use of large diameter pins releases secondary bending moments. This solution has been used for short, medium and long span trusses, obviously with variants.

Good examples of that technique are: the bridge over the Neckar in Stuttgart built in 1977 with two independent spans of 72 and 64,75m [104]; the footbridge over the Isar in Munich, built in 1978, with a central span of 52m[107]; the bridge over the Simme at Wimmis (1989), Switzerland, with a central span of 54m and side fields of 27m [51][105][71][104]. Of high interest, for a 23m span, in the proposal proposal made by J. Natterer for bridge in a competition for the EXPO 2000 at Hannover, in 1997 [109]. The primary structural system of the pedestrian bridge consists of a two-grid truss. The compression diagonals consist altogether of four boards and the tension diagonals of two. Under and upper chords are made of glue laminated timber. The chords are symmetrically attached to both sides of the grid planks. It is reinterpretation of a Town truss. The system was previously use in the horizontal bracing for the footbridge across the Main-Donau Canal near Essing [40], Germany.

3.7 Slotted-in steel plates in long spans (from 90´s)

The use big sections made of glulam and joined by slotted-in steel plates with dowels is now an extended practice all over the world. Most of the current the long span timber bridges have used this technology. Steel plates avoid any splitting, and the use of multiple plates supplies enough shear planes for transferring the whole internal forces.

The footbridge bridge over the river Neckar at Remseck (Germany), built at 1988-89, a three-dimensional truss spanning 80m, uses glulam section with a steel plate let in a slit [71] [104]. Only one plate is used. The use of multiple plates offers a bigger efficiency, and it has been first developed in Norway. Evestand bridge crosses Glomma river in Hedmark [82][83] open this type. It was built in 1996 using five simply supported trusses aprox. 36m free span each (hibrid bewteen bowstring and parallel-chord types) with a Warren-type set up for the diagonals, adding steel hangers for shortening the free span of the deck. The deck is continuous and formed by planks with a transversal prestressing. Glulam is used, with a creosote treatment. Crossbeams of steel are located underneath the timber deck. The joints are made using four parallel slotted-in steel plates. Flisa bridge (2003) [42][43] uses the same technolgy. It has a total length of 196m and a maximum free span of 70.34m. Kjollsawter bridge over the Rena river (2005) has a total length of 158m and a maximum free span of 45m. It was designed for supporting a heavy military vehicle convoy; which makes this bridge a remarkable case study [1]. See also [69][130][62][137].

It must be notice that the dynamical behaviour of structures with these solutions deserve additional studies [117].

3.8 Slotted-in steel plates in medium spans

For medium spans it is possible to use thinner steel plates. This offers an important advantage, as the plate can be perforated by the dowel when it is placed, and this reduce the gaps. Examples of that are the Greim® system, that uses nails with plates between 1 and 1,75mm thick; and the Paslode® system, that uses plates between 2 to 3mm thick. Both would be used for short and medium spans [101]. In the same line, the BSB® sytem (Blumer-System-Bau) dowell steel to timber joints with slim high strength drift-bolts (see e.g. Long-Span Pedestrian Timber Truss Bridge Across the Dahme River at Berlin 60m span [23][95]).

3.9 LVL with gusset steel plates

It has been researched in Europe mainly by the company *Finnforest*, the producer of Kerto® [75][79]. The idea is built like a sandwich with Kerto® layers spaced with plywood, and using steel gusset plates at the nodes in the space of the plywood. A recent example of this technique, combined with Pfeiffer® steel rods at the diagonals, it is the cantilever of 19.2m span built in Vitoria, Spain [46][47]. It is made of 3 Kerto 69mm layers (5 at vertical struts, of Q type the most cases, but also any S type) and 15mm plywood between Kerto® layers (or steel plates at the joints). There are two main advantages in using LVL for trusses. One is the high strength of the material. The other one, even more important, is the possibility of using LVL with cross veneers (Kerto® Q type), that works as a transversal reinforcement, reducing the possibility of splitting caused by the dowels or by hydrothermal changes.

3.10 Punched nailed plates for short and medium spans.

The use of punched nailed plates is now an extended practice for prefab trusses of short spans. For medium spans, the Multi-Krallen-Dübel (MKD®) system is available. It is a sort of punched nail plate, but nails are disposed bilaterally to a steel plate, and therefore multiple sections can be joined, which is not the case for the typical punched nailed plates [101]. A variation of that type, using steel punched nailed plates as local steel reinforcement adding medium diameter pins, is described in [78][12][13][121].

3.11 The use of large diameter pins and densified veneer boards

The concept is again similar to the previous one, but the use of timber boards offers advantages for fire resistance. See [89][90][63][64] for additional details. It is practical use is now limited to short spans.

3.12 Current use of steel for solving the whole node

As it was commented, this working line was initiated at the 20´s. Some current examples are the next ones: the space-frame bridge across the Isar at Thalkirche, Munich, (1991) [40] (that uses a MERO system); the footbridge over the Isar in Munich, close to St Emmeram (2003) [40] (using specially-developed cast steel joints); or the type bridge in Kössen, Austria [44], 50m span. The use of special steel connection for roofs is quite extended; including prefab solutions (see e.g. [31]).

3.13 Introducing reinforced concrete at nodes

The use in concrete at the nodes of timber trusses is quite recent. A remarkable example is the footbridge at Sindelfingen [129], made of glue laminated timber with the nodes made of reinforced concrete, with two spans of 38.4 and 24.03m. See also [16] for current studies of concrete nodes at timber structures.

3.14 Revisiting traditional types with new technology

The use of steel at the tensioned members was proposed at XIX century. It is still a wise combination. An example is the space V-shape the Exhibition Hall 7A in Nürnberg, a simple supported structure of 85m span. [120]. A similar structure, but with a 2D arrangement, was built for the Exhibition Hall number 4A in Nürnberg, with a 60m free span [126]. A remarkable example of a hybrid steel-timber bridge is the Forst bridge over the Klinalkini in Canada, of 63m span [60]. A current variation of Pratt-type truss [124] hides the vertical tension bar inside a timber element using the *Induo*® System [72]. Bertsche connection system® [18] allows also tension joints with high capacity. It uses a transversal reinforcement with dowels (see also [114] fot that type of solutions). A Pratt footbridge truss-type bridge built in Sankt Pölten, Niederösterreich, over the river Traisen, has a total length of 104m [94], a continuous structure with two bays of 50.7 and 39m span respectively, and a depth of 3.9m. The brigde at Ravine, built at 1989, of 36m span, propose and interesting solution at the node joining the diagonal at compression with part of the compression chord to simplify the connection [108]. A project of 2001 revisited the Howe-type truss design [93][26]; the footbridge 31m span uses the Howe´s layout with two improvements: optimization of the layouts of the diagonals and use of glue (resorcines in particular) for joining the timber shear block with the chords. See also [95] for the use of thrust blocks in trusses. The use of CNC technology is helping to recover traditional connections with rational costs. The bridge at Eschenlohe [40] is made with truss-type scheme, rooted in traditional schemes, a triple queen post structure. It is a hybrid structure, made of timber (glulam made of pine wood). The timber trusses for the roof of Velodromo of Sangalhos (2009), Portugal, with a maximum span of 78.6m, is a good example of how to arrange the pieces using a classical dowel connection system without using slotted-in steel plates [102]. The reinforcement of the nodes using plywood board has now been also recuperated for short spans and an industrialized production [25].

3.15 Joining with glue technology

One working line using glue was the use of fibreglass mats as reinforcement of the joint, e.g. at the bridge built in 1999 over the Simme in Garstadt, Switzerland [52], of 24,5m span. See also [28][29][87][65][66][67][116] and [123]. The use of glued-in rods is now a very active topic [134], and it was proposed in particular for the joint of timber trusess [58]. In France, the *Carpenton* project explored the use of glue-in rods (with epoxy resins) for joining bars of a truss; oriented basically for short span trusses [8]. The use of V-shape glued-in rods (anchored at 45º) has a wide practice in Finland [76]. The use of steel glued plates has been also recently proposed [135][136][15].

3.16 Transversal self-tapping screws with large diameter pins.

Self-tapping screws have been proved to be an efficient and simple solution for the transversal reinforcement of timber joints, i.e. to avoid splitting [11][17]. Recent research is been expanding this working line, for short and medium timber trusses, adding large diameter pins [33][84][85].

3.17 Use of CLT

CLT has a current important application. Its use in trusses is recent, e.g. at a train station in Zeltweg (Steiermark, Austria), built in 2013 [118], with a central span of aprox. 44m and a Pratt truss-type design. Diagonals are made using steel bands, and struts with CH steel profiles.

3.18 New concepts in the preliminary design of joints

The use of self-tapping screws at aprox. 45º angle with the plane of the joint has opened new solutions combining high strength with high stiffness (higher that with the typical dowel connections) [14][17][97]. Plasticity can be increased combining screws with angles at 45º and 90º [132]. See [112] for studies about optimization. Once recent remarkable example of this technology is the truss for the roof of the Exhibition hall number 11 in Frankfurt a. M. It is a hybrid timber steel Pratt-type truss, with diagonals in tension made of steel rods and struts and chords made of glulam. The trusses have a free span of 78m and two cantilevers of 17.4 m. A vertical steel plate transfers directly the vertical shear force V, and the horizontal shear force H is transferred to the chords by self-tapping screws at 45 degrees with the horizontal. See also the ideas presented in [122][125].

3.19 Prestressed solutions

The use of prestress tendons or bars in timber structures has been already explored. The Mursteg footbridge at Murau (Austria), 48m span, built in 1995, was a pioneering work. Other remarkable examples are the Radwanderweg bridge between Gaiβau and Rheineck, a simple supported Vierendeel postensioned bridge [91]; and the structure for Seminar in building made in 2004 [113], a post-tensioned truss of 35m span.

4 Conclusion and future work

Basic principles for the optimization of trusses were initially presented. An efficient design is controlled for the layout of the diagonals and a proper slenderness. The way of transfer the shear forces at the joint, the stiffness of the connection, the geometrical arrangement of the members and the slenderness of the dowels are also key aspects in the conceptual design of trusses.

Timber trusses are not typical in the historical carpentry of Europe, which is governed by the Roman tradition. The arch type is prevalent. The truss layout sometimes appears complementing the arch type, or bracing of the deck. Important novelties of the XIX are the introduction of machine-sawn timber, the manufacturing of nails and bolts, cast and wrought iron and finally steel (in industrial production). Traditional carpentry is revisited then. Town, Howe and Pratt types exemplify that period. Only medium spans were possible with these systems, and only with Howe and Pratt types. Plywood is used as local reinforcement for timber trusses from the beginning of the XX century. The first mechanical connections (dowels, toothed rings, etc.) are developed at the beginning of XX century; especially in the period between the two World Wars. Gusset steel plates for the reinforcement of the nodes have there also the first examples, even they are mainly used from the 40´s and 50´s. Traditional carpentry is again revisited on basis of the new technology. The achievements in timber technology up to the 30´s is really amazing, especially in Germany and USA. First timbers standards, in Germany, are from that time. Mechanical connections opened hugely the variety of solutions for short and medium spans; and allow more efficient solutions. After a period of stand-by between the 30´s and the 60´s, first long spans are developed from aprox. 70´s. Lateral gusset steel plates with large diameter pins were first used. Slotted-in steel plates were developed after, aprox. from the 90´s. In these cases, dowels are placed perpendicular to the grain. Dynamical behaviour of those systems seems to deserve additional research. The last decades have expanded the possibilities for solving the joint: new punched steel plates; reinforcement with self-tapping screws; new cast steel able to resist tension; revisiting of traditional connections using CNC; use of glue technology using glue-in rods, gluing gusset plates or glass fibre mats; use of self-tapping screws at 45º for increasing the stiffness of the connection; use of new materials as CLT and LVL. Prestress is been also applied to timber trusses.

In the authors opinion, there is need for clarifying the role of the different systems, i.e. which is their range of span and their advantages and disadvantages. The selection of the connection system is only basically restrained for long spans; and even then there are many possibilities. The existing technical information is huge, but its transfer to the practice is no so high. A bigger link between research and practice seems desirable.

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Pre-tensioned timber-truss pedestrian bridges An innovative bio-hybrid design approach using round-timber

Kay-Uwe Schober[1], *Michael Kuechler*[2]

Summary

This paper presents innovative design approaches for timber truss bridges with round-timber as building material. Truss structures with round-timber reside in some problems in design of the assemblage points. Therefore, innovative design approaches of the joints are presented in this paper using new-jointing techniques based on post hoc grouting, modular construction principals and pre-stressing screw systems to ensure a deconstruction.

Key words: timber-truss bridges, round-timber, concrete-type adhesive, UHPC, eco-efficient, bonded-in rods

1 Introduction

This paper deals with an innovative design approach of timber truss pedestrian bridges for federal motorways in Germany with a maximum length of 15 m and a usable cross-section of ca. 3.8 m. The construction of timber bridges in Germany are very unpopular due to several disadvantages as well as the prepossession of German structural engineers, who deal in general with concrete-, steel- or composite steel and concrete structures. The disadvantages of conventional pedestrian bridges can be summarized with the following keywords:

a) High self-weight lead to high costs for the foundation as well as the sub-construction

b) Negative eco-balance and deficits in sustainability caused by complex structural systems

c) High costs resulting from extensive and time-consuming bridge controlling activities / actions

To neglect the disadvantages of conventional pedestrian bridges as well as fulfilling the increasing demand of an economic and ecologic design of such bridges, new design concepts have to be developed under combining eco-efficient materials like round-timber with high-performance materials like ultra-high-performance concrete. A further important aspect in bridge engineering is the modular construction principal of bridges to ensure a partial demolition as well as a local rehabilitation (maintenance).

In this paper, the conceptual design of the innovative timber truss bridges is characterized through using eco-efficient round-timber as structural element for force transmission, pre-fabricated assemblage points that represents the actual link between the round girders and diagonal struts and the post hoc grouting of the single elements towards a stiff and rigid connection. Regarding the connection of the single elements, two solutions have been analyzed. The first is characterized through a mineral cast and bonding of the joint elements to obtain a stiff connection. The second solution is achieved by pre-fabricated self-pre-stressing structural plane elements made of ultra-high-performance concrete (UHPC) (Figure 1b).

a) Casted and bonded rigid assemblage point **b) Modular self-pre-stressing assemblage point**

Figure 1: Two schematic visualizations of the assemblage points

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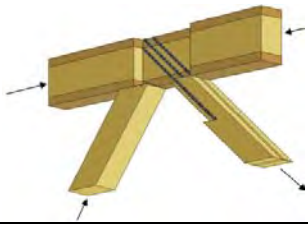
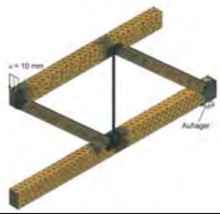
2 Conventional joints for timber truss bridges

The advantages of conventional timber truss structures and their assemblage points can be summarized to:

a) Aesthetics of transparency girders by use of eco-efficient round-timber
b) Light-weight construction for large spans
c) Minor material consumption

The design and construction of such timber trusses find an increasing application worldwide in Timber Bridge Engineering although there are several disadvantages as shown in Table 1.

Table 1: Advantages and disadvantages of several truss structures

Nail-plate connection	Dowel-type connection	Innovative truss – KIT [1]	Timber-concrete truss [2]
			
– Limited span length – Bad Aesthetics – Fire behavior + Quick and cheap assembling	– Assembling-effort – Weakening cross-section – Local force application + Ductile failure behavior + Good fire behavior	– Secondary bending moments + Good Aesthetics + Hybrid glulam + Good fire behavior	– Weakening cross-section without sufficient bond between timber and concrete – Fire behavior

Nevertheless, the design of timber truss bridges with conventional rectangular cross-sections made of solid wood and glulam is very common in Europe, except using round-timber as structural elements due to geometrical restrictions when connecting round elements to each other. Three impressive round-timber bridges are illustrated and described more in detail in Table 2.

Table 2: Advantages and disadvantages of several truss structures

Parameter	Passerelle du Borgeaud Bovernier, Switzerland	Timber Bridge Oberföhring Munich, Germany	Talkirchner Bridge Munich, Germany
			
Construction-type	Open Bridge	Parallel-chord truss	Space framework
Length [m]	28.0	96.0	197.0 / 13 x 13.4
Width [m]	2.0	3.5	13.0
Rode surface	Longitudinal boards	Timber deck	Asphalt
Miscellaneous	Handrail made of round-timber	Framework as steel portal, bracing with steel cables	Passable, Cast steel assemblage points

All of these three different round-timber bridges have one essential aspect in common. The assemblage points were well designed to ensure a safe and robust supporting framework. The bridge in Bovernier, Switzerland was constructed conventionally by carpenters. In the parallel-chord truss bridge in Oberföhring, Germany, the assemblage points were made of sectional steel parts to ensure the load transmission between the round-timber structural elements. The imposing space framework bridge in Munich, Germany was assembled by pre-fabricated cast-steel assemblage points, which were in turn connected to the round-timber via high-strength bolts.

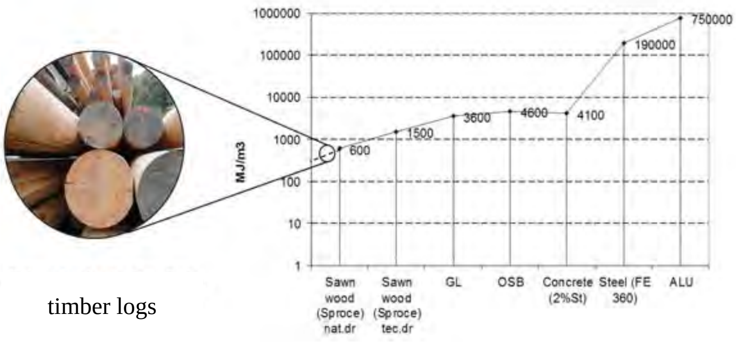
The problems of the abovementioned connection-types are the heavy cross-sectional weakening of timber by the carpenter’s work, the negative eco-balance of steel as well as the high costs, assembling effort and structural behavior under fire loading. Furthermore, the exposed steel elements are susceptible against corrosion and aggressive atmosphere and the crosscut wood is not prevented against penetrating moisture. In consequence of these aspects, the investigations of other joint solutions try to neglect the disadvantages of conventional timber truss structures made of round-timber.

3 Materials

Timber logs and round-timber as structural material

Round-timber is the key element in the truss structure for load transmission. Here, the eco-efficient material is applied for the girders as well as the diagonal struts. Figure 2 show the eco-balance of different materials. It is conspicuous; designing structures with round-timber is very ecological and economical. Further advantages result from the utilization of timber logs on the construction site, the low process energy for fabrication and the 20% higher stiffness and strength values in comparison to other timber products and engineered wood products.

Eco-balance of different structural materials



Material properties of round-timber (C24)

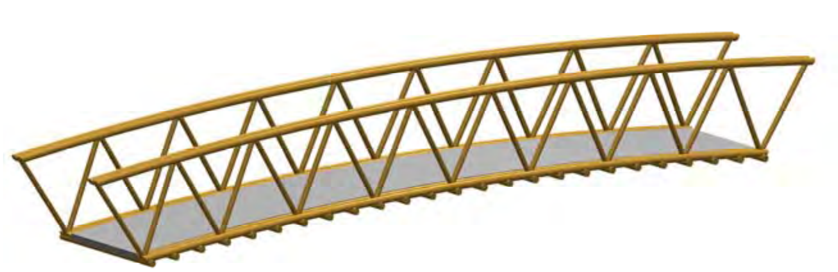
Bending strength	$f_{m,k} = 24.0$ MPa
Tension strength	$f_{t,0,k} = 14.0$ MPa $f_{t,90,k} = 0.4$ MPa
Compression strength	$f_{c,0,k} = 21.0$ MPa $f_{c,90,k} = 2.5$ MPa
Shear strength	$f_{v,k} = 4.0$ MPa
Modulus of Elasticity	$E_{0,\mathrm{mean}} = 11000$ MPa $E_{90,\mathrm{mean}} = 370$ MPa
Shear Modulus	$G_{\mathrm{mean}} = 500$ MPa

Figure 2: Eco-balance and material properties of round-timber

The effect of stiffness and strength enhancement is caused by surrounding sustainable, tightly ringed sap wood, which is not damaged during the manufacturing process than in comparison to sawn wood. Nevertheless, this stiffness enhancement will not be applied in the structural analysis because the increasing factors are highly controversial. Here, the mechanical properties of solid wood C24 apply [3]. In the construction of the timber truss bridge, the cross-section of round-timber will be equated assembled to obtain a structure appropriate for the material involved and to accommodate all the internal forces.

For the preliminary version of the bridge design a maximum diameter of round-timber of $d_{RT} = 240$ mm was chosen, resulting in a maximum clear span of the bridge of $l = 15.0$ m for a double-sided parallel-chord truss structure (Figure 3).

Truss system – longitudinal section



Truss system – cross section

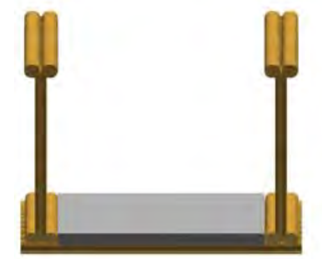


Figure 3: Visualization of the preliminary version of bridge design

Concrete-type adhesive (CTA) as connecting material in joints

The intended purpose using CTA in this construction is to bond the pre-fabricated assemblage points with the round-timber. Here, it is expected to obtain a stiff and rigid connection between both structural elements. Through the big experience in timber engineering using epoxy resin as well as further adhesives for fabricating glued laminated timber, finger joints and glued-in rods for instance, this joining technology is controllable.

Commercial available products have been modified to meet the requirements for the new-type joints. Furthermore, the application process for glued-in mechanical fasteners has been modified to obtain a better quality control and load-carrying behavior for on-site assemblage using a much larger bond line than in common glued-in connections [4], [5], [6].

The proposed CTA is a polymer-bound concrete, formed of two-component liquid epoxy resin with a mineral aggregate. The resin for the production of the CTA is free of solvent, stable crystallizing low-molecular weight epoxy resin based on Bisphenol-A Epichlorhydrin. The hardener adapted for the product is a fluid, colorless to light yellow at room temperature polyamine adduct, which holds an average reactivity for the interlacing of the fluid resin. The mineral additive is composed of well-graded gravel with a grain size of max. 6 mm. All components were mixed together in a special ratio by weight. Here, a detailed characterization of the material properties is presented, which will be used for future numerical analysis (Table 3).

A further positive aspect of using CTA for casting construction elements in situ is the low cure time in comparison to high-performance concrete (HPC) like HPC100/115. Here, the stiffness parameters of CTA reach a comparable level after a quarter of the curing time. This positive effect facilitates a fast and efficient assembling of the timber truss bridge in situ (Table 4).

Table 3: Material properties of CTA

Material property	Value	Unit
Density	1.98	[g/m³]
Modulus of Elasticity	22,000	[MPa]
Shear Modulus	9,300	[MPa]
Bending Strength	30	[MPa]
Compressive Strength – Cube	116	[MPa]
Compressive Strength – Prism	98.2	[MPa]

Table 4: Material properties of CTA compared to HPC

Parameter	Unit	HPC 100/115	CTA
Cure time	[d]	28	7
Compressive Strength	[MPa]	115	116
Bending Strength	[MPa]	11*	30

* The bending strenght result from $f_{ct,flex} = 2 \cdot f_{ct}$ with $f_{ct} = 5.5\,\text{MPa}$ [4]

The high compressive strength of CTA results from the bonding behavior of the polymer binder material and especially the mineral fillers, which leads to a high packing density. Ideal-elastic stress-strain relationship behavior with semi-ductile hardening is observed (Figure 4).

The failure behavior under consideration of tension or shear loading performs in a more brittle way compared to pressure loading. Due to the large amount of gravel in this composite material it is possible to assimilate a large amount of concrete, e.g. for much bigger drill holes compared to conventional drill hole diameters when dealing with glued-in rods, without getting in trouble with exothermic chemical reactions like in higher-content resin and curing agent adhesives compositions.

Stress-strain graph – testing MoE

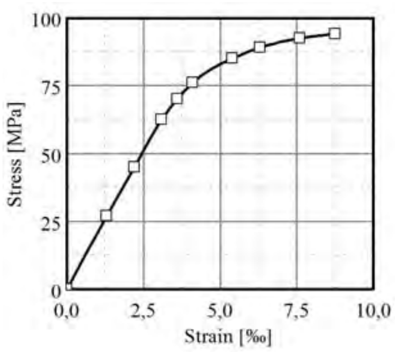


Figure 4: Stress-strain curve from lab tests

Ultra-high performance concrete (UHPC) as pre-fabricated joint

The intended purpose using UHPC in this construction is to form a pre-fabricated joint free from geometrical restrictions when connecting three or more round-shape members together. Ultra-high-performance concrete exhibits compressive strength values till 200 MPa, centric tensile strength till 15 MPa and bending strengths till a maximum of 50 MPa. These high strength values are realized through extremely low cement / water ratio of ca. 0.2 and a high packing density. The high packing density leads to a concrete with approximately no capillary pores resulting in a high resistance against gas, fluids and corrosion. The brittle failure of UHPC can be counteracted by admixing steel fibers resulting in a nearly quasi-ductile load bearing behavior, enhancement of fire behavior as well as minimization of the creeping and shrinkage behavior. In general, the base materials are silica fume, granulated blast-furnace slag, cement, inert additional materials and flow improvers [7].

4 Bridge design

The design of the round-timber truss bridge is characterized by a curved parallel-chord tray system with a continuous road surface placed on the bottom chord. To benefit from the chosen design of the truss bridge, the structural system, distribution of forces within the assembling points and the most restrictive parameters in design codes have to be considered carefully. Furthermore, there have to be innovative designs of the assemblage points, which are in unison with the contribution of forces and design parameters.

For this design approach a very special design feature is the modular construction principal between girders, braces and assembling points as well as the utilization of multi-material solutions and composite constructions, consisting of three essential members:

a) Round-timber as structural material
b) Precast assemblage points made of UHPC
c) Connection with modular bonded-in rods (BIR) / UHPC plates, glued-in by CTA

To design a modular construction kit for varying spans, the angles of the assemblage points have to be constraint to ensure non-variable dimensions of the assemblage points as well as of the single elements of the girders and diagonal struts (Figure 5).

Static model of the truss bridge – 15.0 m

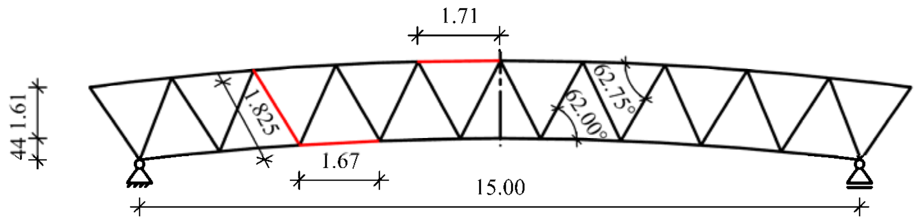


Figure 5: Dimension values of the round-timber truss bridge

The critical designing parts are proof of stability, stress design of the assemblage points as well as mechanical fasteners between the single components of the single elements. This paper will focus on the mechanical description of two variants of the assemblage points (see Chapter 5).

Regarding the durability of the tray system, a roof can be constructed on the top chord to ensure constructive wood preservation. The crosscut round-timber can be impregnated with epoxy resin to resist outside influences like wood-destroying fungi, insects and water damage. The assemblage point as well as the connection between the diagonal struts and the assemblage point can be characterized as durable due to the high packing density of CTA / UHPC. Here, a water penetration along the bond line is not realistic. Furthermore, the round-timber is improved along and around the bond line by the penetrating resin-components of the concrete-type adhesive.

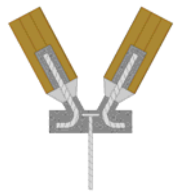
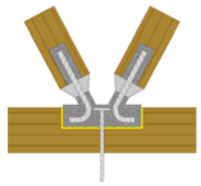
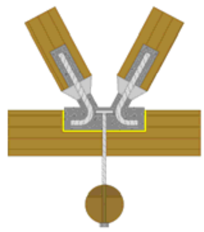
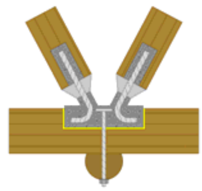
5 Mechanical model of the assemblage points

Casted and bonded rigid assemblage point

The mechanical characterization of the casted and bonded rigid assemblage joint is described by three essential load-carrying elements. The first fundamental element is the assemblage point made of UHPC. This structural element is able to transfer high compressive forces into the girder without obtaining a non-linear material behavior. High tensile loads are passed through by BIR, which are adhered with CTA to transmit compressive and tensile forces of the diagonal struts into the assemblage point.

To compensate construction tolerances, a ductile pressure ring is placed between the diagonal struts and the contact area of the assemblage point. Because of this, all occurring compressive forces will be transferred into the assemblage point centrically neglecting any secondary bending moments. In tensile-loaded struts, the centric loading is ensured through bonded-in rods respectively bonded-in plates. The individual working steps are illustrated in the following table.

Table 5: Manufacturing process of the timber truss bridge – Casted and bonded rigid assemblage points

Struts-Assemblage Point	Girder-Assemblage Point	Road Surface	Pre-Stressing
			
Post-hoc grouting of the framework	Bonding of the girders with the framework	Assembling of construction	Pre-tensioning of all single elements

The assembling of the two single truss systems with a road surface towards a well-performing tray system is achieved by connecting the deck with the assemblage points with bonded-in rods respectively self-pre-stressing rods. Hereby the general stability and bracing of the truss bridge in the lateral direction is guaranteed.

Modular self-pre-stressing assemblage point

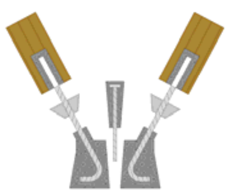
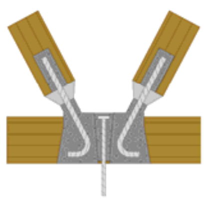
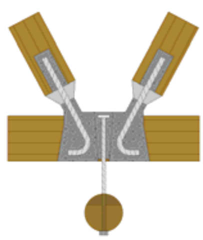
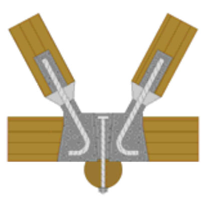
The second assemblage point of the bio-hybrid structure is characterized by precast, modular assembling points made of ultra-high-performance concrete, which enables the assembling points high stiffness and at the same time an articulated connection because of flexible designed glued-in rods [4], [5], [6] as well as modular pre-stressed connection type.

The essential idea of this assemblage point is defined over an optimized manufacturing process, which enables a deconstruction of the single structural elements. Therefore, the assemblage point is based on a push-fit system, which is post hoc pre-stressed by threaded rods. The interaction between the diagonal struts and the assemblage point is achieved through BIR respectively bonded-in plates, which are post-hoc grouted with CTA respectively UHPC.

The anchorage of rods in pre-cast assemblage points made of UHPC can be limited towards very small bond lengths caused through the adhesive tensile strength as well as the very good functioning frictional connection. The bond length within the struts is decisive characterized by shear fracture along the bond line between steel rod and CTA and a bidirectional crack fracture in CTA [9], [10]. For tolerance compensation, ductile pressure rings are placed between the struts and assemblage point. Through this action, an articulated connection with a centric load transmission is guaranteed.

For manufacturing, the start begins by connecting the UHPC-plates with the diagonal struts. To achieve a rigid connection, both elements are grouted by CTA or UHPC. After this, the structural girders are slit and diagonal struts are positioned into the slots. To obtain a tight fit between the three components, a conical joint is placed between the two UHPC-plates and afterwards pre-tensioned. On the bottom side of the girder, the road surface will be positioned and fixed by pre-tensioned rods. The road surface is constructed by across spanning round-timber beams. The actual road surface is supported by the crossbeams.

Table 6: Manufacturing process of the timber truss bridge – Modular self-pre-stressing assemblage point

Struts-Assemblage Point	Girder-Assemblage Point	Road Surface	Pre-Stressing
			
Post hoc grouting of the framework	Assembling of the push-fit system within slots	Assembling of construction	Pre-tensioning of all single elements

6 Results and discussion

In consequence of the beginning of the research project, the modest, essential results can be summarized with the following key sentences:

a) Positive life cycle assessment of the timber truss bridge caused by using yet untapped round-timber as structural elements
b) Guarantee of a stiff and rigid connection between the assemblage points and the girders as well as diagonal struts caused by the post hoc grouting of the single elements with CTA respectively epoxy resin
c) Guarantee of an economical construction caused by using non-variable pre-fabricated structural elements as well as inexpensive round-timber

The key issues of this research project, which have to be analyzed very carefully, are the proof of stability, stress design of the assemblage points and mechanical fasteners. Furthermore, the manufacturing processes as well as the design of the assemblage points have to be optimized to ensure a competition between the innovative truss structures with conventional truss systems.

Further research will deal with the conception of the manufacturing process of the truss structure. Especially the design of a span-varying construction kit for timber truss bridges will lead compulsory to an economic and ecologic design of timber bridges, which will be able to compete against conventional concrete- and steel-bridges or composite steel and concrete structures.

To obtain the bonding behavior under varying load direction, changing MC exposure and acceleration, static and dynamic testing is on-going or in preparation. Future steps are long-term behavior of the composite structure and design guidelines in form of an analytical approach to make this efficient connection accessible for engineering standards.

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Durable Timber Bridges

Kjell A. Malo[1], *Anna W. Ostrycharczyk*[2]

Summary

The present paper describes the DuraTB project, a project dedicated to durable timber bridges. The project aims to improve the general standing and applicability of wood as a structural material in bridges and to contribute to increased use and market shares for environmentally friendly timber bridges. The bridge design concepts to be developed shall be among the best alternatives with respect to environmental friendliness, initial costs and life-cycle costs, and they shall show excellent results in life-cycle analyses. In the latter part of the paper current research on network arch timber bridges performed at NTNU in Trondheim is presented.

Key words: timber bridge, network arch bridge, timber durability

1 Introduction - background

In the last decades it is experienced that most materials used for construction of bridges have limited lifetime. Concrete gets carbonized, steel corrodes and timber may be attacked by insects or fungi. A large number of concrete and steel bridges built after the Second World War was assumed to have little need for maintenance. However, the current state of many of these bridges does not support this assumption. In many cases the bridges are beyond repair. Consequently, the number of bridges in European infrastructure that needs replacement is large.

It is a common perception that the expected lifetime of a timber structure is only a fraction of that of a concrete or steel structure. In spite of this some of timber structures like the Norwegian stave churches are very durable. On the other hand we do have timber structures that show serious decay after only a few years in service due to elevated levels of moisture and consequently growth of fungi and rot. The research aims to significantly improve the general standing and applicability of wood as a structural material in bridges.

The project is a WoodWisdom-Net project and involves an international group of 20 partners from Finland, Norway, Sweden and USA. It is a three-years activity coordinated by NTNU University, Norway. This paper gives an overview of the project. The latter part deals with a design concept for long span durable timber bridges, and is a part of the presented project.

2 Project Objectives and Plan

Bridges for public use are often monumental structures with expected design working life of about 100 years [1] or more. The principle stated in EN 1990 Eurocode Basis of structural design [1] for durability reads *The structure shall be designed such that deterioration over its design working life does not impair the performance of the structure below that intended, having due regard to its environment and the anticipated level of maintenance*. Furthermore it is stated that the environmental conditions shall be identified so their significance can be assessed. Finally, it is also stated that the degree of any deterioration may be estimated on the basis of calculations, experimental investigation and experience from earlier constructions or a combinations of these considerations. However, consulting EN 1995-1-1 Design of timber structures [2] and the specific part on timber bridges EN 1995-2 [3], no method, guidelines or data for evaluation of deterioration are given, only prescriptive rules for improvement are mentioned, like avoiding standing water and the use of preservatives. Consequently timber bridge designers have no measures for estimating lifetime of wooden structures. This is also true for wooden building structures in general, but usually the load-carrying structures in housing are not exposed to climate loading in the same degree as timber bridges.

The overall objective of the project is to develop durable timber bridges with a given estimated technical lifetime.

The scientific and technological objectives cover; development of an exposure model related to risk of decay validated for a given set of timber bridges, development of a performance model relating microclimate representative for wooden bridge elements to the risk of their decay and development of a methodology to account for

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uncertainties in service life design of timber bridges. Furthermore a set of design concepts for durable timber bridges in the span 10 – 150 m will be prepared, including development of splicing technology for massive block-glued glulam cross-sections for bridges, improvement of the performance of stress-laminated wooden deck and details of wooden deck-plates, development of fatigue strength criteria for axial threaded rod (screw) connectors and development of design integrated maintenance concepts for timber bridges. The project deals with quantification of risk of decay related to moisture traps in timber structures by combining data from existing bridges, statistical methods, analytical methods and numerical models. Moreover, quantification of risk for crack development related to moisture distribution in large block-glued glulam members by numerical models are additional ideas to be executed in the project.

Project consists of five main work packages (WP) with related tasks, see Table 1.

Table 1: Project topics divided into work packages and tasks

WP	Task	
1. Coordination	**1.1**	Project management
2. Performance based service life design of timber bridges	**2.1**	Collection of field data from existing instrumented bridges
	2.2	Development of climate exposure model for bridge structures
	2.3	Tests of climate exposure (moisture content, temperature) in structural details
	2.4	Development of suitable dose-response model for fungal decay
	2.5	Methodology for service life design of bridges
3. Hygro-thermal effects in wooden members	**3.1**	Numerical models relating rain, spray, RH, T, to distribute material climate effects in members
	3.2	Moisture distribution, moisture induced stress and risk of cracking in members and connections
4. Design concepts for durable timber bridges	**4.1**	Wooden bridge decks
	4.2	Design concepts for short to medium span bridges
	4.3	Design concepts for medium to long span bridges
	4.4	Splicing of large glulam members
	4.5	Fatigue of axial-carrying connectors in wooden members
	4.6	Performance evaluation of design concepts (structural performance, lifetime, LCC, LCA)
	4.7	Maintenance practices and repair techniques for extending service life of timber bridges.
5. Dissemination	**5.1**	Produce a book or report on design of durable timber bridges
	5.2	Arrange open workshops
	5.3	Prepare proposals to CEN TC 250 SC5 to the new generation of EN 1995-2 Timber bridges
	5.4	Publication scientific papers, journals and conferences

3 Description of each work package

The project is split into 5 work packages. The interrelation between the WPs and flow of information are visualized in Figure 1.

W1. Coordination.

The consortium has 20 partners. In order to be manageable, the partners have been divided into four groups, namely; research organizations (1), road authorities/bridge owners (2), bridge-builders and producers (3), and stakeholders for the wood industry (4), confer Figure 2. The project coordinator and the official contact point for the consortium is NTNU.

WP 2. Performance based service life design of timber bridges

In this WP collection of field data from existing instrumented bridges will be done. Structural details of a number of existing timber bridges have been instrumented for measurements of moisture content development and temperature. The results from these tests will be collected and made available for the project together with information about the bridge and global climate data at each site.

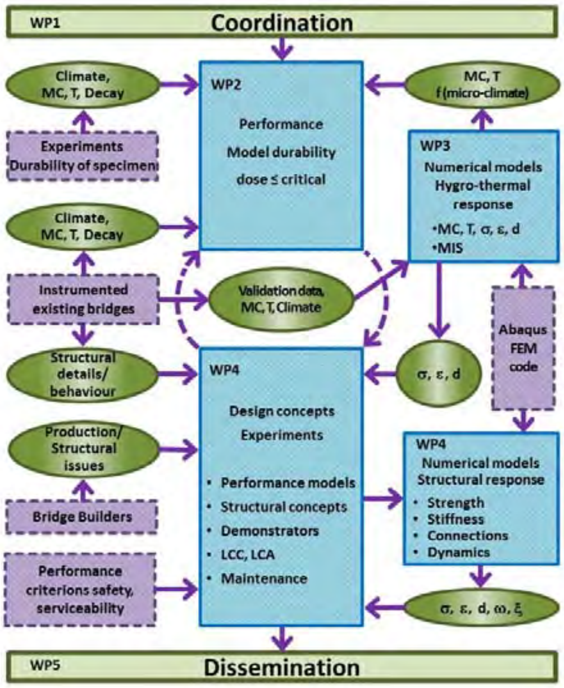

Figure 1: Flow of essential information

The WP covers also development of climate exposure model for bridge structures. In performance based service life design it is important to quantify the microclimatic exposure in wood elements, which is a function of global climate, bridge design, material and surface treatment. A basis are existing models developed for decking and claddings in exterior use class 3 exposure as a part of the WoodWisdom project, Woodexter and the Swedish Vinnova project WoodBuild.

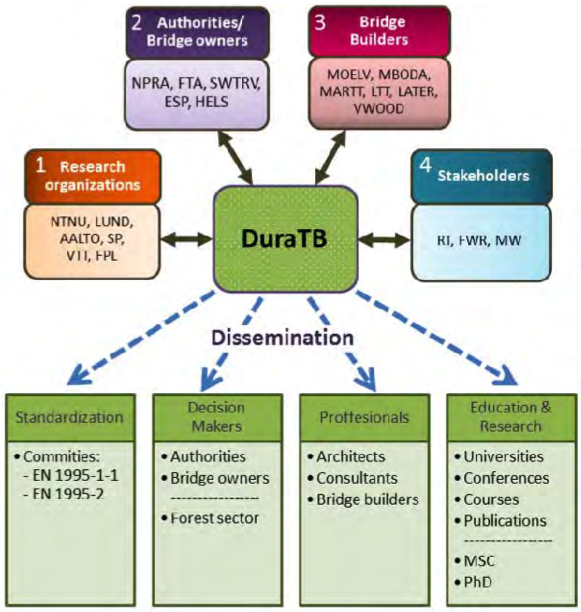

Figure 2: Partner and target groups of the project

One purpose is to rank the relative performance of different details with respect to potential moisture trapping and another purpose is to obtain data which can be used for verification of the exposure models.

Furthermore suitable dose-response model for fungal decay will be developed. Existing dose-response models linking micro-climatic exposure (time series of moisture content and relative humidity) to risk for decay are going to be reviewed with respect to applicability for wooden bridges. The models are going to be validated against data on service life in reality.

The dose-response models are going to be used to evaluate durability and expected service life of the bridge design concepts proposed in WP4. It is expected that classification of details with respect to moisture trapping is an important part of this methodology, which will be evaluated by the industrial partners.

WP 3 Hygro-thermal effects in wooden members

Numerical models (FEM) relating rain, spray, RH, T, to distribute material climate in members is a major topic. External hygro-thermal loads, i.e. histories of RH, T and rain or spray will be represented in a form suitable for numerical simulations. FEM numerical methods available at VTT for hygro-thermo-mechanical modelling of timber structures will be further developed and assessed by including: influence of rain or spray, solar radiation and cracking in tension. The methods will be verified by comparisons with existing measurement data provided by the WP3 partners on small glulam specimens and as well as on full size timber bridge members.

The FEM numerical method will be used to calculate moisture distribution and moisture induced stresses in selected details of timber bridge elements and connections. Zones of contact of steel-wood details will be particularly analysed. The risk of cracking related to high moisture gradients and related moisture induced stresses (MIS) will be analysed by comparing the stress levels with the experimental values. Focused fracture mechanics analyses (or softening-based failure analysis) in critical points of the elements will be carried out. The influence of rain, spray and solar radiation will be particularly studied.

WP 4 Design concepts for durable timber bridges

Bridge decks of wood, especially stress laminated bridge decks and wood-concrete composite decks, are found to be beneficial. The effect of changing climate on large stress-laminated glulam timber decks with heights up to 1m and widths exceeding 10 m will be investigated by numerical methods developed in WP 3. Moisture content, MIS and creep effects, and possible damage due to compressive stresses caused by pre-stressed bars will be in focus. The models will be calibrated with experimental data. Furthermore, design details like waterproofing, paving and edge details on wooden bridge decks will be studied and possible improvements will be proposed.

Wood-Concrete Composite bridges seem to be good solution for small span traffic bridges. There are still possibilities to improve further the structure by testing new type deck and connectors. One promising structure is to use concrete element as deck structure. It fastens the building process considerably. Water proofing methods of the deck are going to be evaluated and developed. Several types of connections between wood and concrete will be considered and promising design concept will be explored and further developed and documented. Design solutions covering short, medium and long spans will be selected with respect to durability properties.

Fatigue of axial-carrying connectors in wooden members is a research covered by the Task 4.5. A very simple fastening technique will be explored to fasten the hangers to wooden members by use axial load carrying screws/threaded rods installed in the direction of the hangers. The fatigue set-up will cover the interface zone between the rod and the wood.

Methodology for performance evaluation of design concepts (structural performance, lifetime, LCC, LCA), will be developed on basis of the ETSI project, (see: http://etsi.aalto.fi).

For repair efforts, systems are being developed to meet current and future requirements, for example, slab-systems and girder systems and pile substructures all have several alternative repair solutions being developed to restore original member strengths or in some cases to strengthen members to carry loads significantly higher than originally designed for. Many variables related to exposure, durability and mechanical strength and stiffness are associated with large uncertainties. These can be represented by corresponding probabilistic models. Results of in-situ measurements can be utilized to update the probability models and thereby obtain a better estimate for the structural performance. The methodology will enable risk based optimization of inspection and maintenance strategies on timber bridges.

WP5. Dissemination

The aim of WP 5 is to disseminate the results and outcomes of the project, to increase knowledge among professionals and stakeholders in timber bridge research, development and construction.

Target groups are researchers, industry, authorities, standard organizations, and end users.

4 Design concept for durable bridges – network arch bridges

Project assumptions.

The choice of design concepts for long span bridges, are made on the basis of minimal moisture trapping and on the results from life-cycle analyses of existing timber bridges. The aim is to develop design concepts for a given span range. For the long span bridges a variant of the network arch bridge using large block-glued glulam cross-sections seems to be a promising candidate. The pre-stressed light-weight timber deck may prove beneficial and it is therefore included as a generic component for use in durable timber bridge design. In the WP 4 of the project, the major objective is to develop the network arch bridge concept in a so-called ‘spoked wheel’ configuration, or ‘spoked’ configuration in combination with a light-weight timber deck.

In network arch bridges, hangers cross each other at least twice and are located in one plane, see Figure 3. This concept was invented and developed by P. Tveit [4]. The main advantage of inclined configuration of hangers is that approximately an equally distributed moment action from loading located in skew position on the deck is achieved.

The design concept with ‘spoked wheel’ configuration of hangers is based on the network arch concept, but instead of single hangers, there are pairs of them having common fastening points on the arch level and separate fastening points on the deck level, displaced in the transverse direction, see illustration on Figure 4.

The main advantage of the new hanger configuration is higher out-of plane stability. When the hangers are inclined both in the plane of the arches as well as in the transverse direction, a lateral displacement of the arch (caused for example by wind) will lead to an elongation of hangers on the rear side and thereby tensile forces will develop to resist the lateral displacement. Even for very small displacements considerable strain will develop, but the sideway support effect will vary along the span [5]. The network arch principle has so far never been used for timber bridges, only for steel bridges with concrete decks. By introducing this concept for timber bridges some interesting properties emerge; it becomes easier to obtain the necessary in-plane stiffness and simplified connections for the hangers may be introduced. Furthermore, an important feature of the spoked wheel configuration is that this design may render the usual wind bracing trusses between the arches superfluous, and no connections are needed on top or side faces of the arch. This has important implications for the durability as most decay problems are located around exposed connections. In case there is a need for cover or surface treatment of the arches, this is made easy by a clean constant cross-section not interrupted by connection details.

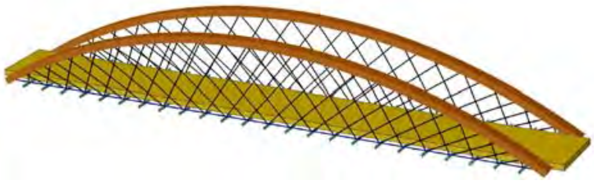

Figure 3: Network bridge

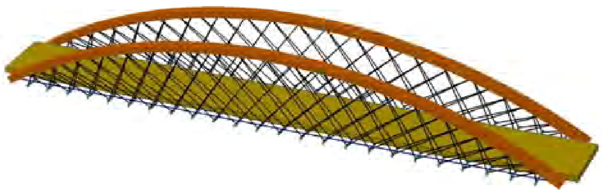

Figure 4: Bridge with spoked hangers

In WP4, configuration of the hangers (inclination and location), fastening of the hangers by axial rods to the arches, spoked wheel hangers and fastening system to the deck structure, and support for arches and deck (abutment) are issues which shall be emphasized. The challenge is to achieve sufficient stability and acceptable dynamical properties (vibrations), as well as necessary strength. The conceptual design will be worked out in three span lengths; 50, 100 and 150 m.

Arches for bridges exceeding 70 m span can hardly be delivered in one piece due to transport limitations, thus a practical splice of a large block-glued glulam section will be worked out and documented in WP4 of the project. First, two or three splicing concepts will be worked out and evaluated, and the chosen concept will later be evaluated by a scaled experimental tests of the splice. Experimental results will be used to validate numerical models.

Current research.

One of the project objectives is to develop arch bridges with self-stabilizing arches using hangers in a spoked wheel configuration. Some research on timber network arch bridges has already been performed at NTNU. The research consisted of the design of a 100 m long bridge and preparation of numerical and laboratory models, scaled 1:10, see the result on Figure 5. Design concept was based on the following assumptions: two lines of road traffic leading to width of deck of 10 m, two glulam circular arches, inclined network hangers in spoked wheel configuration, tension tie and no wind truss between arches and a timber stress laminated deck.

Figure 5: Laboratory model of the bridge

The scaled experimental model has close resemblance to the scaled numerical model and has been used to explore the real behaviour of materials and connections. So far, two dynamic test series have been performed on the model, with the use of the modal hammer method. The first series was performed directly after erecting the laboratory model in order to quantify the structural behaviour. The achieved results suggested need for experimental model modification. The modification consisted of pre-stressing the hangers and improving the deck support by clamping. These actions led to better dynamic response of the structure (natural frequencies and corresponding mode shapes and damping ratios). Simultaneously to laboratory research, numerical models were prepared and evaluated, [5] [6].

Results, conclusion and future work

Selected results of the dynamic tests performed on the laboratory model are compared with numerical results and presented in the Table 2. During laboratory tests, accelerometer was positioned in center part of the structure or on the edge. Both results are presented in the table.

Table 2: Comparison of natural frequencies of the bridge, obtained in a laboratory and by numerical tests; (NF-natural frequency, DR-damping ratio)

		Tests on the evaluated laboratory model				Numerical results
		Accelerometer on side		Accelerometer in center		
Element /trend	Mode number	NF [Hz]	DR [%]	NF [Hz]	DR [%]	NF [Hz]
DECK transversal	1	**16.977**	2.133	**17.016**	2.531	**24.901**
	2	**55.527**	1.934	**54.741**	2.890	**58.470**
	3	**107.43**	1.006	**107.52**	1.335	**96.461**
DECK vertical	2	**17.857**	0.883	**17.399**	0.994	**27.674**
	3	**27.837**	1.271	**29.990**	0.801	**38.362**
	4	**38.282**	1.116	**37.482**	1.060	**47.626**
	5	**51.996**	0.938	**53.144**	1.187	**61.524**
	6	**71.392**	0.656	**68.293**	1.633	**71.818**
ARCH transversal	2	**13.482**	6.930	**14.599**	1.542	**21.786**
	3	**31.233**	2.454	**31.615**	2.143	**40.786**
	4	**53.357**	1.968	**54.321**	2.365	**65.010**
	5	**81.432**	0.677	**81.358**	1.802	**84.383**

The results show good correlation between measured and calculated (FEM modelled) values of natural frequencies and corresponding mode shapes, especially for the higher modes. Furthermore the obtained properties are located in an acceptable range. The physical laboratory model has some flexibility in the lower mode vibrations which are absent in the idealized numerical model. Consequently, there is also a potential for structural improvements both with regard to the deck structure as well as to the behaviour of the connections. The opinion of the authors is that with these improvements the spoked arch configuration will become an interesting alternative for long span bridges.

Future part of the research will be focused on relation between stiffness and stability of network arches. It is expected that hangers with spoked configuration are a potential solution for avoiding wind bracing and be a better alternative for network bridges.

5 Acknowledgement

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Timber Bridges in Styria | Austria as Result of TU Graz Education*

Gerhard Schickhofer[1], Andreas Ringhofer[2], Georg Flatscher[3]

Summary

Even though their span length is limited by material properties as well as their shape is limited by requirements subject to durability, well-designed timber footbridges can be impressive and aesthetic constructions. Motivated by that, we present four timber footbridge examples, which have been erected in the province of Styria (Austria) in the last two decades. Thereby, we not only focus on the main topics 'deck construction', 'structural system' and 'constructive wood protection' but also on the varying boundary conditions influencing the project's realisation process. In major cases, civil engineering students educated at Graz University of Technology, especially by the Institute of Timber Engineering and Wood Technology, played an important role in this process. Finally and with regard to the projects presented, we summarise the major principles to be considered when designing timber footbridges.

Keywords: Timber footbridges; deck constructions; constructive wood protection; cross laminated timber

1 'Timber' in research and teaching at Graz University of Technology

The Institute of Timber Engineering and Wood Technology as part of the Faculty of Civil Engineering in combination with the competence centre holz.bau forschungs gmbh is responsible for research and education activities in the science fields 'timber engineering' and 'wood technology' at Graz University of Technology (TU Graz). The major tasks of the institute can be divided up into four columns: 'lignum_study', 'lignum_research', 'lignum_engineering' and 'lignum_test_center' (LTC, accredited laboratory). Together with the holz.bau forschungs gmbh, which actually has nine shareholders (six from industry, two from science and one association) and Graz University of Technology as main owner, the institute is working on a four-year research programme (duration until 2016). The scientific staff includes 14 members all in all (partially PhD students and postdocs) and works with an total budget of approx. 1.2 Mio. € per year on the following topics:

- Timber Engineering (TE) – Design and Construction Sciences (DCS)
    - Shell and Spatial Timber Constructions (SSTC)
    - Innovative and Smart Connection Systems (ISCS)
    - Assessment, Monitoring and Maintenance of ancient Timber Structures (AMTS)
- Wood Technology (WT) – Material and Structure Sciences (MSS)
    - Material Modelling and Simulation Methods (MMSM)
    - Advanced Products and Test Methods (APTM)

Parallel to the scientific activities mentioned above, courses in the bachelor (Civil Engineering Sciences with Environment and Construction Management) as well as in the master programme (Civil Engineering Sciences and Structural Engineering) are offered by the institute with main focus on the latter one (see Table 1). Thereby, basic skills in timber engineering are taught in the lectures 'Building Materials Basics (part Wood Technology)' and 'Basics of Timber Engineering'. There is also the option to write the bachelor thesis in this discipline. Furthermore, students can complete and expand their knowledge in the context of various master courses as the requirements for the constructive block as shown in Table 1.

* The contents of the paper have first been presented at the Footbridge 2014, 5th International Conference, London

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Table 1: Overview of bachelor and master courses offered by the Institute of Timber Engineering and Wood Technology at TU Graz

Programme	Course	ECTS	Type
Bachelor	Building Materials Basics (part Wood Technology)	0.5	Basic Skills
	Basics of Timber Engineering	4.0	
	Project (Bachelor)	5.0	
Master	Timber Engineering 1	3.0	Completion and Expert Skills
	Timber Engineering 2	3.0	
	Risk and Safety in Civil Engineering	3.0	
	Gluing Technology and Wood-based Materials	1.5	
	Assessment and Maintenance of Timber Structures	1.5	
	<u>Structures in Timber</u>	5.0	Construction Skills
	<u>Timber Bridges</u>	1.5	
	Wood Preservation	1.0	
	Timber and Forestry Economy	1.5	Practical Skills & Excursion
	Practical Course 1	2.0	
	Practical Course 2	2.0	
	Excursion to Steel, Timber, Concrete and Composite Structures	1.0	
	Project (Master)	5.0	
	Master Thesis	30.0	Scientific Skills
PhD	Research Seminar on Timber Engineering and Wood Technology	2.0	Scientific Skills

While the course ‘Timber Bridges’ is organised as a mix of theoretical and practical lectures (teacher-centred frontal instruction), ‘Structures in Timber’ follows the philosophy of the so-called principle of master classes. This means that up to thirty master students share the opportunity to work on real life projects in the frame of an intensive two-week blocked course, while teaching staff as well as additional lectors (engineers and architects from private companies) support them with their interdisciplinary expertise. In the following subsections we focus on these both courses in particular.

1.1 Master Course ‘Timber Bridges’

As shown in Table 1, ‘Timber Bridges’ is a course for master students and contains fifteen lectures (each 45 minutes), in which they acquire the special characteristics regarding the construction of timber bridges. Hereby, the identification of the main components of a timber bridge as well as their structural design, are regarded as the educational objectives of this course. Due to the fact that practical lectures alternate with theoretical ones, students also learn about the diversity of constructive details and how they are designed. The content of the associated script [1] with more than 300 pages includes the following items:

- Introduction: Modern contemporary timber bridges
- History of timber bridge construction
- Timber products and connection technique
- Wood protection
- Standards and guidelines
- <u>Structural analysis of timber bridges</u>
- <u>Deck constructions</u>
- Maintenance manual
- Master details
- Practical example: Structural design of a laterally pre-stressed slab bridge

Therefore, the main chapters are “Structural analysis of timber bridges” and “Deck constructions”. In the latter one, traditional as well as modern deck constructions are analysed extensively:

“1.4 Deck constructions

Deck constructions contain the structural components of a deck, the waterproof seal and the road surface. Board systems – the traditional construction form (use of linear timber products, see Figure 1 left) – as well as slab systems – the modern construction form (use of planar timber products, see Figure 1 right) – can fulfil the major tasks of a deck construction.” [1]

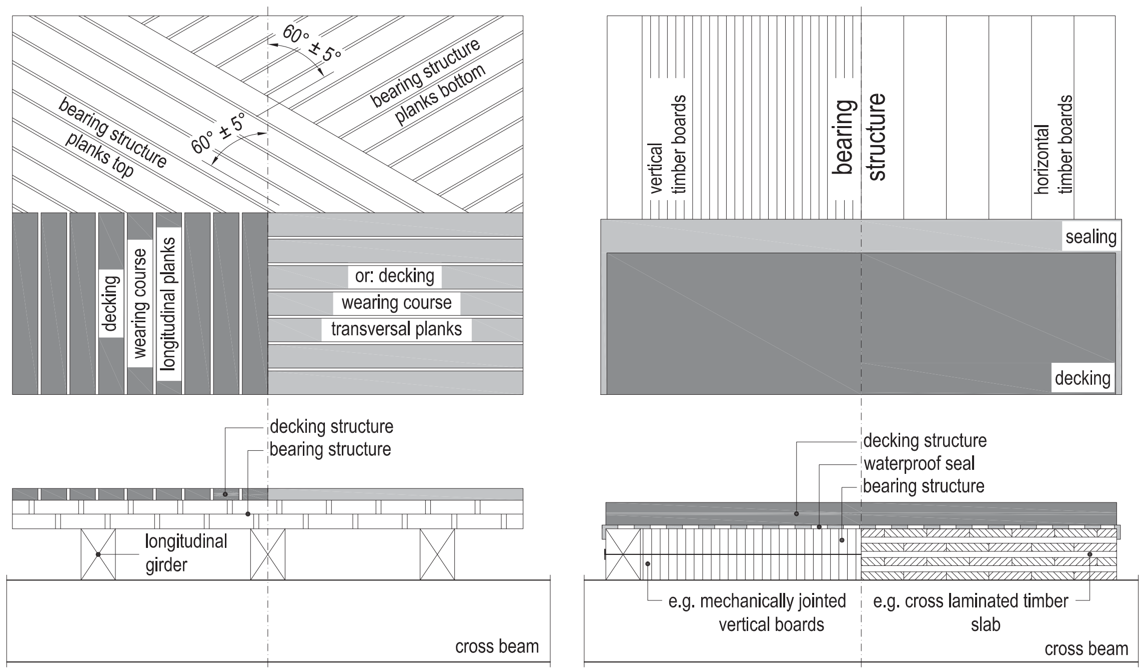

Figure 1: Comparison of traditional (left) and modern (right) deck constructions; according to [1]

Needless to say, both subjects, ‘deck constructions’ and ‘structural systems’, are key topics in the description of the timber footbridge examples given in section 2. In this publication, we define the structural system as the sum of all elements responsible for load bearing and transfer necessary to guarantee the structural safety of the construction. Depending on the bridge type, one or more tasks are fulfilled by one structural element:

- primary structure (PS)
  The primary structure contains all the main bearing components of a bridge, which range from one abutment to the other, responsible for load transfer into the basements.
- secondary structure (SS)
  The secondary structure contains cross beams and – if not part of the primary structure – suspenders and columns responsible for transfer of vertical loads from the bridge deck and/or the roof (traffic, snow).
- tertiary structure (TS)
  The tertiary structure contains the deck construction and – if existing – the deck girders responsible for transfer of traffic loads to secondary and primary structure.

1.2 Master Course ‘Structures in Timber’

As mentioned in section 1.1 and as alternative to the usual teaching philosophy at TU Graz, master students work together in the context of a blocked interdisciplinary seminar which takes two weeks all in all. In chronological order this course is the last one offered by the Institute of Timber Engineering and Wood Technology in the master programme. Therefore, it is seen as a link between theory and practice combining engineering with architecture and, hence, could also be named: ‘From design (or ‘from a blank sheet’) to the construction’.

The course syllabus is quite simple: Eight to ten teams consisting of three to four students each have the choice to accept the challenge testing their theoretical skills and knowledge on a real life problem. Thereby, growing frustration based on missing know-how regarding the design process of a construction is guaranteed at the beginning of the course. The duty to brood over architectural design processes pushes the master students of Civil Engineering Science and Structural Engineering to their limits; they realize the difficult path architects have to go.

After a first intensive week of supervised teamwork including many failed design try-outs, the majority of the students is able to work out two or three good approaches to solve the problems. The final choice, which approach should be treated in detail is based on the outcome of a first presentation and a discussion with the whole class and supervisors. Consequently, the tasks of the second week are the further development of the chosen draft: considering locational boundary conditions when integrating the construction in the landscape; creating the structural design combined with constructive issues; selecting an adequate connection technique and structural detailing. Finally, all groups are evaluated by their supervisors in the course of presenting and discussing their projects in class. The timber footbridge projects described in section 2 underline the fact that those student drafts are not only theoretical ideas but also realised in some cases.

2 Timber footbridges in Styria

The realisation of the so-called 'Holz-Europabrücke' ('European timber bridge'; structural analysis and design: G. Schickhofer, see Figure 2) in 1993 as well as the Styrian National Exhibition entitled 'TimberTime' in 1995 inaugurated the 'Styrian epoch of timber bridges'. The formerly mentioned milestone project demonstrated the high performance of timber constructions and was a forerunner of a variety of timber bridges erected in Styria and in Austria in the upcoming years. The projects presented in the following subsections are an extract of this era having its highlights in the late nineties of the twentieth century.

Figure 2: Impression of the 'European timber bridge'

2.1 Project 'Leonardo Bridge Fürstenfeld', 1995

As mentioned before, in 1995 the Styrian National Exhibition entitled 'TimberTime' took place in the northern Styrian town 'Murau'. In the course of the preparation for this event, a competition among students, young architects and civil engineers regarding the construction of a bridge called 'Rantenbachsteg' was organised and advertised by the Graz University of Technology in cooperation with the association 'proHolz Steiermark' in 1993. One of the submitted 34 projects was a system called 'Leonardo Bridge'. Even though the project was not awarded, the concept awoke curiosity within the jury. One year later the idea of the bridge was realised in another Styrian village called 'Großwilfersdorf'. Two further bridges using this system should follow, one in the eastern Styrian town 'Fürstenfeld' (1995) and another in the northern Styrian village 'Katsch-Frojach' (1996). The following impressions show the interesting bearing system on the basis of the footbridge erected in Fürstenfeld with a span length of 27 m.

Figure 3: Impressions of the timber footbridge 'Leonardo Bridge'

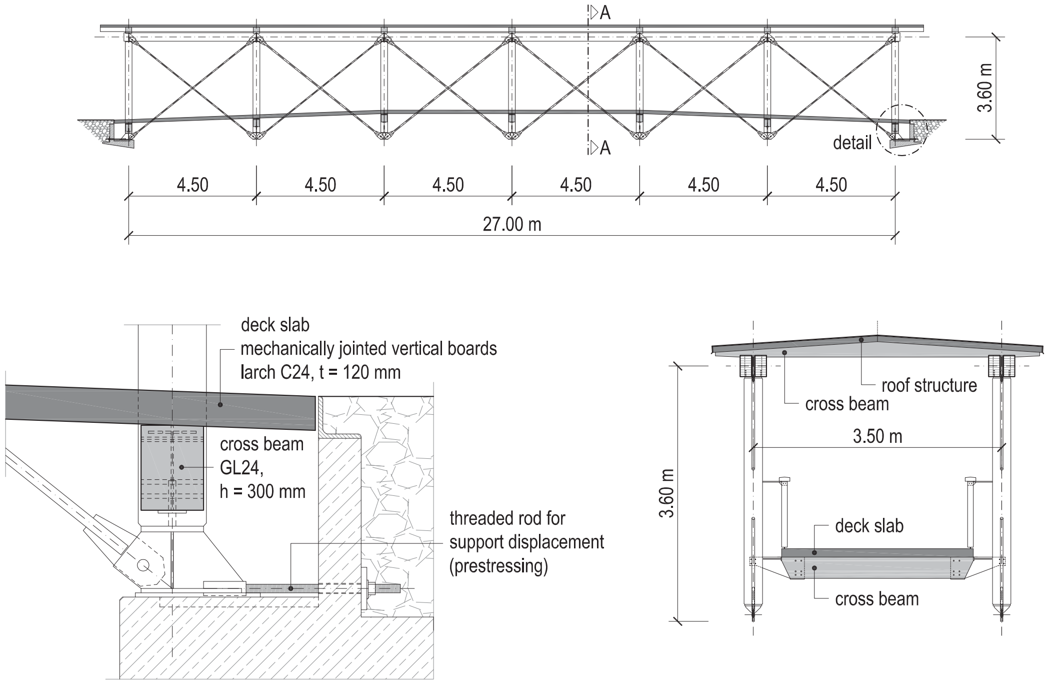

Figure 4: Technical drawings; above: elevation; left: detail; right: section A-A

As already implied by the name, the bridge concept follows a draft proposal made by Leonardo da Vinci in "Codex Atlanticus" (Folio 855), which can be interpreted as a certain form of a reverse trussed single-span system. However, the engineers realized that the idea of Leonardo da Vinci could be used in a more sufficient way if the structure was turned upside down. Only the low stiffness of the system was seen as a challenge. To avoid uneconomic cross sections, which may appear to be necessary to achieve the serviceability limit state, the bridge was pre-stressed by support displacement. In doing so, the primary structure, defined in section 1.1, only consists of the compression chords (GL24h, 2 x 160/300 mm) the steel braces (S355, Ø 40 mm and Ø 50 mm) and the columns (GL24h, 240/260 mm). Herein, the cross beams supporting the deck (GL24h, 220/300 mm) and the roof (GL24h, 220/220 mm double pinched) are regarded as the secondary structure. Consequently, the tertiary structure can be considered as the deck construction (mechanically jointed vertical boards, C24, h = 120 mm) as well as the roof (mechanically jointed vertical boards, C24, h = 80 mm) and the steel braces beneath the deck (S235, Ø 24 mm), which were also used to transfer the horizontal wind loads. As part of the bridge portal, steel rods finally transfer these loads from the roof to the abutment (S235, Ø 24 mm), see Figure 3. All timber elements were produced in larch (Larix decidua) excluding the deck and roof slab elements, which were made of Norway spruce (Picea abies).

As an essential detail of this bridge, the supporting situation has to be highlighted. As shown in Figure 4, the 'rails' on which the steel shoe is standing on, enable a smooth support displacement of 30 mm at each of the four portal columns by using a simple threaded rod. Furthermore, the higher stiffness of the bridge in winter, caused by the lower temperatures in comparison to the month of May when the bridge was mounted, allows the bridge a kind of intelligent reaction against the snow loads, see also [2].

2.2 Project 'Building Corridor Bad Gleichenberg', 1996

In contrast to the other projects described in section 2, which are all parts of the public road network, the building corridor erected in a small eastern Styrian village called 'Bad Gleichenberg' in the year of 1996 connects two building blocks of the spa resort 'Gleichenbergerhof'. The main reason making the structure out of timber, were the lower building costs compared to a solution in steel (see also [2, 3]). Regarding its form and dimension, the construction is similar to a common timber footbridge and thus presented in this publication. Architectural design and structural analysis were done by 'lignum_consult' (group founded by former students of Graz University of Technology), production and mounting by the company 'Buchacher' (now part of the 'Hasslacher Holding GmbH').

Figure 5: Impressions of the timber footbridge ‘Building Corridor Bad Gleichenberg’

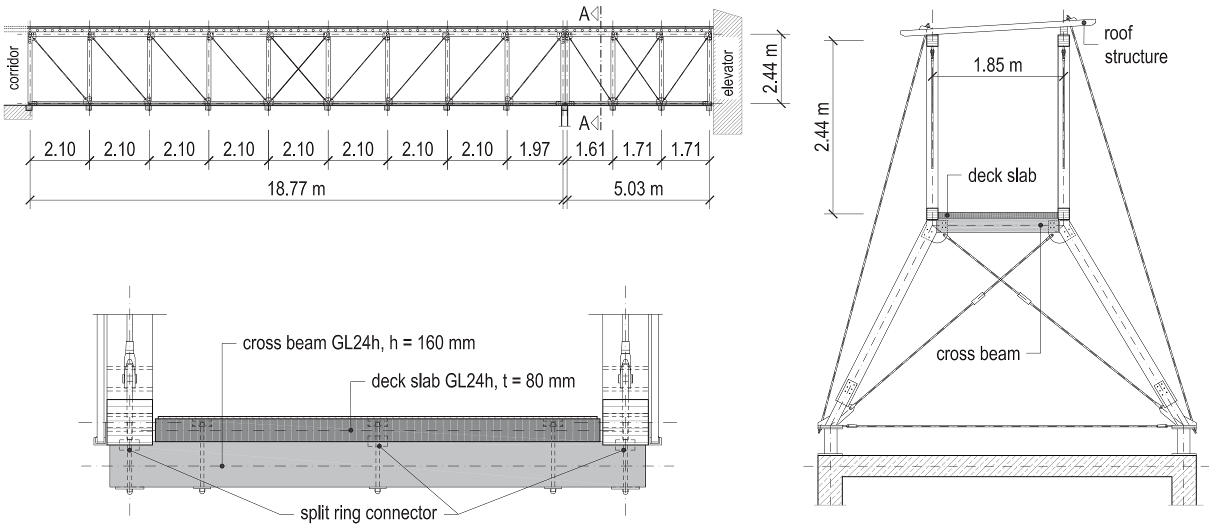

Figure 6: Technical drawings; top left: elevation; bottom left: cross section of the deck construction; right: section A-A

Figure 5 presents the corridor walls made of safety glass protecting the whole structure against water ingress caused by rainfall or snow. Consequently, there was no need to situate additional layers (waterproof surface) onto the glulam deck construction (GL24h) acting as tertiary structure of the system (linear multi-span girder). Thence, traffic loads – only caused by pedestrians – are transferred to cross beams (secondary structure) situated at the nodes of the truss girders. Besides bearing main vertical loads, the deck construction also serves as single-span bending beam in case of horizontal wind loads, which are partially transferred from the glass walls over the bottom chords into the cross beams and from there into the deck girder. In this context we want to point out the horizontal connection between cross beam and deck girder, realised by using a split ring connector (system ‘APPEL’) and by situating it in its centre axis, allowing free deformation caused by shrinkage and swelling of the timber. The second half of horizontal wind loads has to be carried by the truss system situated at roof level which components are the glulam top chords of the primary structure, steel braces and glulam posts (also GL24h).

2.3 Project ‘Raabsteg Feldbach’, 1998

The motivation to realise this footbridge project over the river Raab in the eastern Styrian town named ‘Feldbach’ was to connect a recently erected health centre with the urban road network in the year 1996. Therefore, while attending the course ‘Structures in Timber’ a group of master students developed three concepts : (i) a deck bridge in form of a fish-bellied girder, (ii) a king post truss bridge and (iii) a trussed plate girder bridge. Finally, the clients decided in favour of concept (iii), which consequently was built in 1998 by ‘Holzleimbau Stingl’ (production and mounting) in cooperation with ‘lignum_consult’ (architectural design and structural analysis).

Figure 7: Impressions of the timber footbridge 'Raabsteg Feldbach'

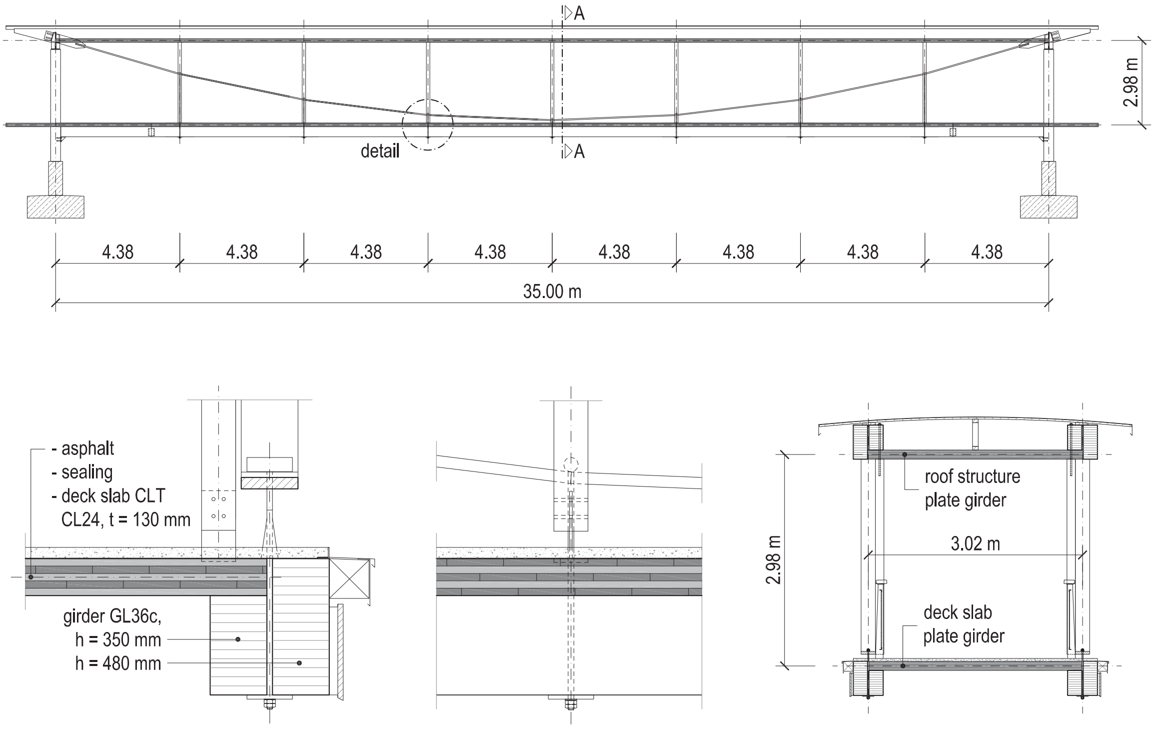

Figure 8: Technical drawings; above: elevation; left and middle: detail; right: section A-A

The cross-section presented in Figure 8 displays two CLT (cross laminated timber, CL24 according to [4], t = 130 mm, 5 layered) - GLT (glued laminated timber, 200/350 and 200/450 mm, GL36c) plate girders in Norway spruce as main components of the primary structure, which can be assumed as an interlinked single-span system with a total span length of 35 m. The trussed girder (tension chord, 200/40 mm, S355) situated in the roof level is responsible for the main load bearing action (symmetric snow and traffic loads; load transfer through columns (120/120 mm, GL28h, larch) and suspenders (Ø 24 mm, Istor TX55) as secondary structure) while the deck girder has to bear asymmetric traffic loads and, in addition, significantly increases the system stiffness. Due to the combined CLT-GLT system in both roof and deck level, horizontal loads (wind loads perpendicular to the bridge axis, (minor) traffic loads in axial direction) are transferred by the in-plane acting CLT elements to the portal frames made of reinforced concrete. Consequently, no arrangement of horizontal bracing was necessary.

In the year 2000 the project won the Styrian award in timber engineering (category: public building). At the same time further research findings concerning cross-laminated timber (CLT) enabled the assessment of vibrations caused by pedestrians according to the (at that time) new ENV 1995-2 [5]. This procedure contained structural analysis using 3D beam analysis software (RM2000) as well as in-situ measures and was performed in the context of a diploma thesis at TU Graz, see [6]. According to this assessment the first vertical eigenfrequency resulted to $f_{1,pred} = 2.94$ Hz (measured : $f_{1,exp} = 2.64$ Hz), which is located in a range in which pedestrians are already negatively affected by vibrations. However, as a consequence of proper damping behaviour ($\delta = 0.043$ at $f_{1,exp} = 2.64$ Hz) the arrangement of vibration absorbers was not necessary.

The constructive wood protection as key aspect in the design of timber bridges was fulfilled by using a roof structure as well as a road surface as modern deck construction including asphalt combined with a waterproof protective coat. The only timber members affected by rainwater are the columns. Hence, larch wood with a higher durability compared to Norway spruce was used.

2.4 Project ‘Truss Bridge Kindberg’, 2013

In 2013 the old ‘train-station-bridge’, connecting the centre of Kindberg, a small town to the north of the Styrian capitol Graz, and its train station by bridging the river Mürz, was replaced by a new one. In the context of the seminar ‘Structures in Timber’ in 2008, five concepts were designed by the participating students. After the seminar, two of those concepts, (plus a completely new one) were further developed in the context of master projects supervised by the institute (see Table 1). In the year 2009 the clients decided to realise the herein discussed truss bridge in a meeting by assessing the formerly mentioned drafts. Four years later, the bridge was built with minor adaptions compared to the students’ project by ‘Kulmer Holzbau’ (production and mounting) in cooperation with ‘DI Helmut Stingl, Tragwerk Consulting Engineering’ (structural analysis).

Figure 9: Impressions of the timber footbridge ‘Truss Bridge Kindberg’

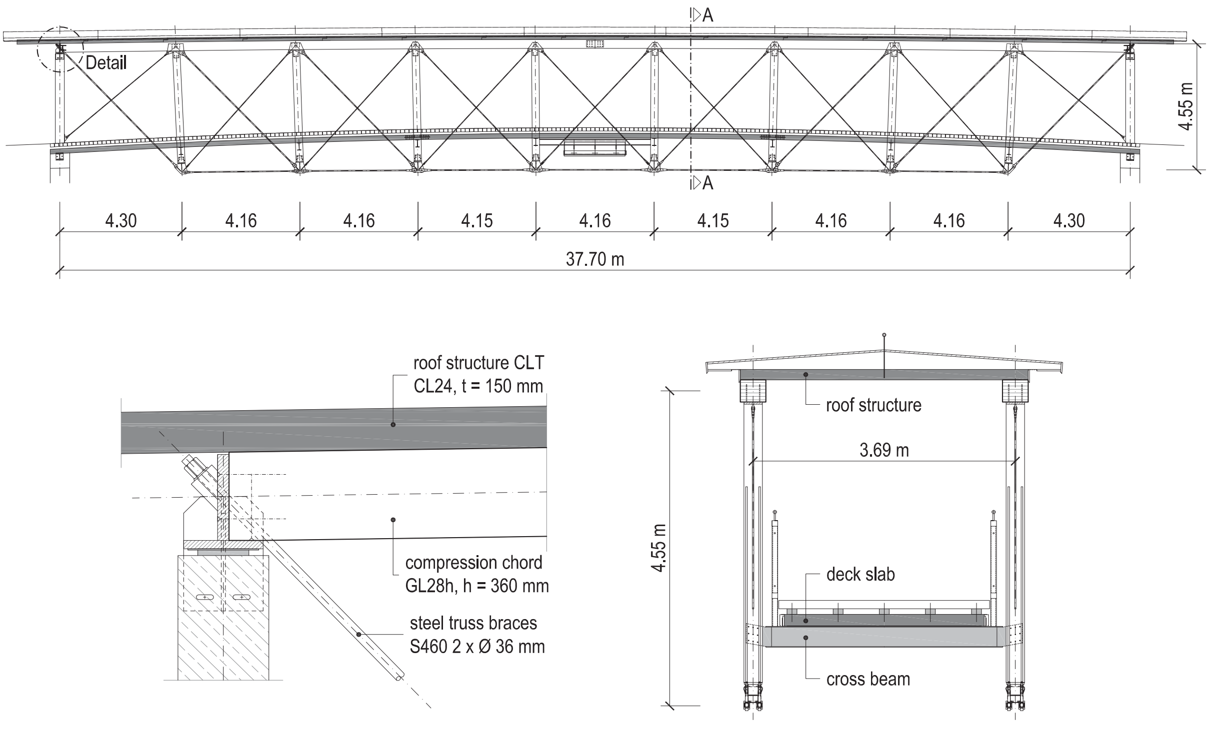

Figure 10: Technical drawings; above: elevation; left: detail; right: section A-A

The primary design idea of the bridge was quite similar to the ‘Leonardo Bridge’ (see section 2.1), but due to the larger span length (about 38 m), it was impossible to fulfil the serviceability limit state criterion regarding the vibrations. Hence, a trussed system was chosen in which the primary structure could be detected as the compression chord (GL28, 320/360 mm), the steel truss (2 x Ø 36 mm to 2 x Ø 48 mm, S460) as well as the steel braces (Ø 20 mm to 2 x Ø 36 mm, S460) and the columns (GL28, 260/260 mm). The secondary structure just includes the I-section steel cross beams (IPE 270 mm, S235), which were jointed to the columns of the primary construction via a dowelled connection. The tertiary structure contains the deck construction, consisting of the load-bearing CLT-element (CL24 according to [4], 160 mm, 5 layered), a doubled waterproof seal, non-bearing pressure-impregnated LVL beams and the larch planking as wearing course, and the roof (CL24, 150 mm, 5 layered).

Added to their main load bearing functions (snow, traffic loads), these structures also have to transfer the horizontal loads to the abutments.

As one of the most interesting details we want to single out the point in which the axial tensile forces from the braces get introduced to the compression chord (see Figure 10). Compressive reaction forces are directly transferred by contact, which avoids the arrangement of a highly stressed dowel-type or screwed connection. A further highlight can be regarded in the fact that the option for placing a damping mass in the middle of the bridge – after its mounting – was considered in design process. The motivation behind this step was based on the experience concerning the incomparability of the calculated eigenfrequencies of foot bridges to the measured ones after having finished the construction process (in contrast to the results shown in section 2.3). After erecting the bridge and in corporation with the clients, the decision was made to use the option of situating the damping mass, even though it was obvious that the eigenfrequencies were in an acceptable range in the context of normal use.

Finally, the construction of the deck as well as the sheeting of the columns have to be outlined as essential points of the constructive wood protection (in addition to the roofing of the bridge).

3 Concluding Remarks

In section 2, four timber footbridges erected in the province of Styria (Austria) were presented and especially discussed regarding their structural system, the way their deck construction was designed and their constructive wood protection. Regarding the formerly mentioned issues the Institute of Timber Engineering and Wood Technology played a major role in education and consultation of engineers realizing those projects. The broad knowledge as result of this process makes it possible for us to provide a final summary of the major principles for the design of timber footbridges:

- compliance with the principles of constructive wood protection from design to construction;
- service class 2 for all load bearing timber members to guarantee an economic and durable construction;
- making use of planar elements like cross laminated timber for deck and roof constructions which able to bear vertical (traffic) as well as horizontal (wind) loads;
- selection of well-known and proved structural systems applied with stress-related materials in form of hybrid constructions and intelligent combinations of steel, timber and concrete;
- application of simple and pre-fabricable connections, especially adapted to the distribution of forces given.

The course compilation which is offered to civil engineering students at Graz University of Technology (see Table 1) underlines the extensive but also compact educational path in the field of ‘Timber Engineering and Wood Technology’, lasting from the bachelor to the PhD programme. Both courses ‘Timber bridges’ and ‘Structures in Timber’, focusing on the construction skills, follow the main objective to derive the students’ pleasure in design and construction. As a consequence of ‘learning-by-doing’, the ‘art of trying’ and the teamwork, students acquire the skills and self-assurance necessary to competently represent their solutions to their prospective clients.

4 Acknowledgement

Finally, we would like to acknowledge the following companies for supporting us with documents concerning the footbridge projects presented in section 2: ‘DI Helmut Stingl, Tragwerk Consulting Engineering’ and ‘Kulmer Bau GmbH’. Special thanks go to David Fruhmann for drawings, Architect Dr. Helmut Pierer for photographs and Gerald Raser and Reinhard Brandner for text corrections.

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COST ACTION FP1101

Damage and decay of a timber footbridge, based on multi-annual deflection and acceleration measurements

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Summary

The dynamics of a 30m-long timber footbridge in Patras, Greece, were systematically investigated in annual surveys between 2007 and 2013 using GPS, robotic total stations (RTS) and accelerometers. The monitoring was mainly focussing on the response of the bridge to free attenuating oscillations following forced excitations by pedestrians. Dominant frequencies along the lateral axis deriving from RTS and accelerometer measurements are analysed. The first survey of 2007 indicated a good condition of the bridge, with much stiffness in the lateral axis (modal frequency 2.6Hz). The analysis of the following years indicated a dramatic drop (1.6Hz) of this modal frequency followed by additional gradual drop in the following years. The inferred rapid changes of the bridge stiffness are reliable, occurred during an interval characterised by a strong earthquake and an extraordinary for the area icing event; signs of decay in the wood and metal elements of the bridge are obvious.

Key words: timber footbridge, monitoring, non-destructive testing, structural health, fatigue

1 Introduction

Damage and aging of structures are reflected in the reduction of their main modal frequencies (loss of stiffness) [1, 2]. However, (1) such changes are usually small and not easy to be identified, (2) the dynamic characteristics of structures depend on the loading and ambient conditions [3, 4, 5] and (3) systematic monitoring of structures is rare, especially in timber bridges [6]. For these reasons very rarely there have been identified changes in the dominant frequencies of structures, clearly reflecting decay or damage.

For this reason the observed changes of the dynamic characteristics (changes in the dominant frequencies) of a wooden footbridge in Patras, Greece, the Kanellopoulos footbridge (Fig. 1) may be of major interest [7, 8] . Our study was in the framework of a program for the development of a methodology for the measurement of deflections and evaluation of the structural health of various structures, mostly relatively stiff bridges (dominant frequencies >1Hz), using geodetic instruments (GPS and Robotic Total Stations, RTS) and accelerometers [9-13].

In this paper we summarize some results from the study of the Kanellopoulos footbridge on the basis of measurement of lateral deflections and accelerations of the bridge when excited by a group of pedestrians [14-18]. Several geodetic sensors and accelerometers were used to measure the deflections and accelerations of critical points in the mid-span of the bridge in three directions in a coordinate system independent of the bridge during forced excitations by pedestrians. In this paper, however, we focus on the changes in dominant frequencies in representative spectra of lateral deflections deriving from RTS and deriving from accelerometers and describe the first natural frequency of the footwbridge. These spectra correspond to free attenuating oscillations following each excitation event. The analysis of such spectral peaks from repeated annual surveys provided evidence of both episodic and gradual, significant changes in the stiffness of this structure. These results are consistent with eye-witnessed evidence of decay, and the study-bridge is usually avoided by pedestrians, especially because of the feeling of discomfort due to large-scale vibrations [8].

2 The Pedestrian Bridge

The pedestrian timber bridge studied (Fig. 1) is crossing the Kanellopoulos St., at the northern entrance of Patras, a few kilometers away from the Patras University Campus. It was constructed in 2000, has a maximum length of 29.5, a central opening of 26.5m and consists of wood and certain steel elements. The bridge deck is formed by two twin horizontal beams on the two sides of the bridge floor, supported by two wooden arches. Thus, the bridge can be regarded as a continuous beam based on two joint columns and with intermediate support provided by the timber arches [8].

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The study bridge is rather stiff in the vertical axis due to the two sets of the twin beams running along its deck and the additional support by the wooden arch, while excessive horizontal lateral deflections were planned to be avoided by two layers of stiff, contiguous square frames along its main axis, beneath the deck and along its ceiling. However, many crucial elements of the square frames were omitted or were defectively constructed, not guaranteeing the desired stiffness, especially along the lateral axis. Furthermore, the loosening of the connections between the bridge structural members, permitted, especially in the last years, relatively large deflections of the deck; this created a feeling of insecurity to the crossing pedestrians, who have almost abandoned the bridge mainly, after 2008. This phenomenon is somewhat similar to the lateral deflections of the Millennium Bridge in London [19, 20].

Figure 1: *Side view of the Kanellopoulos pedestrian timber bridge in Patras. A Robotic Total Station (RTS) is shown in the foreground, while arrows indicate collocated sensors, including RTS reflectors and accelerometers.*

3 Bridge annual monitoring

The methodology of the bridge monitoring, which was adopted in all years between 2007 and 2013, was to force the bridge to oscillate using different types of excitation by a group of pedestrians (heel drops, walk, coordinated walk, running, coordinated jumps [14-18]. The goal was to excite the bridge until a maximum vertical or lateral displacement was obtained, just before the sense of an impending collapse. Then the excitation was abruptly interrupted and the free attenuating oscillation was recorded [11, 13].

The deflections of the bridge were measured by a number of geodetic sensors and accelerometers. More specifically, two sets of collocated instruments consisting of one GPS receiver, one high-quality (optical) reflector sighted by a Robotic Total Station (RTS), set on the ground and one three-axial accelerometer were rigidly clamped on each side of the bridge deck, on the stiff bridge handrails (Fig. 1). A GPS was also established on the crown of the bridge arch. The adopted methodology is analysed in [8-13]. Additional instruments were used, including a high accuracy distance measurement instrument (Tellurometer MA200) recording horizontal distances with an accuracy of 0.5mm at a sampling rate of 58Hz.

In this study we focus on the recordings of an RTS sighting on a reflector on top of an accelerometer fixed on one of the stiff handrails at the mid-span of the bridge. These two fully collocated sensors recorded the overall bridge response to excitations produced by jumps, nearly identical for all annual surveys. The instruments used were a Leica TCA 1201 RTS with upgraded software to record movements with an average sampling rate of 7 Hz and time stamping of 0.01sec, an AGA reflector and a Geo-Sig AC-23 force-balance tri-axial accelerometer with a sampling rate 100Hz and GPS timing.

4 Data analysis and results

The RTS records provide a set of observations of changing polar coordinates of a reflector following the movement of the midpoint of the bridge. These data were transformed into Cartesian coordinates adapted to the bridge axes and to deflections relative to the equilibrium. Details on the methodology of analysis and time stamping of the RTS data can be found in [11, 13]. In the last years, lateral deflections of several cm were observed in this footbridge [8]. Acceleration during excitation events were also recorded, and on the basis of these measurements it was possible to identify the intervals of free attenuating oscillations, up to approximately 8 seconds long [8], at the queue of certain successful excitation events.

From the deflections and accelerations in the lateral axis the corresponding spectra were obtained and are shown in Figs. 2 and 3. Because the excitation intervals were short (a few sec long) and the sampling rate of RTS is low (7Hz on the average) and unstable, spectra were computed using the NormPeriod Code which is based on least-squares [21]. The advantages of this spectral analysis technique, which offers an accuracy compatible with FFT in common cases (see [7]), are among others, that this Code permits to analyse short time series without padding (i.e. introducing additional noise) and to define the uncertainty level of spectral peaks (shown as a horizontal dashed line in Figures 2 and 4).

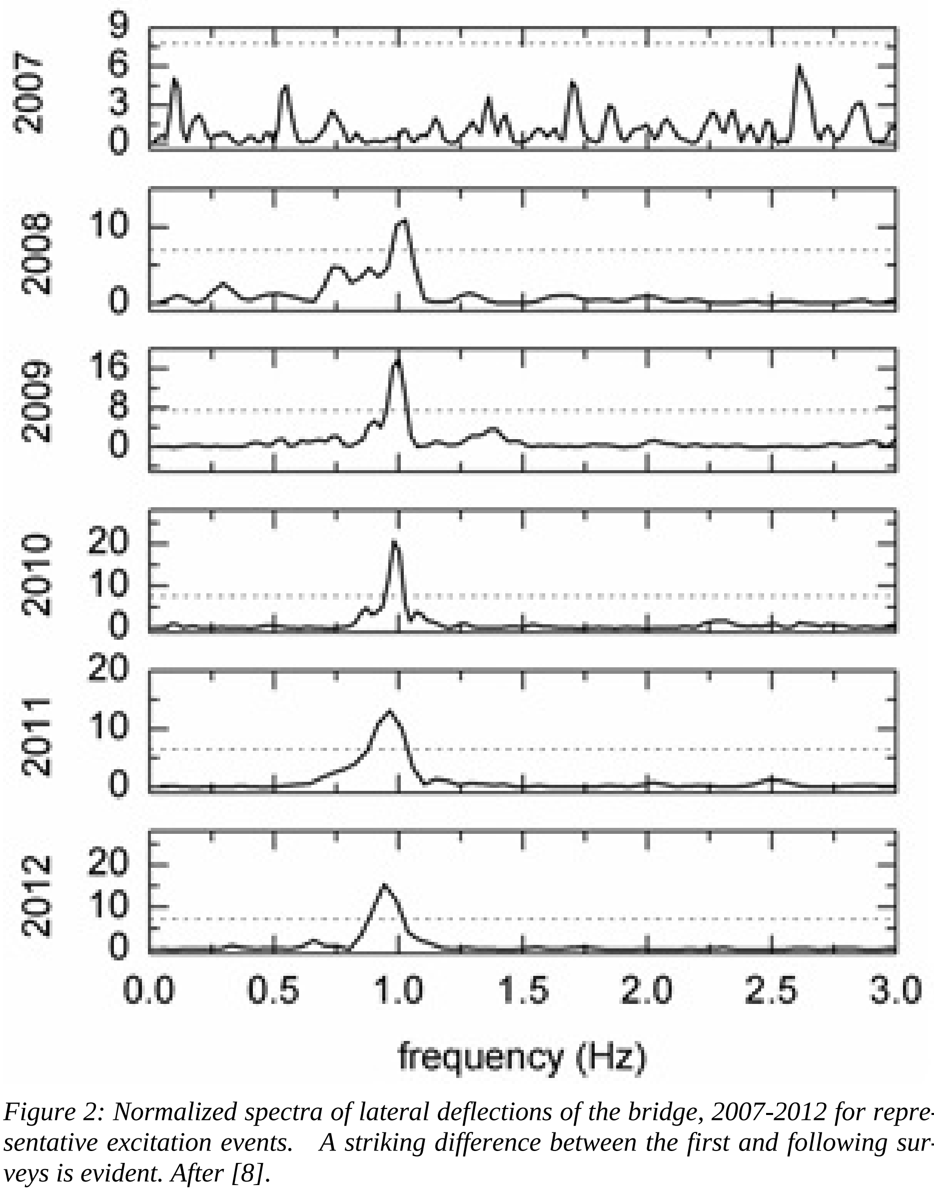

Figure 2: Normalized spectra of lateral deflections of the bridge, 2007-2012 for representative excitation events. A striking difference between the first and following surveys is evident. After [8].

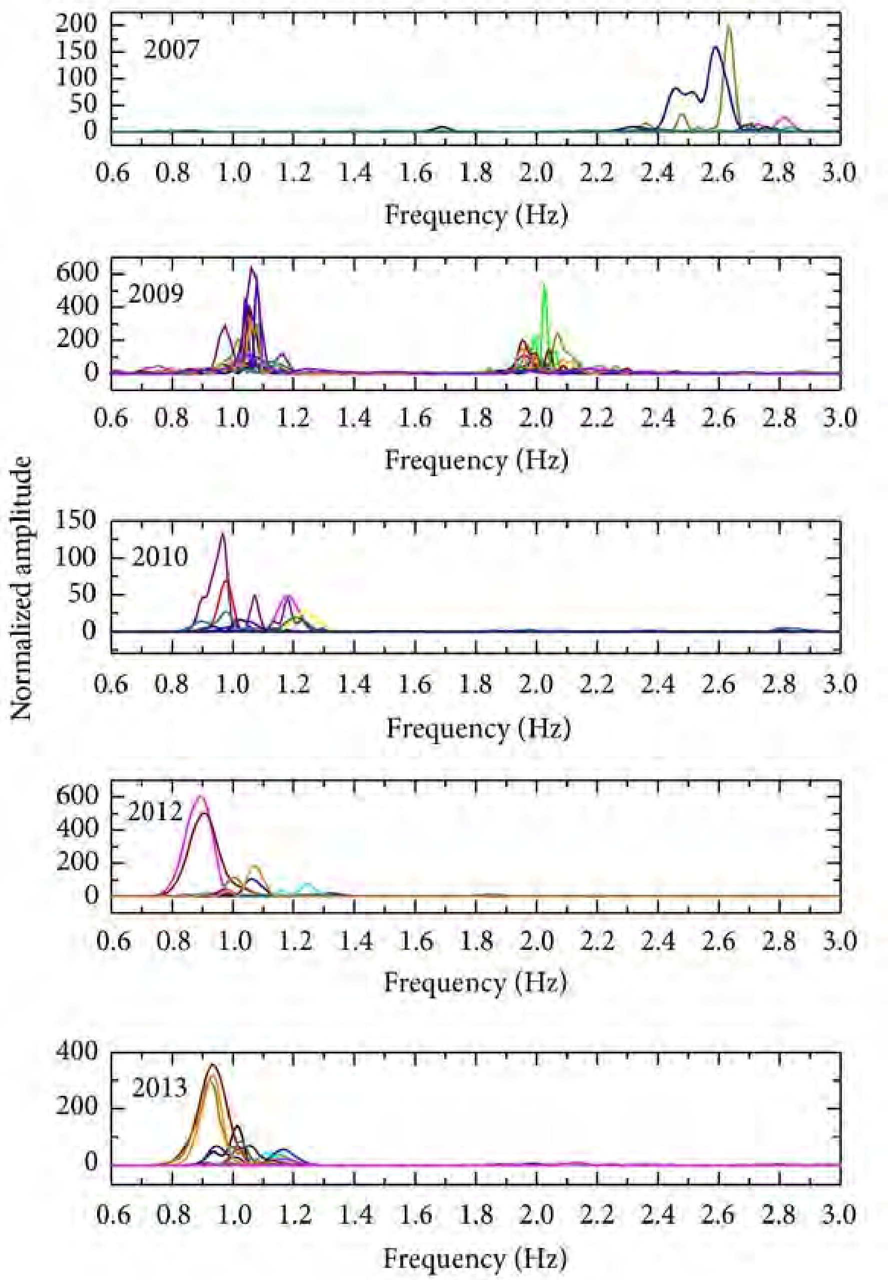

Figure 3: *Normalized spectra of acceleration along the lateral axis of the bridge, 2007-2013. A striking difference between the first and later surveys is evident. Colours indicate different excitation events. After [7].*

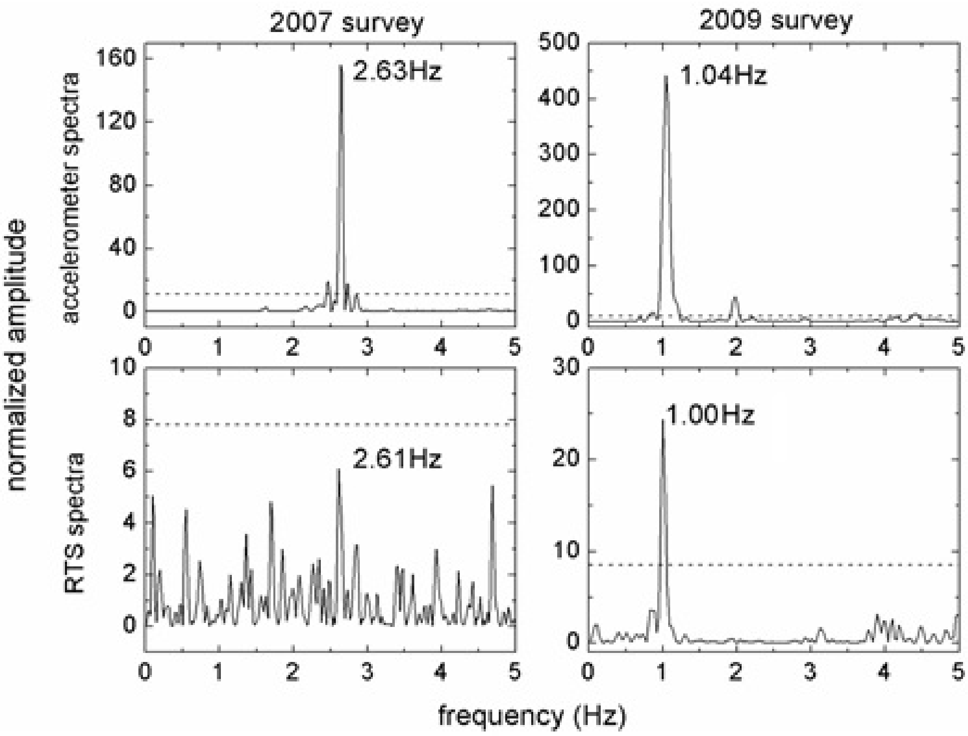

Figure 4: *Comparison of spectra from RTS and accelerometer measurements for selected events of the 2007 and 2009 surveys. A dramatic shift of about 1.6Hz for the dominant lateral frequency is evident. RTS and accelerometer spectra are consistent, but the 2007 RTS spectra are noisy.*

4 Discussion

All surveys were made using a standard excitation method, and for this reason comparative spectra were expected. Still, measured dynamic deflections during the 2007 survey were small, close to the detection level of RTS (some millimeters for field data [12]) and for this reason the corresponding spectrum of Fig 2 is noisy and very different from those of the subsequent surveys during which larger deflections with similar excitations were obtained. This result is reliable, because is corroborated by accelerometer spectra for the same excitation events (Fig. 3). In addition, it is confirmed by a comparison of the RTS and accelerometer spectra for the 2007 and 2009 surveys (Fig. 4).

These results indicate a dramatic permanent change in the stiffness of the Kanellopoulos footbridge between 2007 and 2008/2009, about 1.6Hz, and then an additional gradual loss of the bridge stiffness along the vertical axis. This change is summarized in Fig 5 and is systematic, not reflecting difference in loading and in ambient conditions [8]. Damage between 2007 and 2008/2009 and then gradual decay can only be the explanation for such changes in the natural frequencies of the bridge, consistent with signs of decay, increasing every year [8], and sensed by pedestrians who tend to avoid the bridge in the last years.

During the critical period 2007 to 2008/2009 occurred two potentially damaging unusual effects, a Mw6.4 earthquake in June 2008 and a flash icing event in February 2008 [8]. This last event was extraordinary for the area, which is usually free of snow, and led to massive destruction of water pipes and water tanks in solar panels in Patras. The explanation proposed is that the footbridge was somewhat structurally defective (see above and [8]), and a combination of excessive shaking by winds and the earthquake with freezing of the water inside joints produced damage and fatigue, which was reflected in the dramatic loss of stiffness in the lateral axis, recorded by the 2007 and 2008/2009 surveys. Continuing decay since then is reflected in the nearly linear drop of natural frequencies (Fig. 5).

An explanation for this decay is that damage initiated by the flash icing event brought the structure to a modal frequency close to that induced by human walking [16-18] and perhaps to wind gusts, and hence exposed the bridge to resonance effects which intensified damage. The total absence of repair and conservation of the wood certainly played a very important role. For these reasons the bridge is currently usually avoided by pedestrians, although it remains stiff along its vertical axis (unpublished data).

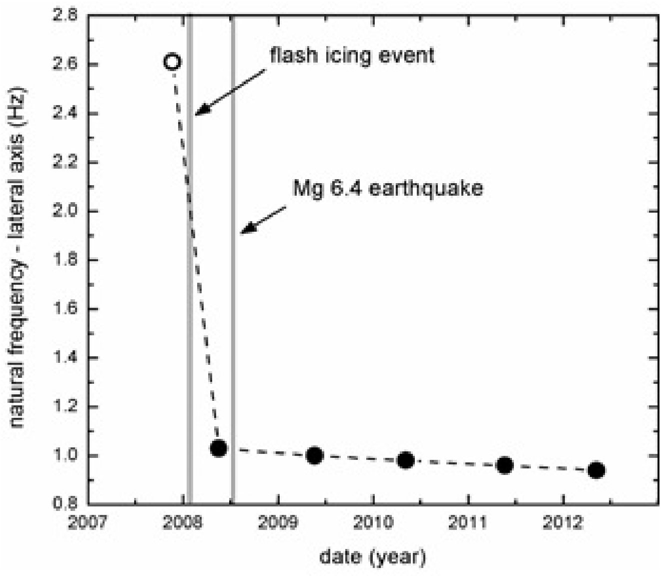

Figure 5: Changes in the dominant frequencies of the bridge in the lateral axis, 2007-2012.

5 Conclusions and future work

The Kanellopoulos footbridge represents a perhaps unique example of a structure systematically monitored for deflections and accelerations for several years. These surveys permitted to document its time-varying nature from the structural point of view, and record phases of probably episodic and of gradual damage and decay.

Data presented come from a few sensors describing lateral deflections and accelerations. Data describing vertical deflections and accelerations are also available and are in the processing and analysis stage. These data show that the bridge is very stiff along the vertical axis (dominant frequency of the order of 6.5Hz), but a frequency drop of 0.4Hz was also documented along this axis during the critical period of 2007 to 2008/2009.

The above results confirm the efficiency of geodetic instrumentation, combined with accelerometers, to monitor the structural health of structures, including timber structures and permit non-destructive structural quality tests. The perspective, however, is to combine monitoring with studies of the quality and degradation of the materials, as well as of studies for assemblage of structural members, and of their repair and conservation.

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Health assessment of a pedestrian glued laminated timber bridge

Helena Cruz [1]

Summary

The proposed paper presents the results of the health assessment of a 26m span pedestrian timber bridge located in the south of Portugal (Figure 1). This truss bridge has been in use for about 10 years and the assessment was carried out with the main purpose to check the preservation conditions of the European whitewood glued laminated timber members.

Proposed corrective measures will also be discussed.

1 Results

The information collected from visual inspection was analysed in the light of the structural design and joints detailing, helping to distinguish between delamination of the glued surfaces due to weathering and a crack due to tensile stresses perpendicular to the grain (Figure 2).

Figure 1: Pedestrian bridge

Figure 2: Crack of the timber member

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Assessment of Timber Bridges in Slovenia

Tomaž Pazlar[1], Miha Kramar[2]

Summary

This paper is focusing on the assessment of timber bridges which are – regardless to the high forestation territory percentage – very rare in Slovenia. Since many of timber bridges have already been replaced with structures made of other materials, the importance of preservation of the remaining ones is evident. In the last 30 years the Section for Timber Structures of the Slovenian National Building and Civil Engineering Institute has participated in the assessment of several (historic) timber bridge structures. The approach and technique used are presented in this paper on the basis of typical assessment examples.

Keywords: timber bridges, assessment, visual inspection, NDT testing.

1 Introduction

Timber bridges can be considered as one of the oldest structures in the human history. The capability of timber to withstand the bending loads was recognized by the human race in the Stone Age. In Slovenia, the remains of timber bridges in the simple form, can be found in the wetlands near Ljubljana, which date in the Neolithic period. The basic structural system, logs, crossing single or multi spans, has not changed. Only the logs were replaced by the bruted and then by the sawn timber.

Most of the existing bridges were constructed in the beginning or in mid of 20th century as a replacement for boats used for crossing the rivers [1]. Hardwood species (oak) or more resistant softwood species (larch) were used as bending and compression members. Typical timber bridge from that period is presented in Figure 1.

Figure 1: Timber bridge over Krka river

Figure 2: Timber bridge over Sava river in Vikrče

Unfortunately, many of the significant timber bridges built in the 20th century today exist only in memories and in the photo documentation. 85 m covered timber bridge over Kokra river in Kranj for example was built in 1938, but it was replaced after 30 years with concrete arch bridge. As a reason for the replacement, the damaged supports and poor maintenance was indicated. As it turned out, the structure was not in such bad shape, because the blasting with dynamite was not successful and they had to set fire afterwards [1].

Many of the timber bridges were damaged or destroyed by the flooding in early 1990. Most commonly other material, preferably concrete, was used for the substitute structures. Luckily, the largest covered timber foot bridge form 1934 over Sava River in Vikrče near Ljubljana managed to withstand the flooding (Figure 2). Although the load bearing structure of the bridge is timber truss hanging on the steel cables, the bridge is often presented as an exemplary case of timber engineering.

The brief overview of the timber bridge history reveals the contradictory conclusions: timber has been used in construction for centuries and several maintained and preserved structures demonstrate its durability very well. However, the timber is biodegradable and due to numerous reasons like poor detailing and poor maintenance the capacity of structural timber elements is decreased and many of structures do not meet their life expectancy. In order to identify decay and other deterioration, and consequently minimize the risk of collapse, the timber struc-

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tures should be – in a certain period of time – a subject of a thorough inspection which should be an important aspect in deciding among several alternatives in the building lifecycle. In general the result of assessment should give information whether it is from structural point of view possible to preserve existing structure – with replacement or re-strengthening of deteriorated members – or should it be decommissioned completely.

2 Assessment of bridges in Slovenia

Slovenian National Building and Civil Engineering Institute (ZAG) is from itse beginnings deeply involved in the assessment of bridges. Bridge inspection procedures were developed at ZAG in 1990 setting up a systematic approach for assessment of all kind of bridge structures [2]. The established methodology consists of the general description of the structure and furthermore describing the observed damage by the following parameters:

- Structural member on which the damage was detected.
- Detailed information about the location of the structural member.
- Type of damage.
- Area within the structure where the damage is located: pier, beam, in waterbed etc. or description of the damaged area.
- Location of damage in longitudinal and lateral directions according to the bridge axis and its altitude.
- Degree of damage.
- Extension of damage.
- Proposed remedial measures.
- General findings.

Each parameter of inspection is associated with a letter or numerical code, which minimises influence of inspector-specific style of reporting and quantifies the damage and is used to calculate the condition ratings of the bridge.

To be able to describe the structure with numerical code, the bridges are divided in the following main parts: bridge surrounding, riverbed, foundations, piers/abutments, bearings, superstructure, roadway, tunnel, expansion joints, safety features – signalisation – pipelines/installations and drainage equipment. Each main part is further divided in more detailed part of the structure. Superstructure, for example, is further divided into slab, main girder/beam, secondary bearing elements, box girder, cross beam, arch, vault, truss and sidewalk structure.

Damage that appears on structural elements of a bridge is denoted with a 4-digit code, with the first 2 digits representing the damage header: faults in behaviour of the structure, damages and flaws in materials and damages and flaws in structural elements. Damage can be described even more precisely. For example, damage in material – steel can be defined as mechanical damage, mechanical damage of protective coating, corrosion of structural steel, crack, missing, or rotation of node. The location of damage, degree of damage and size or extension of damage also has to be given. The remedial measures also have to be defined based on the degree of the damage and its consequences.

The damage is furthermore numerically evaluated and weighted with different factors: K_1 as element factor which identifies the effect of the damage in relation to where it appears, K_2 as damage intensity which describes the stage in which the damage is, K_3 which describes the damage extent, and K_4 as the emergency factor describing necessity of remedial actions (not urgent, not later than in 5 years, immediately, urgent – danger of collapse).

The methodology was updated through the years as well as software support toll called ebridge (Figure 3), which is today available as internet application - all bridge inspection reports are stored in a mySQL database on a web server. The application allows:

- accessing the software and the database from any computer and from any location (also on bridge itself)
- providing photos of the damages linked to the damage descriptions,
- quantifying damages using the damage rating indices,
- providing damage ratings, in numerical and graphical forms, for the main parts of the structure (substructure, super-structure, riverbed & surrounding and pavement & equipment) and the bridge as a whole, to allow bridge ranking based on the degree of deterioration.

Figure 3: ebridge software

The inspection summary consists of ratings of parts of the structure (substructure, superstructure, pavement and equipment) and of rating of structure as a whole. Codified bridge condition is also given in 1 to 5 scale.

The section "description" is dedicated to the bridge owner/manager who has to obtain, based on the detailed report, a clear, reliable and comprehensive picture about the bridge condition. The bridge inspector must in this short, free text form emphasise the most characteristic damages on the bridge, particularly those that are jeopardising structural safety and durability, and thus require urgent actions.

The section remedial measures summarizes the actions, proposed within the description of individual damages, and that are not linked to a specific damage but proposed by the inspection as a general measure. Remedial measures are grouped into: general measures, standard repairs, with estimate of quantities and the deadlines to fulfil them, urgent repairs and non-urgent repairs.

Unfortunately this methodology originates from the concrete bridges assessment and therefore the numerical ranging of timber bridges does not always reflect the actual condition of the specific timber structure. Therefore, a special attention is needed when applying the K factors in order to define reasonable ranking of the timber based bridges.

Presented methodology can be used for all kind of assessments: regular (2 years), main (6 years), special assessments, detailed, zero, and assessments before expiry of the warranty period. The main advantage of presented methodology is with periodic inspections where trends of evolution can be identified on the levels of structural elements, entire structures or on the bridge network level.

3 Periodic assessment of timber bridges

For many years Slovenian National Building and Civil Engineering Institute performed periodical bridge inspections for Slovenian Roads Agency (DRSC). Systematic period inspections gave a unique opportunity to observe at least part of bridge lifecycle - most of the timber bridges were built long before DRSC started with periodic inspections. Since ZAG is currently performing the assessments of bridges only on motorways, the assessment presented in this paper is based on the records dated from 1994 to 2004 [3,4]. Due to the limited scope only the general description of the assessment is given. The assessment report is – as described in previous chapter - more profoundly identifying deteriorated members, their location, intensity, extent and necessity for remedial actions.

The majority of preserved traditional timber bridges which are today protected as urban cultural heritage are located in the south of Slovenia in the river Krka basin. Two in Slovenia well known oak pile bent timber bridges are presented – they both cross the river to island town of Kostanjevica na Krki. Regular periodic inspections of timber bridges in discussed time period were performed as visual inspection combined with simple acoustic emission.

The periodic inspection of bridges using presented methodology begins with the records of general information about inspection (like date, weather condition, responsible persons). Preformed (or skipped) maintenance works, suggested at the previous assessment together have to be recorded. Additional extraordinary events and reading on installed measuring devices also have to be recorded.

Figure 4: Kostanjevica na Krki: north bridge

Figure 5: Kostanjevica na Krki: south bridge

Figure 6: Trucks overloading the bridge.

Figure 7: Corrosion of steel and cracks in stone pillar

The methodology of assessment is focused on structure itself and not on inspection procedure. Different persons/bridge inspectors have different approaches, but in order to avoid errors it is advised to use the same approach all the time. Inspection of bridges commonly begins with examination of approaches to the structure. The dilatation profile installed at both ends is commonly a problematic member and in most cases needs to be at least retightened (Figure 10). Asphalt on approaches is commonly crushed as possible consequence of overloading with trucks (Figure 6) and road structure (or soil) displacements. Pillars of both assessed bridges are made of stone – cracks with falling out mortar are commonly present (Figure 7). Hollow sound indicates that cracks are present not only on surface. In some cases even falling out stones can be found. River banks in the area of stone pillars are not well maintained. In general they are overgrown and used as bio-waste disposal, but there are also some spots of erosion originating from improper draining from bridge.

Foundation is in regular assessments evaluated visually. If necessary, inspection by diving has to be performed.

Piles were in relatively good shape – regardless to the river debris which can be found at almost each inspection (Figure 9). There were some bracing elements identified which have to be replaced. Steel connection elements (bolts, and clamps in deck structure) were corroded and often have to be retightened. The level of deterioration of pile caps is commonly relatively high (Figure 8). This might (also) origin from bad connection detail of bridge railing support. Pile caps are consequently under fungi attack (Figure 8). The same conclusion is valid also for the main bridge beams and for the lower deck (Figure 12). Elements of upper deck commonly have to be retightened, (Figure 13). Bridge railing commonly presents relatively deteriorated member, some parts may also be missing (Figure 11). Minor cracks were also identified in main structural elements (main bridge beams), but these fissures mainly originate from drying process.

Figure 8: Deteriorated bent cap / railing support

Figure 9: Deteriorated railing and river debris

Figure 10: Dilatation profile needs retightening

Figure 11: Part of railing is missing

Figure 12: Fungi attack - main beams, lower deck

Figure 13: Deck planks need to be re-fixed

The inspection summary rates the structure with numerical value 23.12 and with codified bridge condition as 3 (satisfying condition). The assessment report concludes that only urgent maintenance work was done since the previous regular assessment. Report also lists the standardized measures (fixation of dilatation profile, fixation of deck planks, and reparation of railing, removing the river debris) and urgent repair / repair with deadline (rehabilitation of pillars). The overloading is exposed as a special measure which should urgently be dealt with.

4 Detailed assessment of timber bridges

In general, timber bridges in Slovenia are not maintained properly. Many property owners/managers are aware of this fact when the road authorities decide the set the limit for heavy traffic over the bridge or even close the bridge. Unfortunately this is most common cause for the assessment. A comprehensive rehabilitation of timber bridges is reasonable to expect in such circumstances. Most commonly the property owners/managers beside the general assessment conclusions also expect detailed list of timber members that have to be replaced in order to be able to prepare public procurement – exact as possible.

In such cases the detailed assessment has to be performed. Although general principles of assessment are the same as with periodic assessment, there is evident difference in level of details. It is important that level of detail is agreed between parties involved. Detailed assessment can be time consuming and the property owners/managers often cannot accept the cost estimation of the assessment or even worse – in some cases they are considering it only as an additional cost.

Since the purpose of detailed assessment differs from the periodic assessments, most commonly the descriptive approach is used instead of using the numerical evaluation. Most of the timber bridges are managed by local communities and persons responsible are not familiar with the codified system used.

Some researches claim that only 30 % of damage in timber structures can be identified by visual inspection [5]. Therefore, it is necessary to combine the different non-destructive methods: simple acoustic method, core drilling and simplified resistance drilling using small diameter drill and regular drilling machine. Although more exact NDT methods exist, we are still trying to implement them in timber bridge assessment.

The result of detailed inspection is a report including the detailed description of damaged elements that need to be replaced or their graphical representation [6]. Although the structural system is similar to the previously presented assessment case, the deterioration level with the second assessment case, timber bridge over Krka river in Loke, is much higher (Figures 14-17).

Figure 14: timber bridge over Krka river in Loke

Figure 15: Deteriorated main beam

Figure 16: Connection detail

Figure 17: Reduced cross section of pillars

Figure 18: Deteriorated tie beam

Figure 19: Corosion of steel tie elements

Bridge was assessed in 2010 and renovated in 2011 by taking into consideration the assessment findings. Unfortunately due to the cultural heritage protection not all poor details could be improved in order to extend the bridge life expectancy.

5 Conclusions

Traditional timber bridge structures can be considered as simple structural systems – the load bearing mechanisms and connection details are relatively simple. Consequently, possible points of deterioration can be relatively easy to predict, but they are not always accessible. If there is a reasonable doubt about the uncertainty of deterioration level, then – similar as with other assessment cases – the conservative decisions have to be made.

In most cases, the assessment concludes that deficiencies identified originate in bad detailing and poor maintenance. It is clear that authorities which are responsible for the bridge maintenance do not always except the fact that most of the presented type of bridges require more maintenance work than comparable concrete bridges.

Since 1990 the timber has been used again for some bridge structures in Slovenia, mainly in foot bridge construction. The misconception that timber provides a short service life and that it should be used only for temporary structures was at least partly ruled out. Glulam offers wide range of shapes / lengths and material protection techniques were also improved. However, the need for (periodic) assessment will always exist and due to the complexity of modern structures the implementation of more sophisticated NDT methods is probably a must in order to improve the assessment procedure and results.

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State of conservation of unprotected timber footbridges in Central/Northern Italy

Alberto Cavalli[1], *Marco Esposito*[2], *Marco Togni*[3]

Summary

12 timber foot-bridges, different for age, dimensions, structure, exposition, were investigated and monitored by means of the visual inspection, to check their structural health and state of conservation. The bridges were unprotected against weathering; all of them have been directly exposed to weather. The bridges were studied and then subjected to a visual inspection for assessing their state of conservation and for verifying the design details in relation to the correct governing of moisture. The visual inspections revealed some important lack in the durability design, leading to premature wood-decay problems in the 100% of the cases, namely the water "traps" due to contact surfaces between timber members or steel-to-timber and the horizontal timber elements with no physical protection. Without a maintenance program and the repetition of the visual inspection, the shortening of the service life can be avoided only with some accurate remote long-term monitoring operations.

Key words: design for durability, visual inspection, service life, remote monitoring.

1 Introduction

In Italy, during the last twenty years, many timber bridges have been built, due to the renewed interest on wood, because of environmentally friendly and, into natural landscapes, particularly respectful. Wooden pedestrian bridges in Italy are typically short span bridges; rarely, dimensions can be relevant. Most part of recent bridges are directly exposed to weather because uncovered. For Eurocode 5 [1] it means that structure component have to be assigned to the service class n. 3. In other Countries, where timber structures were used extensively also for vehicle bridges, e.g. in U.S.A. [2], there is also an attention to avoid, if possible, directly exposed parts, designing and building mainly covered bridges [3]. An uncovered bridge would last shortly, especially if the design for durability is poor and it has not been able to avoid moisture problems and subsequent biological degradation [4]. Differently the use of more durable material is necessary: naturally durable timber (e.g. hardwoods in Durability classes n. 1 or 2 [5], or softwoods in class 3 [5]), treated timber (by means of deep impregnation with chemical preservatives), or recent products with a conferred durability by means of industrial processes (e.g. Accoya®, Kebony© etc.), can increase the service life, although for a limited time [6]. Where natural wood is used, surface treatments, differently from pressure treatments, are not enough to change the wood properties and to better the characteristic durability. To preserve the structures from a fast ageing and from failure/rupture, a planned maintenance schedule is needed ([2],[3],[4],[7],[8],[10]) and sometime restoring interventions [9],[10] cannot be avoided, bearing in mind that if a wooden bridge has been neglected too long, it could not be good candidate for rehabilitation [6],[11].

The study concerns 12 pedestrian timber bridges, without any covering wooden boards, (subjected to a periodic change), or with limited metal flashings, but no roof. The aims of the paper are:
- checking of their conservation conditions;
- assessing of the eventual decay level;
- verifying the design details in relation to the correct governing of moisture.

2 Material and methods

The 12 footbridges are located in Central and Northern Italy, in three different Region: Emilia-Romagna, Tuscany and Trentino. The bridges are different for building year, dimensions, shape of the structure, exposition, latitude and altitude. The most important characteristics and the images are listed in Table 1. The wooden structural components are made of conifers glulam beams, namely made with Norway spruce and/or silver fir (*Picea abies* Karst. and/or *Abies alba* Mill.). The bridges are on a stream or a river, in two cases only, on road.

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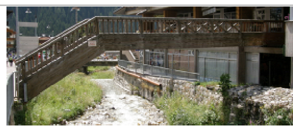
Table 1 – Timber footbridges location, functions, dimensions and age of building (span: S=small <20m, M=medium 20m<X<60m, L=large >60m, age of building: R=recent <10 years, A=aged >10 years).

N	***place: municipality and Region ( )***	***geographic coordinates and altitude***	***crossing over on a***	***span***	***age***	***shape snapshot***
1	Bedonia (Emilia-Romagna)	44°30′06″ N 9°37′49″ E 544 m	river	S	A	
2	Barberino di Mugello (Tuscany)	43°59′23″N 11°14′42″E 270 m	river	M	R	
3	Campitello di Fassa (Trentino)	46°28′37″N 11°44′29″E 1448 m	river	M	A	
4	Cerreto Guidi (Tuscany)	43°45′42″N 10°52′37″E 38 m	river	S	R	
5	Empoli (Tuscany)	43°42′52″N 10°55′20″E 28 m	road	M	R	
6	Fontanazzo – Mazzin (Trentino)	46°28′04″N 11°43′52″E 1372 m	river	M	A	
7	Fucecchio (Tuscany)	43°43′14″N 10°48′45″E 25 m	river	S	R	
8	Lamporecchio (Tuscany)	43°49′00″N 10°54′00″E 56 m	road	S	R	
9	Milano Marittima – Cervia (Emilia-Romagna)	44°16′41″N 12°20′53″E 7 m	river	M	A	
10	Rimini (Emilia-Romagna)	44°03′54″N 12°33′04″E 10 m	river	L	A	
11	San Giovanni alla Vena - Vicopisano (Tuscany)	43°41′02″ N 10°34′14″E 11 m	river	S	R	
12	Vicopisano (Tuscany)	43°41′00″N 10°35′00″E 12 m	river	S	R	

To outline the main characteristics of the bridge their spans are divided in 3 clusters: small (S) for spans shorter than 20 meters, medium (M) for spans between 20 and 60 meters, and large (L) where the spans overcome the 60 meters. Only 1 bridge has a "large" span: the pedestrian bridge n. 10, characterised by a 90 m span. Bridges were differently grouped for age: bridges inspected within 10 years from the construction are marked as recent (R) and the other one as aged (A).

The bridges were investigated by means of the visual inspection [12], with the aim to check their structural health and the state of conservation. In some cases the inspections were repeated with the purpose to keep the structures monitored. The on-site inspection is the core strategy for a correct and complete examination [13]; it was carried out by means of simple tools (hammer, gimlets, screwdrivers, etc.) without employing any instrumental tests (e.g. static/dynamic hardness, free/forced vibration frequency, ultrasounds speed, etc.). Purposes of the examinations were the coarse estimation of the general health of the structures; the precise and punctual assessment of each timber member and joint, based on non-destructive devices, is not the goal of this paper.

For each bridge some important aspects concerning the durability were highlighted, linking, where possible and visible, effects to causes, and, concerning the conservation assessment of the wooden structure, associating context information (environment, functions, etc.) with the outcomes of the on-site inspection.

3 Results and discussion

The visual inspections revealed some important lack in the durability design, considering that the bridges are uncovered. Summarising results are shown in Table n. 2 and 3.

The 100% of studied bridges is characterised by some water "traps", which could be better defined as moisture "traps", i.e. some wooden surfaces which can be wetted by free water (rainfall) but that cannot dry quickly because not airy. Moisture "traps" found on the bridges were different for dimensions, water collection, moisture holding and so for the potential risk of decay by fungi and future impacts. The moisture “traps” in the joints were the most dangerous, while the contact surfaces between beams could be less critical (easier drying), but also to be avoided. None of the investigated structures was designed considering the possibility to build airy connections. The contact surfaces between steel plates and timber, to be considered as dangerous moisture “traps” in rainfall exposition, influenced by the holes containing the mechanical connection (bolts, pin, etc.), often cannot be avoided or reduced because contact is needed for load carrying. Water wets all the contact surfaces and fills the gaps. The high moisture content is maintained for long time, in absence of ventilation and because the metal plates or connectors are not porous, so they do not allow water to move and to escape. Year after year this condition makes certain the degradation by fungi.

10 bridges (83%) were built with some horizontal structural members, not covered: without metal flashings or other protections. Decks are not considered in the visual inspections, because typically they are subjected to a periodic replacement. 2 bridges (n. 4 and 5) are built with curved glulam beams, so really there are not horizontal surfaces on the main structural elements, although the small slope cannot be considered enough to avoid stagnation, and not so safe against fungi, for the long period.

Figure 1: Timber-bridge n. 9. Fruiting bodies of brown-rot fungi, close to moisture trap (hole, bolt and washer).

5 bridges (1, 3, 6, 9, 10 - 42% of the total) were already showing a decay by brown-rot fungi; concerning the diffusion on timber members, decay was localised only in some specific timber members in bridges 1, 6 and 10, but with important local wood degradation and some easily visible fruiting bodies (Table 2, decay column, bridges n. 1, 6 and 10), showing the good development of the alteration organisms, while in bridges 3 and 9 the degradation was very severe and diffuse. All the 7 remaining footbridges were recent (R in Table 1) and so no sign of decayed wood was present. Nevertheless it has to be noted that the low durability of the wood of the beams (Norway spruce and silver fir are in the Durability class n. 4= low durable [5]) to the fungi alterations, expose them to high risk of decay: we can hypothesize that the decay will appear soon.

Table 2 – Exemplification of the main problems discovered on timber footbridges and relative images. Selected bridges characterised by incoming decay.

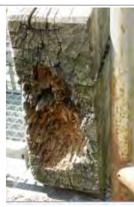
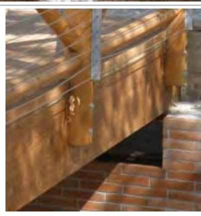
N.	***uncovered horizontal surfaces in load-bearing glulam components***	***moisture traps***	***decay: diffuse in* 3 e 9 *localised in* 1, 6 and 10**
1			
3			
6			
9			
10			

Table 3 – Exemplification of the main problems discovered on timber footbridges and relative images. Selected bridges without decay.

N.	***uncovered horizontal surfaces in load-bearing glulam components***	***moisture traps***	***decay:*** ***expected* [E] (2, 4, 5, 7, 8, 11, 12);** ***unexpected (none)***
2			E
4	None, excepted deck (extrados covered by metal flashing)	 	E
5	None, excepted deck (extrados covered by planks)	 *[a]*	E
7			E
8			E
11			E
12 - Vicopisano (Tuscany)		 *[b]*	E

[a] The deck, made of planks, is open to rain: water can come down along the surface of the structural timber member.

[b] The bridge is characterised also by a design and mounting error. The main horizontal beams are notched and consequently broken longitudinally for the tension perpendicular to the grain.

4 Conclusion and future work

The study on twelve pedestrian bridges in this work shows the importance of the “design for durability” on timber load-carrying structures without shelter, for their longer duration. According to the results, the primarily required action is the visual inspection, which allows to identify possible problems: incorrect details design, water traps, poor ventilation, incipient decay etc., in an easy and cheap way.

The use of not naturally durable timber species and the lack in design and maintenance caused a premature ageing of the structures, determining the poor state of conservation for all the bridges older than 10 years at the time of inspection: evident decay due to brown-rot activity were found in correspondence of moisture traps and horizontal members not protected.

For the other bridges, presenting similar design, but with age less than 10 years, the decay could be present at early stages (still not detectable) and it will be expected soon.

To face hardly the lack of the “design for durability”, a long time monitoring program is needed. Such a program can be intended as the repetition of the visual inspection: some periodic assessments of the bridge for the control of its state of conservation. Differently a remote monitoring" (RM) [14], is to be set for checking some conditions of the timber components: principally moisture content, for preventing decay, and deformations, to assess possible overloads or malfunctioning on structural joints.

The results of RM may guide the maintenance operations and, if need, the restoring interventions. The monitoring can be also used as a real time system to alert, in case of risk, for structures and people. Although RM programs could seem an innovative approach for the footbridges, the first monitoring experiences on timber bridges have been developed during the nineties in North America by the *"Timber Bridge Initiative"* for the evaluation of structural health of those structures [15]. The Forest Product Laboratory developed some remote monitoring systems to be adaptable for different kinds of timber bridges. Many solutions of RM are still adopted for the monitoring of bridges [16],[17],[18],[19],[20], for the assessment of their safety and structural reliability.

The monitoring is the only system for giving the possibility to check continuously the state of conservation and to prevent some irreversible decay of wood/ timber members/joints, by means of specific interventions, promptly guided by the data collected from repeated visual inspections or continuous sensors monitoring.

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Moisture monitoring of beam and pylon in a timber bridge

Anna Pousette[1], Anders Gustafsson[2], Per-Anders Fjellström[3]

Summary

A 130 meter long pedestrian and bicycle bridge passing over the river in Skellefteå was built as a cable stayed bridge. Long-term monitoring of moisture and temperature in wood was performed with wireless sensors from the time when the bridge was opened. This paper presents moisture conditions in the timber beam and pylon with comparisons between different monitoring points. Different levels of humidity can be expected in different parts of the bridge. The bridge works well, the moisture content is mostly below 20 %, with some peaks in winter. The moisture content was lowest and most stable on the outside of the beam where it was protected by cladding and at the top of the inside of the beam. This type of measurement can be used to verify that the construction is kept dry and also point out parts that are exposed to moisture and should be examined carefully during regular inspections.

Key words: timber bridge, cable-stayed bridge, wood, moisture, moisture monitoring, health monitoring system

1 Introduction

Many timber bridges have been built in Sweden during the past 20 years. In general, the health of bridges is assessed by visual inspections at regular intervals. Continuous measurements could complement the inspections to provide a basis for the scheduling of maintenance activities. A research project with the overall objective to develop monitoring tools for timber structures to guide the planning of maintenance was carried out in conjunction with the construction of a new bridge. The bridge is passing over a river and it was built as a cable-stayed bridge spanning 130 meters. It is a slender and complex structure and was an object where sensor systems could be installed during the construction of the bridge. In the late summer 2011 the bridge was taken into use, named "The Älvsbacka Bridge", see Figure 1.

Figure 1: The Älvsbacka Bridge

This advanced timber bridge gave an opportunity for testing assessment methods for timber structures and to study durability of timber bridges. Monitoring systems and wireless sensor networks were installed to test analysis tools and also measurement and data transmitting techniques. The bridge was monitored for temperature, moisture, movements, deflections and vibrations at different points enabling an analysis of the bridge health [1] [2]. Temperature, wind and rain are also measured at the site. Analysis of the bridge can also help increase the competitiveness of this type of timber bridges. Cable-stayed wooden bridges are often appropriate to span large distances, but the slender design can provide dynamic problems [3]. In addition to measurements, extensive dynamic analyses of the bridge have also been made [4] [5].

This paper presents the moisture measurements of beam and pylon in the bridge during the years 2012-2014. The moisture content of wood is in equilibrium with the relative humidity and can additionally be increased by rain and melting snow. Moisture content below 20-30% is a rule of thumb for when no fungal decay occurs in the wood that may lead to reduced strength and structural failure. Moisture content is consequently essential for

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durability of all wooden outdoor structures. Impregnated wood provides an additional security against decay, but in this case, the structural parts are made of untreated spruce. High moisture contents should however not occur if the bridge is properly built and drainage and claddings work as intended.

2 Material and method

This paper presents moisture data from a timber bridge over the river in Skellefteå in the north of Sweden. The bridge is a cable-stayed bridge with a span of 130 meters. Pylons at each end are 23 meter and made of glulam 900x900 mm^2. The width of the bridge is 4 meters between two main beams of glulam with a cross-section of 645x1100 mm^2. The beams are made of three beams glued together to one beam. The bridge deck is a plank deck of preservative treated pine (Pinus Sylvestris) on cross-beams.

The beams and pylons are painted with an opaque wood stain in a yellow colour. The pylons and the outside of the beams are covered with painted panels for weather protection. The wood of all painted parts is Norway spruce (Picea abies). The pylons are treated with one coat of paint (behind the painted cladding) and with an oil primer and one coat of paint on the ends. The beams are treated with one coat of paint on the outside (behind the painted cladding), and with an oil primer and two coats of paint on the bottom and inside surfaces and on the ends. The stain is called “Beckers Perfekt Täcklasyr” [6]. It is waterborne and based on an alkyd oil acrylate hybrid binder fortified with linseed oil, containing 10–25 wt% alkyd oil/linseed oil hybrid, 2.5-10 wt% acrylate co-polymer binder and 2.5-10 wt% linseed binder. The oil primer “Beckers Perfekt Grundolja” is an impregnating oil primer based on modified linseed oil containing 10-20 wt% alkyd oil/linseed oil hybrid that penetrates the wood.

Several sensor systems were installed in connection with the manufacturing and building of the bridge. The aim was a synchronized monitoring system to easily perform correlations between data from the sensors without any time shift. Remote access to data acquisition systems was established by an internet connection. The main objective of data from the health monitoring system was to verify the long-term behaviour of the bridge. Data have been collected since 2011 with some interruptions and moisture measurements are still working.

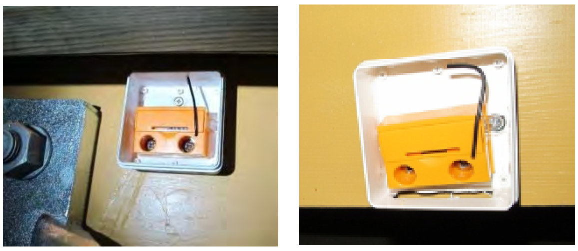
Long-term monitoring of moisture and temperature in wood was performed by SP Monitor with wireless sensors from Omnisense [7]. The wireless in situ measurement system for wood moisture has been used and tested in other projects [8]. Data from sensors were collected by a gateway. The sensors were activated once an hour and they sent data to the gateway placed in a control box at one of the foundations. The gateway delivered measurements to a database via connection to the Internet. The measured wood humidity values were then compensated for wood species and temperature.

The sensors and positions are presented in Table 1 and Figures 2-5. The sensors were mounted with screws and measure the highest moisture content from the surface and about 25 mm into the beam. The temperature and moisture content was registered every hour. From the beginning there was a conflict with another gateway in a nearby wooden house, see Figure 4. The gateways interfered with each other and there was no order in which data was sent to the database. This meant that since October 2012 there were no measurements on the bridge for a few hours each afternoon, when instead the system in the house could measure.

Table 1: Moisture monitoring points in the bridge

Point	**Position**	**Distance from foundation**
A.	Beam inside, top	7.2 m
B.	Beam inside, bottom	7.2 m
C.	Beam outside, bottom, behind cladding	12.6 m
D.	Pylon, halfway up, partly behind cladding	

Figure 2: Bridge from underside with sensors



Top (A) Bottom (B)

Figure 3: Moisture sensors on beam inside, (A) and (B)

Figure 4: Moisture sensor on beam outside, bottom (C)

Figure 5: Moisture sensor on pylon (D)

Temperature, wind and rain were measured at the site with a weather station situated at the top of the south-east pylon. Several climate parameters were measured, air temperature, relative humidity, wind speed and direction, barometric pressure, rainfall intensity and duration. The weather station was a Vaisala WXT 520 with the accuracy ±0.3 m/s wind speed, ±3° direction, ±3°C temperature, and ±3 % RH relative humidity [9]. A similar weather station was situated on the roof of SP Wood Technology about 900 m from the bridge in a westerly direction, see Figure 6. The altitude was about 6-10 m above ground and the distance to the river about 300 m. This weather station is used in this paper to show the climate data.

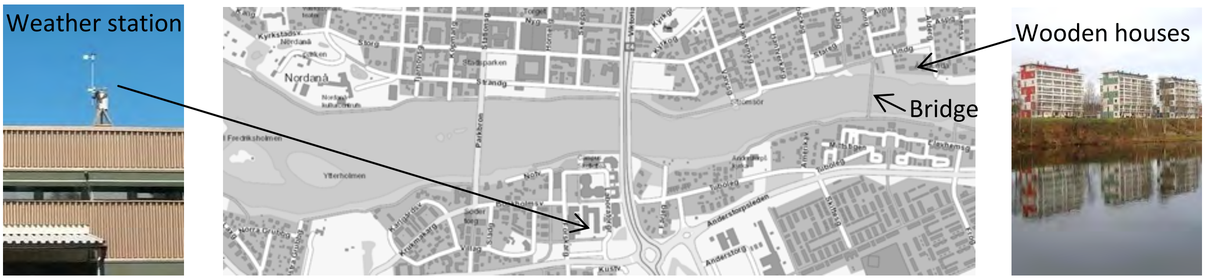


Figure 6: Location of bridge, weather station (at SP Wood Technolog)y and wooden houses

3 Results and discussion

At the bottom of the beam there are some differences between the moisture contents of inside and outside. The moisture content of the inside varied significantly between winter and summer season, see Table 2 and Figure 7. The MC at the bottom on the outside was quite stable during the year, only slightly higher during winter. The mean inside MC was 25.2 % in the winter period and 17.8 % in the summer period. The mean outside MC was 16.2 % in the winter months and 14 % during the summer months.

The outer sensor was protected by cladding. The inner sensor was not protected and the inside moisture content indicated that when the temperature was above freezing point in the winter, some snow thawed and water ran down on the inside of the beam and increased the moisture content. When moisture values increased dramatically during periods with many degrees below freezing, it could also be due to measurement errors because of condensation or snow on the sensor. High moisture content only for short periods in winter with low temperatures is however no risk of fungi growth. During the summer with higher temperatures the MC was under 20 %. Summer period is in Table 2 from 21 March, but also after this date there are sometimes cold and snowy winter days which is indicated by the high maximum value (37.6 %).

Table 2: Moisture content values in bottom of beam inside (B) and outside (C)

Point	Period	Mean (%)	Max. (%)	Min. (%)
B.	Winter (21 September to 20 March)	25.2	42.6	17.9
B.	Summer (21 March to 20 September)	17.8	37.6	15.3
C.	Winter (21 September to 20 March)	16.2	18.0	13.8
C.	Summer (21 March to 20 September)	14.0	16.1	12.3

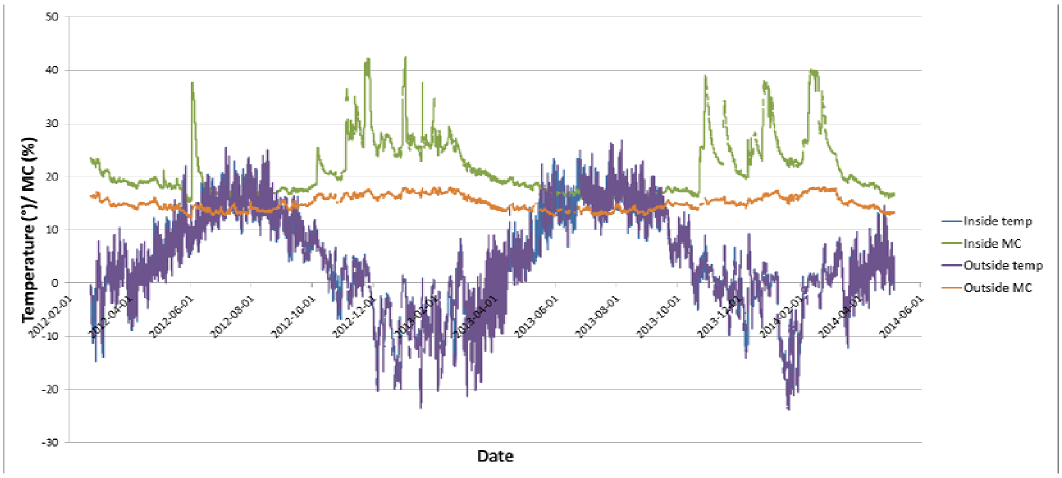


Figure 7: Temperature and moisture content in beam, inside bottom (B) and outside bottom (C)

There was a difference in temperature between the outside and inside of the beam, see Figure 8. Outside is toward the east where the sun rises early in the summer and in the morning the beam was warmer on the outside. During the summer season the temperature differed a lot, but in winter, the sun rises later and not far above the horizon and the temperature did not differ so much. The biggest difference was 7.9°C (outside warmer) and the lowest was -4.1°C (inside warmer). The temperature was measured outside the beam, not in the wood. These rapid temperature changes, however, did not affect the measured MC values in wood as the sensors registered the highest MC in the outermost 25 mm of the beam which did not change during these few hours, see Figure 9.

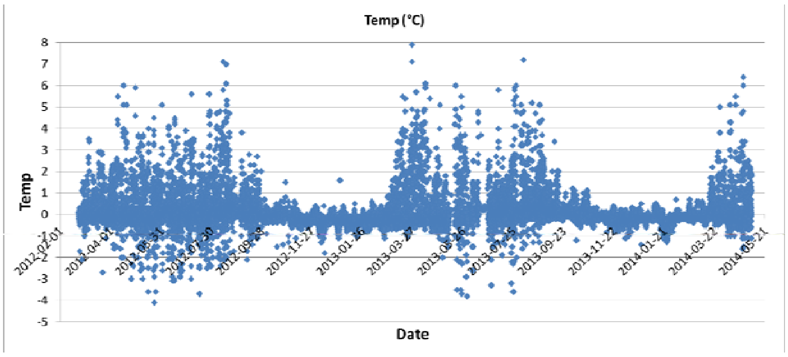
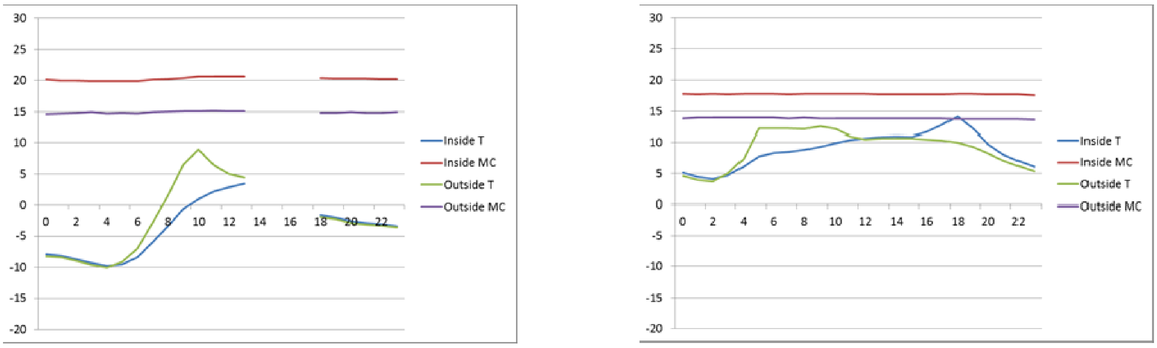


Figure 8: Temperature difference in beam between outside beam (C) and inside beam (B)



Max. difference 7.9°C (in the morning)
2013-03-26, sun rise 05:14, sun set 18:11

Min. difference -4.1°C (in the evening)
2012-05-22, sun rise 02:49, sun set 22:19

Figure 9: Max and min temperature difference in beam between outside beam (C) and inside beam (B)

The moisture content at the top of the beam on the inside was approximately 16.3 %, maximum 23.7 % and minimum14.5 %. There was a tendency of higher moisture levels in the winter, during the same periods as of the sensor in the bottom of the beam, see Figure 10. The temperature did not differ much between the top and bottom on the inside of the beam.

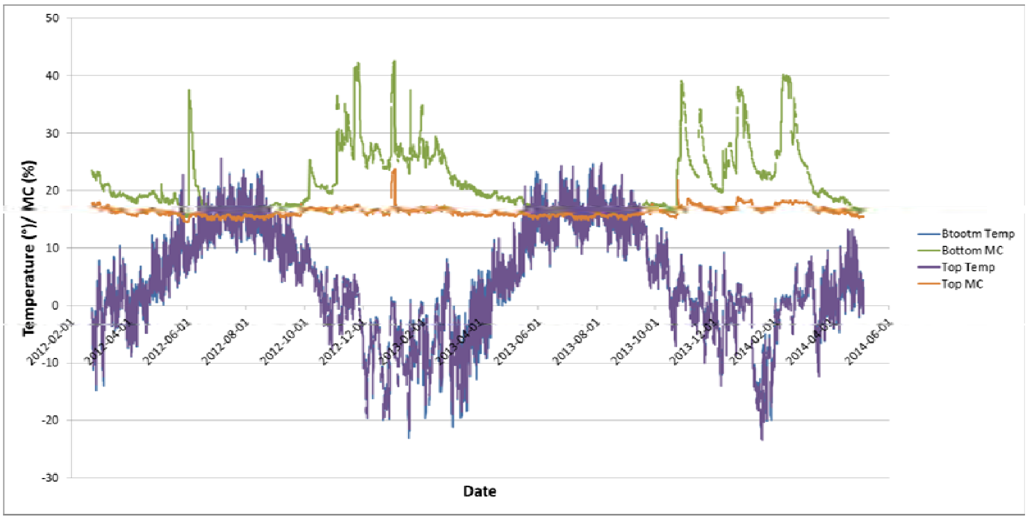


Figure 10: Temperature and moisture content on the inside of beam, at top (A) and at bottom (B)

There were some missing data for the pylon, see Figure 11. MC level of outside pylon was higher compared to outside beam, both protected by cladding. In the pylon the sensor was mounted close to the cross-beam at the hole in the cladding where the cross-beam was attached to pylon. The sensor was thus not entirely covered by cladding. This may explain the tendency of high MC from thawing snow or condensation in the winter period, likewise as for inside of beam at the bottom.

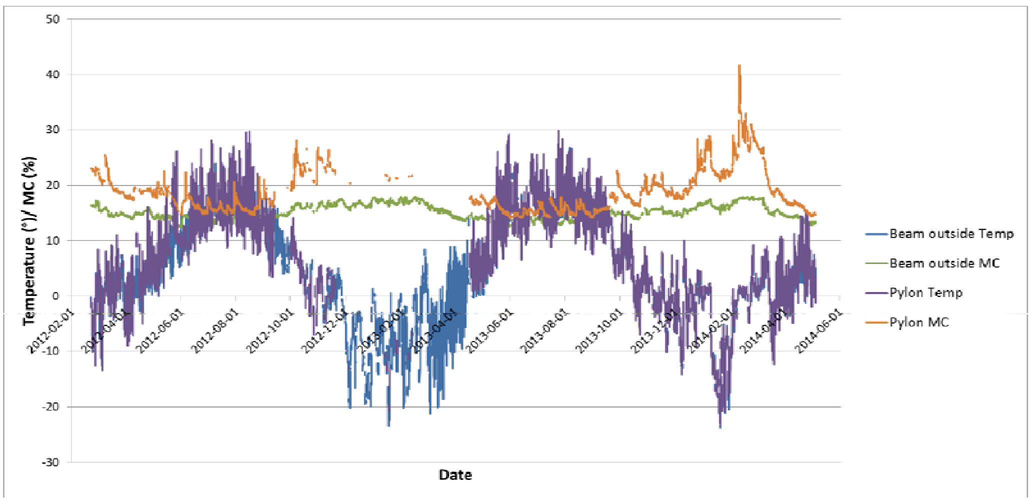


Figure 11: Temperature and moisture content outside beam (C) and pylon (D)

The weather station provided data for temperature and relative humidity (RH) of the air, see Figures 12-13. There was a difference in temperature and RH between day and night, especially in the summer, see Figure 13. The climate in Skellefteå is quite easy on outdoor wooden structures.

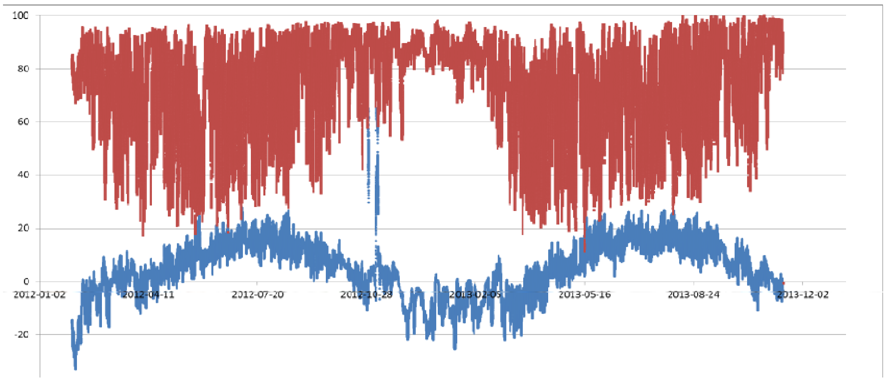


Figure 12: Temperature (blue) and relative humidity in the air (red) at the weather station, for two years

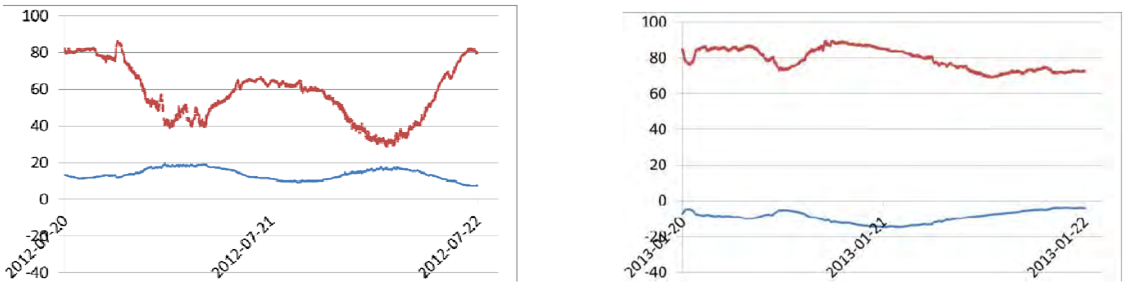


Figure 13: Temperature (blue) and relative humidity in the air (red) at the weather station, for two days in the summer and two days in the winter

4 Conclusion and future work

The sensors were not completely stable to give continuous readings. There were some missing measurements and several days were lost for some sensors. But a system that continuously measures during a long time will in all cases provide enough data to follow the changes of moisture content over the year. A monitoring system in strategic points with risk of moisture can help to observe the conditions and make actions to ensure the bridge. The measurements will continue, to get longer series of data and to study the life of the monitoring systems to ensure future use. The moisture sensors use batteries that will need replacement after some years. With difficult access to the underside of the bridge a long life and little maintenance is desired, and this is important to consider for the control of the bridge health.

This study showed that different levels of humidity can be expected in different parts of the bridge. The moisture content was lowest and most unchanging on the outside of the beam where it was protected by cladding and at the top of the inside of the beam. On the inside at the bottom there was not the same protection against frost and incoming water and the moisture content was higher and more variable. The inside is protected by a plank deck, which is not completely tight. There are a few millimeters wide gaps between the planks, and also a gap between the deck and the beam. There is a plate over the beam top surface which on the inside ends a few centimeters down along the beam with a drip edge. On the outside, the plate covers the cladding making the beam fully protected. For this type of construction with open decking, it may be appropriate to consider also a cladding on the inside of the beam.

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Hygro-thermal numerical models for stress-laminated timber decks

Stefania Fortino[1], Petr Hradil[2], Merja Sippola[3], Tomi Toratti[4]

Summary

In this paper FEM simulations based on the recent advances in hygro-thermal modelling of wood are carried out for the stress-laminated timber deck (SLTD) of a timber bridge exposed to Northern climates. The work compares the performance of two different FEM models that were implemented in Abaqus code and have been also used in earlier works. The first model is based on a simple single-Fickian approach which uses only the moisture content as a variable and the second one is a full coupled analysis based on the multi-Fickian theory with sorption hysteresis of wood. Both models were previously verified for cases of wooden specimens in laboratory conditions as well as for bridge elements under warm Mediterranean climates. The first results of hygro-thermal simulations of a Norwegian bridge deck provide useful suggestions for the future development of numerical models for timber bridge elements under Northern climates.

Key words: Stress-laminated timber deck, Moisture content, Monitoring, Hygro-thermal models, FEM.

1 Introduction

The monitoring of timber bridges exposed to natural climate fluctuations is important to understand their long-term performance, namely for durability, serviceability and safety. Due to their exposure to outdoor climates, timber bridges are strongly affected by the continuous variations of both relative humidity and temperature that can cause relevant peaks of moisture content as well as high moisture gradients inside the wood. Since wood is a hygroscopic and biodegradable material [1], the moisture content influences its physical, mechanical and rheological properties and its swelling and shrinkage behaviour [1-4]. The differences in shrinkage and swelling due to moisture gradients generate the so-called moisture induced stresses (MIS) that can initiate cracks in wooden elements during their service life [5-10].

The effects of European climates on the moisture gradient and MIS in timber components were numerically studied in [5]. That study pointed out that the yearly variations of relative humidity (*RH*) in Northern Europe (average values over 80%), cause higher moisture gradients compared to the Southern Europe variations of *RH*. However the analyzed structures were sheltered from rain and solar radiation. In [6] it was found that high daily humidity variations together with large changes of humidity at high levels of *RH* provide higher moisture gradients. This was confirmed by the numerical monitoring of a glulam beam of a pedestrian timber bridge in Lisbon [7] although the studied beam was not directly exposed to sun and rain. For timber bridge elements directly exposed to outdoor climatic conditions, the effects of solar radiation, rain and wind direction influence the fluctuation of *RH* and temperature and can produce significant moisture gradients in wood.

As discussed in [7] the use of numerical hygro-thermal methods can be useful, alongside with sensor techniques, for monitoring temperature and relative humidity in timber structures under natural climates. The main benefit is that numerical methods are able to simulate the moisture states in each point of a wooden member by using as external loads the relative humidity and temperature of the air provided by *in-situ* measurements or by a close meteorological station. This approach can be used as a support for the current techniques for monitoring the hygro-thermal response of timber bridges during their service life. This can be exploited to reduce the overall costs for monitoring in the context of modern maintenance plans.

In the present work FEM simulations based on recent hygro-thermal models for wood are proposed for the numerical monitoring of moisture content in stress-laminated timber decks (SLTD) of Norwegian timber bridges. As a case-study, the hygro-thermal behaviour of the deck of a Norwegian bridge is simulated. The work compares the performance of two different FEM models. The first one is based on a simple single-Fickian approach which uses only the moisture content as a variable [10] and the second one is a full coupled analysis [7] which uses the multi-Fickian theory with sorption hysteresis of wood [11]. In the single-Fickian approach the

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temperature is taken into account only in the equation that simulates the moisture flux through the exposed surfaces. In the multi-Fickian model the variables of the problem are the vapour water in pores, the bound water in cell walls, and the temperature. Both models were implemented in Abaqus FEM code [12] and have been used also in earlier studies [7, 10]. However, these models were tested for laboratory cases [8] and for timber members under warm Mediterranean climates [7, 13].

Starting from the results provided by the above hygro-thermal models, suggestions for the future development of these models in the presence of Northern climates are given.

2 Material and method

The Evenstad Bridge is studied (Figure 1). Evenstad bridge was built in 1996 across the river Glomma about 100 km North-East from Hammar city in Norway. It consists of five simply supported 36 meters long spans, carried by glulam trusses with arched upper chords. It is one of the longest timber bridges in the world designed for full traffic load. The stress-laminated timber deck is made of 48×223 mm planks. All of the 600 m^3 of wood (excepting railings) was creosote impregnated to keep the moisture content level under 20% to avoid the risk of wood decay. The temperature and relative humidity of the bridge deck and truss members is monitored since 2000. The monitoring data were provided by Norwegian Public Road Administration (NPRA) [14].

Figure 1: Left: View of Evenstad bridge. Right: detail of the stress laminated deck.

The moisture content *u* in the sensors locations was calculated by using Dyken's equation [15]**:**

$$u(RH,T) = \left(a_A T^2 + b_A T + c_A\right) RH^2 + \left(a_B T^2 + b_B T + c_B\right) RH + a_C T^2 + b_C T + c_C \qquad (1)$$

where *RH* represents the relative humidity in percentage, *T* is the temperature in Kelvins and the other coefficients are listed in Table 1.

Table 1: Coefficient of Dyken's equation.

	a	**b**	**c**
A	$0.57 \cdot 10^{-6}$	$29.34 \cdot 10^{-6}$	$1.7007 \cdot 10^{-3}$
B	$70.3 \cdot 10^{-6}$	$2.8933 \cdot 10^{-3}$	$87.6041 \cdot 10^{-3}$
C	$2.0553 \cdot 10^{-3}$	$2.4551 \cdot 10^{-3}$	4.181162

The measurements of temperature and relative humidity were taken by humitters (integrated transmitters type Vaisela Humitter 50Y) in three different depths (see Figure 2). Their operating range from -40°C to 60°C and from 0% to 100% of relative humidity is sufficient for the expected internal conditions in the material. Vaisala humitters are also used to monitor outside temperature and relative humidity which serves as the boundary conditions for our numerical models. The data are collected by data logger Campbell AM416 and transferred to Treteknisk by modem.

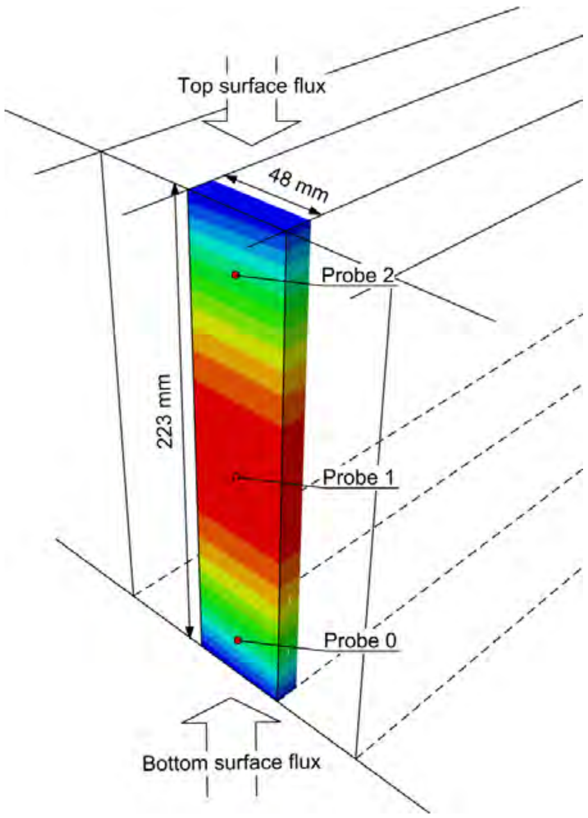

Figure 2: Cross section analysed by Abaqus and measurement points. Distance from the bottom: 20 mm (Probe 0), 110 mm (Probe 1) and 210 mm (Probe 2).

As mentioned in the Introduction, in the present paper both a single-Fickian and a multi-Fickian method for moisture transfer in wood are used to simulate the hygro-thermal behavior of the bridge deck. Both methods are shortly summarized in this section.

Following [7], in the single Fickian approach the moisture transfer was modelled using a three-dimensional Fickian equation:

$$\frac{\partial u}{\partial t} = \nabla \cdot (\mathbf{D} \nabla u) \tag{2}$$

where u represents the moisture content of wood and **D** the diffusion matrix of moisture transfer. The moisture content $0<u<1$ is expressed as $u=(m-m_0)/m_0$, m being the mass of the specimen and m_0 the dry mass. The diffusion coefficients in radial and tangential directions are $D_R\,(u) = D_T\,(u) = 8.64 \times 10^{-7}\, e^{4u}$ [m^2h^{-1}] while in the longitudinal direction, the $D_L\,(u)$ adopted in [7] is used. The moisture flux from the air to the surface is given by the following equation [7]:

$$q_n = \rho_0\, S(u_{air} - u_{surf}) \tag{3}$$

where q_n is the value of the flux over the boundary, ρ_0 the dry density of wood, S the surface emission coefficient, u_{surf} the moisture content on the wood surface, and u_{air} represents the equilibrium moisture content of wood corresponding to the air humidity defined below:

$$u_{air} = 0.01 \left\{ \left[-T \ln(1-h) \right] / \left[0.13 \left(1 - T / 647.1 \right)^{-6.46} \right] \right\}^{1/(110\ T^{-0.75})} \tag{4}$$

where T is the temperature in Kelvin degrees and h=0.01 RH. For uncoated wooden members the reference value of surface emission is S=3.2×10^{-8} e^{4u} [m/s]. In the presence of coatings, this value varies based on the following equation proposed in [5]:

$$\frac{1}{S} = \frac{1}{S_{surf}} + \frac{1}{S_{paint}} \tag{5}$$

where S_{surf} assumes the value for uncoated case, and S_{paint} depends on the type of used coating. The flux equation across the surface is implemented in DFLUX subroutine of Abaqus code (see [7] for the details).

According to [7], the macroscopic differential equations governing the hygro-thermal behaviour of wood within the multi-Fickian theory are defined as

$$\frac{\partial c_b}{\partial t} = \nabla \cdot \left(\mathbf{D}_\mathrm{b} \nabla c_\mathrm{b} \right) + \dot{c} \tag{6}$$

$$\frac{\partial c_\mathrm{v}}{\partial t} = \nabla \cdot \left(\mathbf{D}_\mathrm{v} \nabla c_\mathrm{v} \right) + \dot{c} \tag{7}$$

$$c_\mathrm{w} \rho_0 \frac{\partial T}{\partial t} = \nabla \cdot \left(\mathbf{K} \; \nabla \mathrm{T} \right) + \nabla \cdot \left(\mathbf{D}_\mathrm{b} \nabla c_\mathrm{b} \right) h_\mathrm{b} + \nabla \cdot \left(\mathbf{D}_\mathrm{v} \nabla c_\mathrm{v} \right) h_\mathrm{v} + \dot{c}\, h_\mathrm{bv} \tag{8}$$

where (6) and (7) are the transport equations and (8) represents the equation of energy conservation. The unknowns of the problem are the concentration of bound water c_b, the concentration of water vapour c_v and the temperature T. $\mathbf{D}_\mathrm{b}$ and $\mathbf{D}_\mathrm{v}$ represent the diffusion tensors for bound water and vapour water, respectively, and $\mathbf{K}$ is the thermal conductivity tensor. Furthermore c_w is the specific heat, ρ the dry density of wood and h_b, h_v are the specific enthalpies of bound and vapour water while $h_\mathrm{bv} = h_\mathrm{b} - h_\mathrm{v}$ is the specific enthalpy of the transition from the bound water phase to the water vapour. The coupling term $\dot{c} = H_\mathrm{c}(c_\mathrm{bl} - c_\mathrm{b})$ represents the so-called sorption rate in which $c_\mathrm{bl} = \rho_0\, u_\mathrm{bl}$ $c_{bl} = \rho_0 m_{bl}$ is the bound water concentration in equilibrium with a given relative humidity, according to the Hailwood – Harrobin isotherm

$$u_\mathrm{bl} = \frac{h}{f_1 + f_2 h + f_3 h^2} \tag{9}$$

defined by the shape parameters f_1, f_2 and f_3. The hysteresis of wood is included by using two sorption isotherms (adsorption and desorption) that represent the equilibrium states for moisture in wood and surrounding humidity. The f_i parameters for the curves of adsorption and desorption used within the hysteresis model are the ones proposed by Ahlgren for Norway Spruce (Table 2), see references in [11]. The expression of H_c can be found in [7]. To defined the hysteresis of wood, it is assumed that a feasible state (u_bl, h) lies in the domain bounded by the adsorption and desorption boundary curves $u_\mathrm{a}(h)$ and $u_\mathrm{d}(h)$. According to [16], the equilibrium concentration c_bl is defined as $c_\mathrm{bl} = c_\mathrm{bd}\, s + c_\mathrm{ba}\,(1-s)$ where c_ba and c_bd are obtained through the adsorption and desorption isotherms, while scanning curve s represents the degree of exploited sorption defined as in [16]. The reader is referred to Frandsen's PhD thesis [11] for the detailed description of the variants of the multi-Fickian model and their material parameters.

On the surfaces in contact with air, the boundary condition on the bound water concentration is assumed to be $\nabla c_\mathrm{b} = 0$ while the flux of water vapour is expressed as

$$-\boldsymbol{n} \cdot \mathbf{D}_\mathrm{v} \nabla c_\mathrm{v} = k_\mathbf{v} \left(c_\mathrm{v}^{'} - c_\mathrm{v}^{a} \right) \tag{10}$$

where c_v^{a} is the water vapour concentration in the surrounding ambient, k_v represents the surface emission coefficients for water vapour and $c_\mathrm{v}^{'} = c_\mathrm{v} / \varphi$ is the concentration of water vapour based on the volume of the cell lumens, φ being the porosity, i.e. the volume of lumens with respect to that of the dry wood. The definition of thermal flux and its related parameters can be found in [7]. In the presence of a coating, the resistance to the humidity exchanges between wood and the surrounding environment is calculated as

$$\frac{1}{k_\mathrm{v}} = \frac{1}{k_\mathrm{vs}} + \frac{1}{k_\mathrm{p}} \tag{11}$$

where k_v is the mass transfer coefficient of water vapour flux, k_vs is the surface emission coefficient for the uncoated wood and k_p the coefficient for coating based on the so-called coating permeance (see details in [7]).

Table 2: Multi-Fickian method. Shape parameters of the isotherms.

	f_1	f_2	f_3
adsorption	1.804	13.63	-12.12
desorption	1.886	7.884	-6.526

It is worth to point out that this model does not account for temperature dependence of the sorption isotherms. However in the multi-Fickian model used in the present work, the diffusion coefficients are functions of the temperature. In particular, the matrix of water vapour diffusion depends on the partial vapour pressure $p_v = h\, p_{vs}$ where p_{vs} is the so-called saturated vapour pressure that depends on temperature [16]. The empirical equations of p_{vs} for temperatures above the freezing point and for vapour pressure over ice are provided in [16]:

$$\exp(53.421 - \frac{6513.3}{T} - 4.125\ln(T)) \qquad T > 273.15\text{, water} \tag{12}$$

$$10^{\left(\frac{9.5(T-273.15)}{T-7.65}+0.7858\right)} \qquad T \leq 273.15\text{, ice} \tag{13}$$

Finally, in the present research, an automated Python script which generates, executes and evaluates Abaqus simulations with different geometrical and material parameters is developed to be able to calculate many similar bridge decks in a sequence. The existing user subroutines that contain the core of single- and multi-Fickian methods are modified so that measured weather data may be used.

3 Results and discussion

In the present work, a representative section of vertical lamellas of Evenstad's timber deck is modelled in Abaqus (Figure 2). Both the single-Fickian and the multi-Fickian analyses refer to Norway spruce of density ρ = 450 kg/m^3. The thermal analysis of Abaqus Standard [12] was used for the numerical simulations. DC3D8 four-node linear heat transfer elements of Abaqus were used in the single-Fickian approach. A user element implemented in the subroutine UEL was adopted for the multi-Fickian analysis.

The effect on the diffusion analysis of the glue lines between different laminations was neglected in both analyses. The initial condition of a uniform moisture content (MC) corresponding to the measured moisture content was assumed for the cross section.

In Figure 3 the histories of relative humidity of the air (*RH*) and air temperature *T* (˚C) monitored at the bridge location are shown for a period starting from 16.5.2003 to 6.7.2004. In the same figure the measured values of MC at the probe locations (Figure 2) are reported. The numerical response in term of moisture content in wood is simulated by using both the single-Fickian and the multi-Fickian hygro-thermal approaches for two different periods named History1 (starting from 16.5.2004) and History 2 (starting from 16.4.2004). In both periods the values of air temperature are mainly over zero, with only some values under zero up to a temperature of -5 ˚C. The results of numerical simulations at Probe 2 location are discussed in this paper.

In Figure 4 the results of single-Fickian simulations for the hygro-thermal load History 2 are reported for different values of the emission coefficient *S* of Equation (3). The results show that the simulation of creosote preservative requires emission coefficients in the range of 1E-9 e^{4u} – 5E-9 e^{4u} [m/s]. It can be observed that the numerical solution in the presence of coatings is quite smooth compared with the measurements. This is also due to the fact that only one sorption isotherm is adopted (Equation 3).

Figures 5 and 6 illustrate the comparison between single-Fickian and multi-Fickian results under the hygro-thermal loads History 2 and History 1, respectively. Due to the use of a hysteresis model for wood, it can be observed that the multi-Fickian solution follows better the measured variations of MC inside the material even if the values of simulated moisture peaks are lower than the measured ones. For both histories, the numerical solution in a point closer to the external surface (6 mm from the surface) exhibits a behaviour more similar to that measured at 13 mm (Probe 2). One reason may be that the used multi-Fickian method was calibrated for wooden elements sheltered from rain and sun while Evenstad's deck is directly exposed to the climate. Furthermore, in earlier works this method was used to simulate laboratory cases (T~ 20 ˚C) or for warm Mediterranean climates (according to the Köppen-Geiger climate type classification of Europe, see [5]). The present hygro-thermal histories refer to a Northern climate that are characterized by very high variations of *RH* (see Figure 3) and with frequent rain events that cause peaks of *RH* of 100%. The high relative humidity is also affected by the presence of the river.

Also the temperatures of the specific climate affect the solution. In particular, the current methods are not developed for cases with negative temperatures. In the single-Fickian model the external temperature appears in the equation of the sorption curve (4) but while it does not affect the solution under service life temperature of buildings [10], it has not been tested for negative temperature in outdoor conditions. In the multi-Fickian model, apart from Equation (13) for vapour pressure over ice, the hysteresis curves of the method are temperature-independent also over zero and this can affect the shape of the MC solution discussed above. An attempt to include the temperature dependent hysteresis in the multi-Fickian model was done in [13] but the simulated cases referred to warm Mediterranean climates and were not compared with measurements. In the case of Northern climates the temperature dependence is a challenging topic since most available measurements of sorption curves are at temperature greater than zero (see paper 6 in [11]). However, in the present work it seems that the method works well for negative temperatures up to -5 ˚C.

The simulation of coating is another important aspect of the hygro-thermal modelling. The surface emission coefficient in both the presented numerical approaches were tested against monitoring data only for cases of temperatures over zero without direct exposure to rain. In addition, it is not easy to find reference values for surface emission coefficients suitable to describe the creosote preservative that has a different behaviour compared to the most tested coatings of alkyd oil type [5,7]. Finally, the effect of solar radiation and wind can also affect the surface emission [17].

The levels of moisture inside wooden elements of timber bridges exposed to rain or driving rain is important in estimating the service life and in drafting maintenance schemes. This moisture information may be applied as an input load for models to predict the durability performance as has been presented in reference [18] for instance.

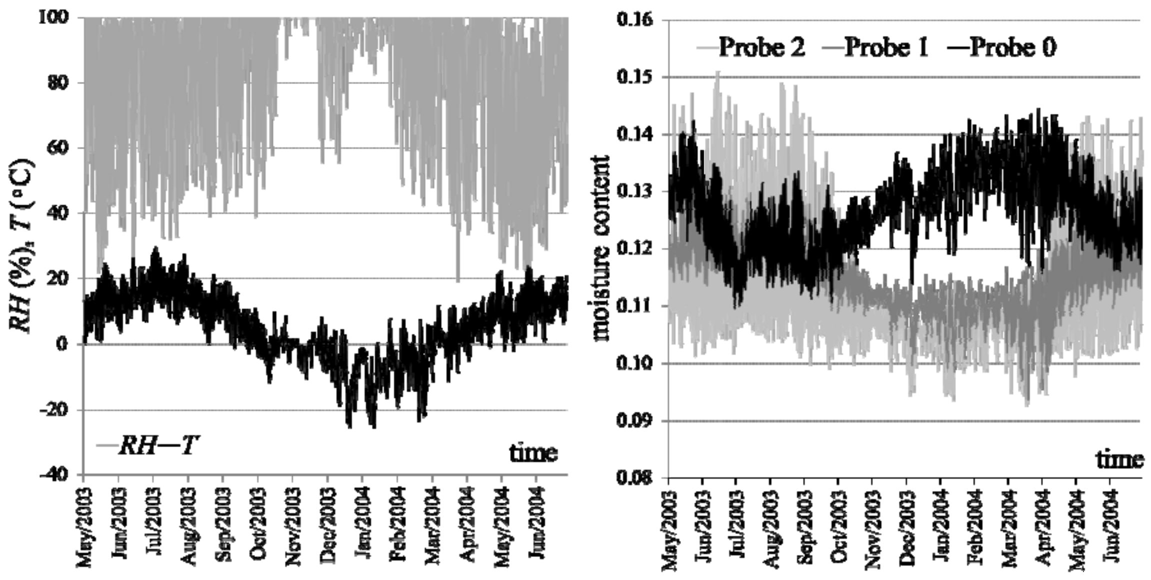

Figure 3: Left: Relative humidity (RH) and temperature (T) of the air from May 2003 to July 2004. Right: measurements in the probe locations of in Figure 2.

4 Conclusion and future work

In this paper earlier numerical FEM models for moisture transfer in wood (a single-Fickian and a multi-Fickian based model) were used to simulate the hygro-thermal behaviour of stress-laminated timber decks impregnated by creosote. As a case-study, the stress-laminated timber deck of Evenstad's bridge in Norway was calculated. The results show that both methods agree in average with the results but they can be further improved to simulate more accurately the hygro-thermal response of the creosote-impregnated decks under Northern climate conditions. More data and experimental tests on small and medium size wood specimens with and without coating treatment are needed under direct exposure to Northern climatic conditions to calibrate the surface coefficients and to assess the hysteresis curves to be used in the hygro-thermal numerical models.

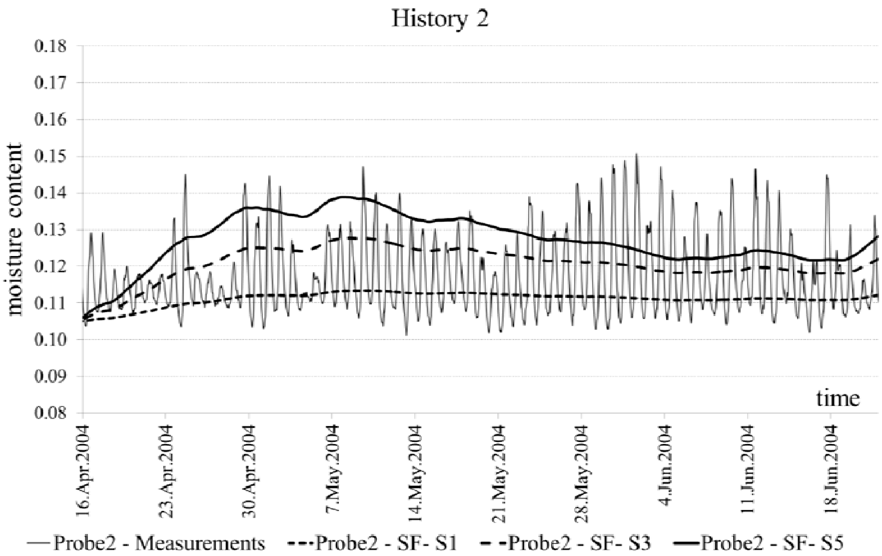

Figure 4: History 2. Single-Fickian simulation (SF). Effect of the emission coefficients S1=*1E-9* e^{4u} *m/s,* S_3*= 3E-9* e^{4u} *m/s,* S_5*=5E-9* e^{4u} *m/s, u= moisture content of wood.*

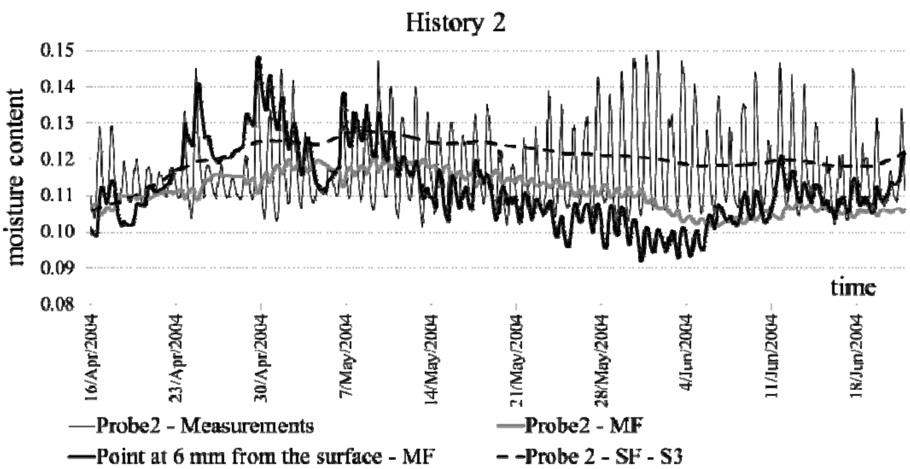

Figure 5: History 2. Comparison between single-Fickian (SF) and multi-Fickian (MF) simulation results and measurements at Probe 2. The numerical moisture content at a 6 mm from the external surface is also reported.

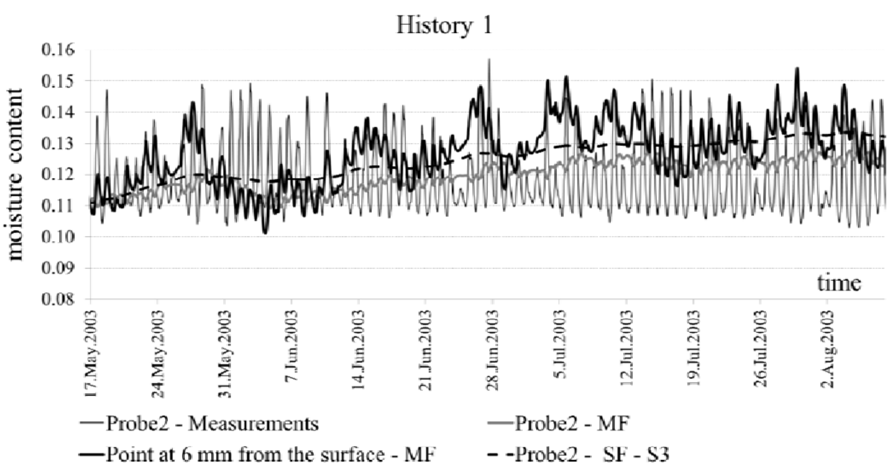

Figure 6: History 1. Comparison between single-Fickian (SF) and multi-Fickian (MF) simulation results and measurements at Probe 2. The numerical moisture content at a 6 mm from the external surface is also reported.

5 Acknowledgement

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Finite element modelling used to support wood failure identification based on acoustic emission signals

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Summary

Fractures in timber structures result from the accumulation of microscopic failures, which can be detected with acoustic emission sensors. Recent research has provided evidence that different types of wood failure can be distinguished based on the acoustic emission signature. Pattern recognition yielded two types of acoustic emission signals, mainly differing in weighted peak frequency. The cluster with higher frequencies could be related to cell wall cracks, while the low frequency cluster was correlated with cell separation. It was hypothesized that the different speed of crack growth in both failure types resulted in the distinct frequency spectrum. To prove this hypothesis, we implemented a finite element model and conducted a systematic study with a range of different excitation times, locations and orientations. We concluded that the excitation time (and thus crack speed) has the strongest potential to significantly change the frequency content of the AE signal. However, sensor sensitivity may have a major influence on the detectability and distinctiveness of different failure types.

Key words: acoustic emission detection, finite element modelling, frequency analysis, tensile test, wood fracture

1 Introduction

Fractures in timber structures result from the accumulation of microscopic failures that finally lead to macroscopic damage. Monitoring microscopic failure with the acoustic emission (AE) technique may be used as an early warning system for fractures of timber structures. This non-destructive technique is also useful for the mechanical assessment of wood or composite wood [1–4] and even for the assessment of drought-vulnerability of woody species [5,6]. However, there is still a knowledge gap between the detection of acoustic emissions and the understanding of underlying microscopic mechanisms. Identification of microscopic failure mechanisms based on the acoustic emission signature would be of large interest for both assessment of wood materials and monitoring of timber structures. Recent research on spruce wood (*Picea abies* (L.) Karst.) has taken the first steps in this direction [7–9]. Tensile tests were performed on miniature specimens (cross-sections of a few mm^2) of solid spruce wood loaded in radial and longitudinal orientation [7,8] as well as on laboratory-scale plywood specimens [9]. Pattern recognition yielded two types of acoustic emission signals in all tests, which differed mainly in weighted peak frequency. These were hypothesized to reflect source mechanisms with different speeds of crack growth. Combining the acoustic emission technique with X-ray micro-computed tomography supported the hypothesis that rapid and slow crack growth were probably related to cell wall cracks and cell separation phenomena, respectively. In this work, we used micro-mechanical finite element modeling to investigate the link between the acoustic emission signature and the speed of crack growth for the miniature specimens. Cracks were modelled as dipoles and faster and slower cracks were simulated by a shorter and longer excitation time, respectively. We present a systematic study with a range of different source excitation times, locations and orientations.

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2 Material and method

2.1 Finite element modelling

We implemented a model of a miniature test specimen in COMSOL with aluminium specimen holders (Fig. 1) using the average dimensions of the specimens used by Ritschel et al. [8]. For the adhesion between aluminium holder and specimen, a polyurethane adhesive was used. The properties of the different materials are listed in Table 1. The specimen was oriented in a way that the z-direction, in which the tension was applied, was aligned along the grain. The x-z plane, on which the sensor was attached, was perpendicular to the wood rays. A convergent solution was reached with 5 x 10^{-8} s time step and a mesh size of 1-25 x 10^{-5} m with maximum element growth rate of 1.5 for the wood specimen and a mesh size of 1-100 x 10^{-5} m with maximum element growth rate of 1.2 for the holders and adhesive glue.

The microscopic fracture was implemented as an internal dipole [10], consisting of opposite directed loads that act on two points, spaced at 0.08 mm from each other. As loading function we implemented a linear force pulse (Eq. 1) [11] with F_{max} of 3 N and excitation time τ of 0.5 µs.

$$F(t) = F_{max} \cdot (t/\tau) \qquad t \leq \tau$$
$$F(t) = F_{max} - F_{max} \cdot 2(t-\tau)/\tau \qquad \tau < t \leq 1.5\tau \qquad (1)$$
$$F(t) = 0 \qquad t > 1.5\tau$$

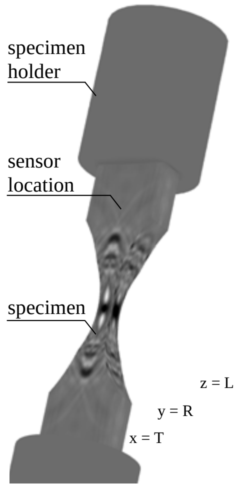

Figure 1: Wave propagation 4 µs after onset of dipole at the centre of the specimen

The dipole was oriented along the z-axis and located near the centre of the specimen. Because a dipole exactly in the centre is not realistic and would suppress all anti-symmetric wave modes, we chose a location slightly off-centre (0.01 mm).

We systematically studied the influence of source location, orientation and excitation time to study the influence of these parameters on the simulated signal. We shifted the dipole twice along the x- and y-axis by 1/6 of the thickness in the respective direction and along the z-axis it was translated twice over 0.5 mm. Dipoles were also simulated in the x- and y-direction and excitation times were varied between 0.1 and 20 µs (0.1, 0.5, 1, 3, 5, 10 and 20 µs).

The average y-displacement over the sensor surface (with diameter 3 mm) was evaluated at the sensor location at 7 mm from the centre of the specimen. We did not explicitly model the sensor geometry and sensor coupling to the wood specimen. To enable comparison with experimental results of Ritschel et al. [8], we down sampled the signal to 10^7 samples per second and used only the initial 256 samples (25.6 µs) to calculated the Fourier transformation. The frequency spectra shown in this manuscript are normalized by the sum of the moduli over 0-1000 kHz.

Table 1: Elastic properties and density of spruce wood [12], *aluminium and polyurethane.*

	Density (kg m^{-3})	Young's modulus of elasticity (MPa)		Shear modulus (MPa)		Poisson's ratio	
Spruce wood	280	E_T	420	G_{LR}	623	ν_{TR}	0.31
		E_R	818	G_{TL}	743	ν_{RL}	0.056
		E_L	12000	G_{TR}	42	ν_{TL}	0.035
Aluminium	2700	E	70000	/		ν	0.33
Polyurethane	1520	E	690	/		ν	0.25

2.2 Description of system response using pencil lead breaks

In an experimental set-up, the AE signal that is available for analysis is influenced by the sensor response, the bandwidth of the preamplifier (30-1000 kHz) and filter settings of the acquisition board (30 to 960 kHz). To interpret the experimental signals based on the theoretical simulations, we have to filter the simulated signals according to the experimental system response. By means of a pencil lead break on an aluminium plate, we investigated the system response of the measurement chain used by Ritschel et al. [8]. The sensor (type M31 from Fuji Ceramics Corp.) was installed on a thin aluminium plate (6 x 380 x 400 mm^3) and pencil lead breaks were made at a distance of 120 mm from the sensor. We simulated this experiment in COMSOL and compared the frequency spectra of simulated and experimental signals. Because the elastic properties of aluminium (from COMSOL library) and the source function for a pencil lead break (Eq. 2, with τ equal to 1 µs) are well known [13], the modelling results of the surface displacement are expected to be valid.

$$F(t) = 0 \qquad t < 0$$

$$F(t) = 0.5 - 0.5 \cdot \cos(\pi t/\tau) \qquad 0 \leq t \leq \tau \tag{2}$$

$$F(t) = 0 \qquad t > \tau$$

In the experiments, two sensors (type M31) were installed symmetrically on the wood specimen, one sensor at each side of the taper, and only the first arriving signal (first “hit”) was used for the analysis. Therefore, we used the average experimental frequency spectrum for calculation of the system response. The ratio of the experimental and simulated frequency spectra was used as a filter to mimic the response of the system used during tensile tests. After filtering the simulated frequency spectra with the calculated system response, we extracted the frequency features that showed good differentiating capacity for the experimental signals. These features are weighted peak frequency (geometric mean of peak frequency and frequency centroid) and partial power of the frequency range 600-800 kHz.

3 Results and discussion

3.1 Simulated AE signal spectra are most sensitive to excitation time of AE source

The spectra of AE signals from simulated cracks at different locations along the x- and z- axis were very similar (Fig. 2A and 2C), but different depth positions below the surface at which the sensor was attached (Fig. 2B) changed the frequency distribution. Frequencies at 0-300 kHz appeared when the source location was farther away from the centre of the specimen (at the origin of the coordinate system). The reason for this lies in the occurrence of symmetric (S0) and anti-symmetric (A0) wave modes in the plate-like wood specimen. A larger

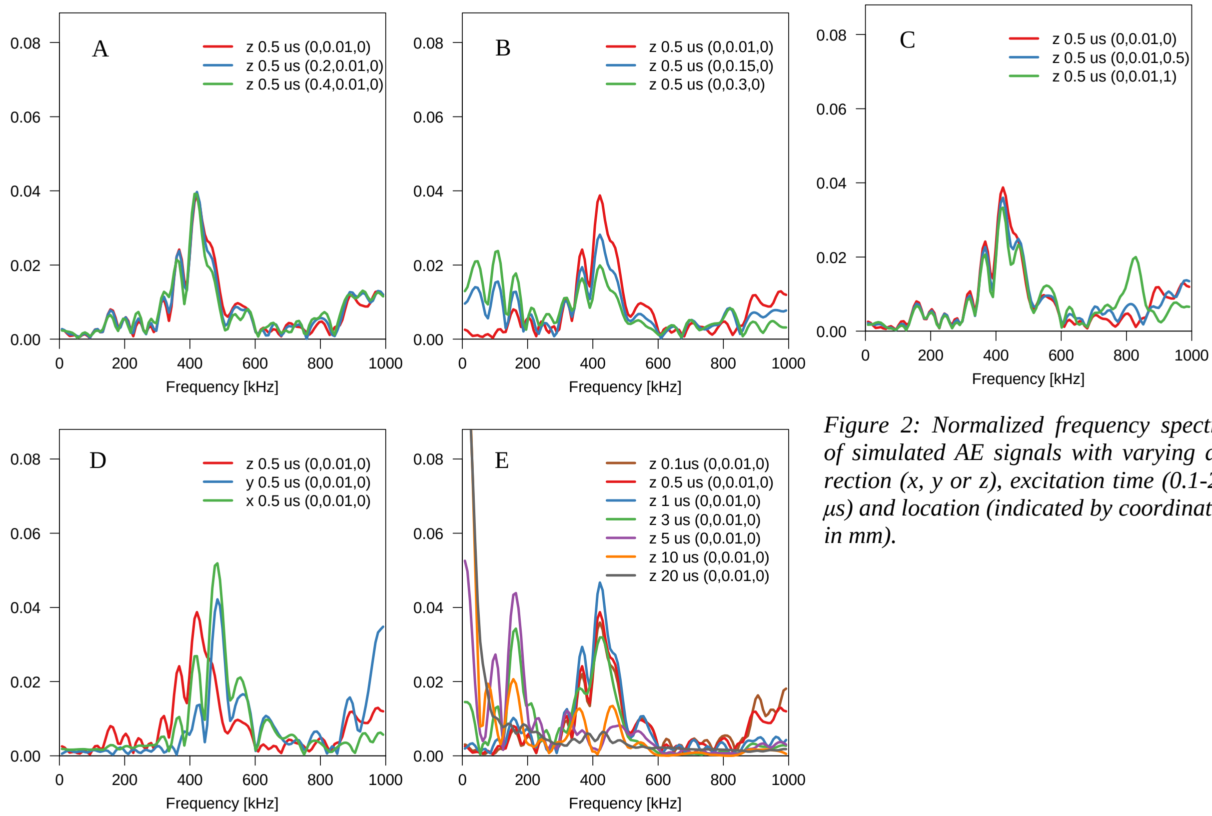

Figure 2: Normalized frequency spectra of simulated AE signals with varying direction (x, y or z), excitation time (0.1-20 µs) and location (indicated by coordinates in mm).

distance from the AE source to the symmetry plane results in a higher contribution of the A0-mode, which typically has a lower frequency than the S0-mode [14]. The A0-mode only acts in the direction perpendicular to the symmetry plane. This explains why the A0-mode activated by off-centre locations in the x-direction is not detectable by the sensor (Fig. 2A), which only measures surface displacements in the y-direction. Small differences in the spectrum above 800 kHz due to displacement along the z-axis (Fig. 2C) are probably due to the shorter source-to-sensor travel path, but the influence is very limited.

Changes in source orientation, keeping all other parameters constant, were also reflected in the frequency spectrum (Fig. 2D). When the simulated crack opened along the z-axis (in the loading direction, crack plane perpendicular to the z-axis), the main frequency peak was located at 300-500 kHz. For cracks perpendicular to the tensile load, the peak was shifted towards 400-600 kHz and for the y-directed crack opening, a peak above 800 kHz appeared.

From all parameters investigated, the source excitation time had the largest influence on the frequency spectrum of the detected AE signal (Fig. 2E). With excitation time ranging from 0.1 to 20 µs, the main peak in the frequency spectrum shifted from 300-500 kHz to 100-200 kHz and was <50 kHz for the slowest AE sources. When two different sources of acoustic emissions act in a distinct time scale, e.g. 1 versus 10 µs, the influences of source direction and location become negligible and both AE sources will be distinguishable based on the frequency spectrum. However, the sensors used in the experiments have a steeply dropping sensitivity below 200 kHz [8]. The low frequency peaks (Fig. 2E) are thus hardly observable in the experimental signals and this might be a disadvantage for the detectability and distinctiveness of different speeds of crack growth.

3.2 System response is important for interpretation of frequency features

Comparison between experimental and simulated AE signals from pencil lead breaks (Fig. 3) showed a non-flat frequency response with a global sensitivity between 100 and 900 kHz, strong sensitivity at frequencies 600-800 kHz, a drop at about 200-250 kHz and a global increase from 100 to 700 kHz (Fig. 3C). The lower boundary of the sensitivity range and the drop above 200 kHz corresponded to the frequency response curve of the sensor as obtained by Ritschel et al. [8]. The low-pass filtering at 900 kHz was realized by the acquisition system. However, the gradual increase in sensitivity from 100 kHz tot 700 kHz and the extremely high sensitivity at 600-800 kHz was not represented by the sensor response curve obtained by Ritschel et al. [8]. This response is probably specific to our experimental set-up, which differed from the calibration set-up [8] with respect to material used and direction in which wave propagation occurred.

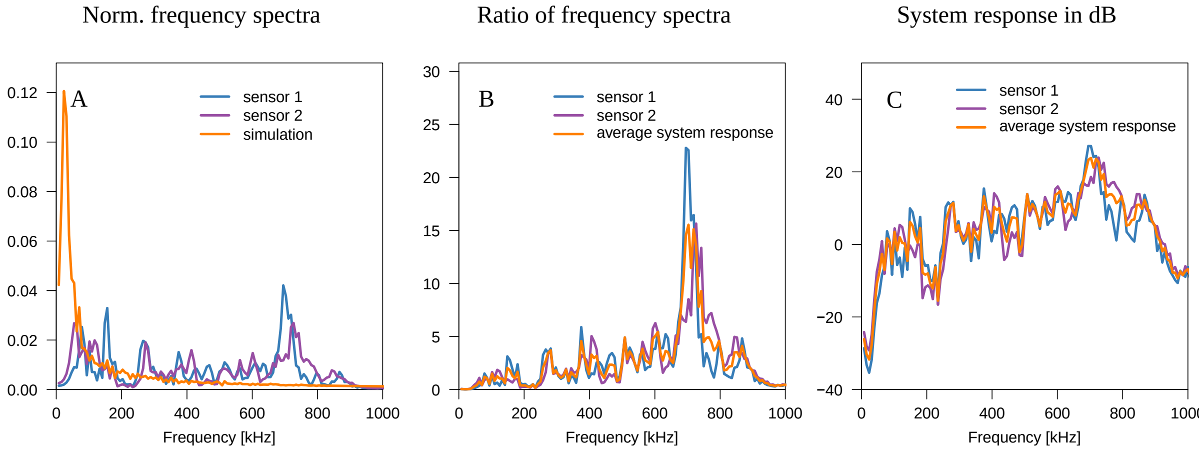

Figure 3: Normalized frequency spectra of simulated and experimental AE signals caused by pencil lead break on aluminium plate (A), ratio between the experimental and simulated frequency spectra (B) and the ratio expressed in dB (C).

The features extracted from the filtered frequency spectra (Fig. 4) do not correspond to the features obtained from the experimental signals [8]. Ritschel et al. [8] found two clusters located in a small band of weighted peak frequency around 350 and around 600 kHz, while our results showed weighted peak frequencies in the region 400-600 kHz and a distinct lower value at about 250 kHz. Also the values of partial power at 600-800 kHz were much lower in our results (0-40%) than in the original experiments (10-70%). Probably the assumption of a linear system response is not valid for the resonance behaviour of the sensor at 700 kHz, making it more complicated to approach the actual measurements using finite element modelling. A better match between experiments and simulations could be obtained by detailed modelling of the sensor interior [15].

Although the locations of the simulated AE signal clusters did not correspond with the experiments, it was illustrated that translation and direction of the AE source had only little influence on the calculated weighted peak frequency. The source excitation time had a large influence on the frequency spectrum (Fig. 2E), but after filtering only the signal with 5 µs excitation time clearly fell in a 'cluster' with lower weighted peak frequency than the other signals. The signals with 10 and 20 µs excitation time were unexpectedly located in the same cluster as the high frequency signals. The frequency spectrum of these signals (Fig. 2E) showed that the main part of the frequency content was below 50 kHz. Considering only the spectrum above 100 kHz could be misleading for these signals. Another type of filter (i.e. other sensor), with higher sensitivity at low frequencies, would move these low-frequency signals towards the leftmost point in Fig. 4E.

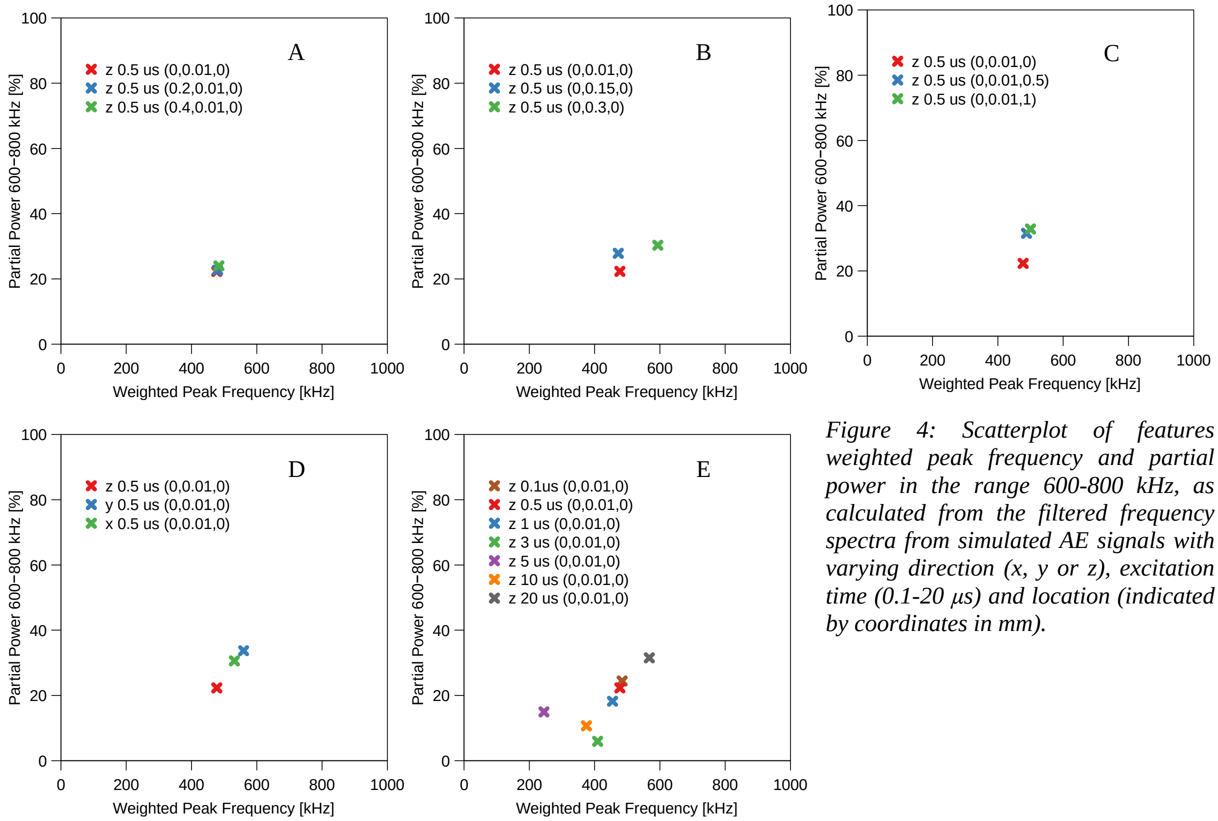

Figure 4: Scatterplot of features weighted peak frequency and partial power in the range 600-800 kHz, as calculated from the filtered frequency spectra from simulated AE signals with varying direction (x, y or z), excitation time (0.1-20 µs) and location (indicated by coordinates in mm).

4 Conclusion and future work

The non-flat frequency response of the sensor complicated the comparison between simulations and experimental results. However, based on the results of our sensitivity study we can conclude that the excitation time of the AE source mechanism has the strongest potential to significantly change the frequency content of the AE signal. Our results support the hypothesis that the two clusters obtained during tensile tests probably originate from two different source mechanisms with distinct excitation time and thus different speeds of crack opening. Further investigations could provide more solid proof, e.g. by implementing a detailed sensor model or by using a broadband sensor with flat frequency response during the tensile tests.

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Using Smartphones to Identify Dynamic Characteristics of Timber Bridges

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Summary

This paper presents the preliminarily outcomes of a joint pilot project between the University of Bath and the University of California, Berkeley, and aims to demonstrate the feasibility of implementing the Smartphone as a device for the system identification of timber bridges. Two timber bridges were selected for vibration measurement, Black Dog Bridge in the UK and Knight Ferry Covered Bridge in the United States. To ensure the reliability of the data, a Smartphone was calibrated on a shaking table in the laboratory before measuring onsite. Two vibration tests were carried out on both bridges, these being free vibration and ambient vibration tests. The results were then analysed in both time and frequency domains. This paper also demonstrates the feasibility of Smartphones being used to identify the dynamic properties of timber bridges.

1 Introduction

Timber is a bio-degradable material that, for public safety structural health monitoring, is becoming an important technique to ensure the long term structural integrity of timber bridges. A number of health monitoring methods have been reported. For example, Tannert, Müller and Vogel used Non-destructive Testing methods (NDT), including core drilling and drilling resistance, to investigate the local condition of timber within a bridge, and monitored the moisture content of the timber members in the long term [1]. This project was limited to investigating the condition of individual members and obtaining material conditions where the NDT techniques were applied. Unlike NDT, limited to detecting local damage, deflection reflects how a timber bridge performs at the structural system level. Long term deformation due to creep and moisture content variation were investigated by using fibre optic sensors and GPS system and the outcomes compared with those obtained from conventional tri-axial accelerometers [2]. However, this still limits the engineers to analysing the structural integrity of timber bridges from static loading. With the system identification (SI) of timber bridges it is important for engineers not only to understand the dynamic properties of the structure they have designed, but also to be able to assess the structural health of a timber bridge.

Vibration measurements on infrastructures, in particular steel and concrete bridges, have been widely used to analyse the dynamic properties of the structures and assess their structural health in Europe and the United States [3-5]. Most of these measure the traffic-induced or ambient vibrations of concrete and steel bridges. Timber bridges are more flexible and therefore more suitable for vibration=based structural health monitoring methods to assess their conditions. Several projects have reported the assessment of the structural integrity of timber bridges by vibration measurements in which they were excited by instrumented hammer [6] or vehicles [7]. These vibration based structural health monitoring techniques normally require installing accelerometers on the structures. However, installing the conventional cabled sensors, in particular with high density sensor networks, has put industry and academics off due to the cost of the equipment itself and the installation of the coaxial cables. In the late 1990s, wireless sensors integrated with radio were proposed which substantially reduced the cost of the structural monitoring systems [8]. Since then, wireless sensors have quickly developed as a viable solution for structural health monitoring purposes [9].

2 Selection of Smartphones

Different makes and models implement different hardware and software, which leads to variations in the efficiency and suitability of Smartphones used as vibration measurement sensors. The parameters that need to be considered include: (1) sensitivity of accelerometer; (2) sampling rate; and (3) computational capacity. Table 1 summarises a variety of makes and models of off-the-shelf Smartphones. With the rapid development of Smartphones, computational capacity has become less of an issue. From Table 1 we can see that the Samsung Galaxy

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S IV has the best combined sensitivity and sampling capacity performance and hence in this project it was chosen as the sensor for vibration measurement of timber bridges.

Table 1 Technical data of Smartphones

Make and model	Sensitivity of accelerometer	Sampling rate
Samsung Galaxy S III	0.0006 m/s^2	100 Hz
Samsung Galaxy S IV	0.0006 m/s^2	100 Hz
HTC Sensation	0.0383 m/s^2	50 Hz
HTC One X	0.0120 m/s^2	100 Hz
Motorola Droid Razr HD	0.0012 m/s^2	125 Hz

3 Laboratory calibration

Before the tests, we calibrated the Smartphone on the Smart Shaking Table at the University of California, Berkeley (Figure 1). The Smartphone and an accelerometer were mounted on the structure on the shaking table. Two different tests were carried out, free vibration and forced vibration. The vibration signals captured by the accelerometer were compared with those captured by the Smartphone. It was observed that good agreement was found from the free vibration tests. However, high frequency noise was observed in the data from the Smartphone, which implies the need for appropriate filter techniques.

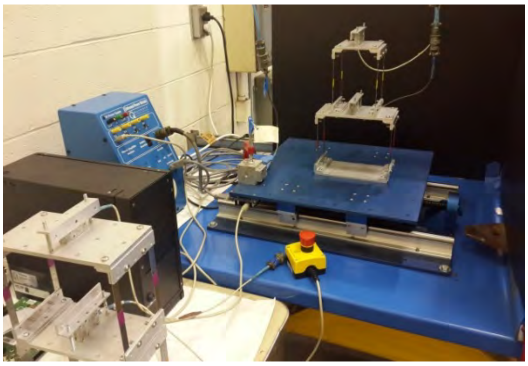

Figure 1 Smart shaking table for the Smartphone calibration

4 The bridges

In this paper, we have selected two bridges to measure the vibration, the Black dog Bridge in the UK and the Knight Ferry Covered Bridge in California, US. The Black Dog Bridge (Figure 2 and Figure 3) is located in Wiltshire, UK, one of the Millennium bridges in the area. The bridge spans 40 metres across the A4 road and is part of the National Cycle Network. It has a lightweight 2.1 metre width deck supported by a Glulam parabolic arch with a section of 675x675 mm and concrete foundations at both ends. The bridge is exposed to the environment with no protection from water and sun and therefore moderate bio-degradation was observed. The combined traffic and human induced vibration is the main source of the structural vibration of Black Dog Bridge.

Figure 2 Black Dog Bridge from the road

Figure 3 Black Dog Bridge from the deck

The Knight Ferry Covered Bridge (Figure 4 and Figure 5), a trussed and covered bridge, was completed in 1864, and is located in Stanislaus County, California. It features the longest covered bridge in California with a span of 100 metres, supported by two piers at both ends. The bridge is currently restricted to passenger access due to the discovery of structural safety issues, and has steel reinforcement. Its timber members are well protected by the cover.

Figure 4 Knight Ferry Covered Bridge from outside

Figure 5 Walkway of Knight Ferry Covered Bridge

5 Vibration measurement

Two different types of tests were carried out, i.e. free vibration and ambient vibration. At the Knight Ferry Covered Bridge, the Smartphone was placed at the middle of the largest span so as to capture the largest vibration signal. Excitation was then generated by a person weighing approximately 100 kg jumping on the bridge, and free vibration was measured for 2 minutes. Then a new measurement was started at the same location and measured the ambient vibration of the bridge for 30 minutes. When measuring the vibration of Black Dog Bridge, both conventional vibration measurement system and a Smartphone were used so as to compare the results. Both sensors and Smartphone were placed in the middle of the span, free and ambient vibrations of the bridge were then measured using the same procedure.

6 Results and Discussions

Black Dog Bridge

Figure 6 compares the time history data obtained form both accelerometers and Smartphones. It shows that the acceleration captured by Smartphone was smaller than that obtained from accelerometers. Fast Furiour Transform (FFT) was applied to investigate dynamic properties in the frequency domain. The natural frequency of the Black Dog Bridge, calculated from the time history record captured by acclerometers is 2.44Hz with damping ratio of 2.6%, compared with 2.44Hz with damping ratio of 3.3% from Smartphone. The analyses in frequency domain shows that it is more difficult to simply use a peak picking method due to the fact that the Smartphone data shows significant noise in high frequency as can be seen in Figure 7. The ambient vibration data of vertical movement of Black Dog Bridge is shown in Figure 8, and Figure 9 shows the FFT spectra of the ambient vibration data. It is obvious that the frequency domain technique is not suitable to analyse the ambient vibration data of the bridge as captured by the Smartphone. It is therefore proposed to use the time domain technique to obtain ambient vibration data.

Random Decrement Technique was implemented on the time-history records of ambient vibration of the Black Dog Bridge obtained from both accelerometers and Smartphone as shown in Figure 10 and Figure 11. Ibrahim time domain (ITD) method was then applied to determine the corresponding dynamic properties. The natural frequency and damping ratio from acclerometers calculated by ITD are 2.49Hz and 3.18%, whereas those from Smartphone are 2.49Hz and 3.32%, respectively. It shows good agreement between natural frequency and damping ratio obtained from both acclerometers and Smartphone.

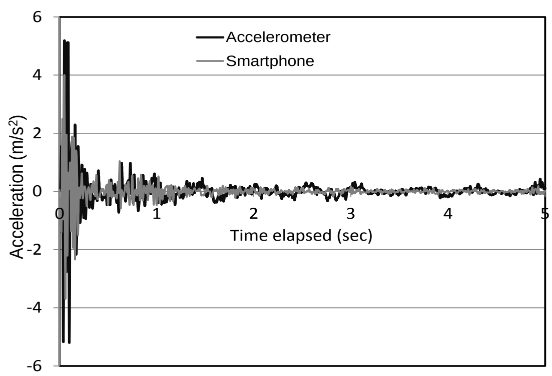

Figure 6 Comparison of free decay signal from accelerometer and Smartphone

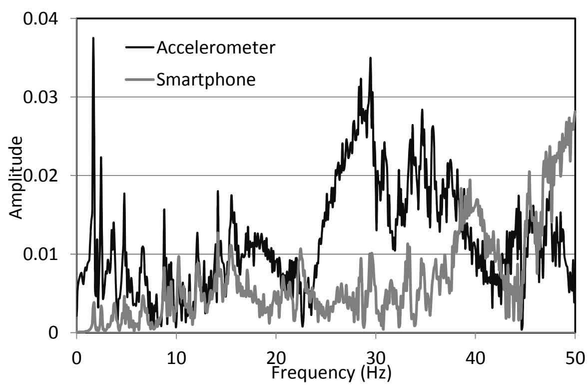

Figure 7 Comparison of frequency spectra of free decay signal

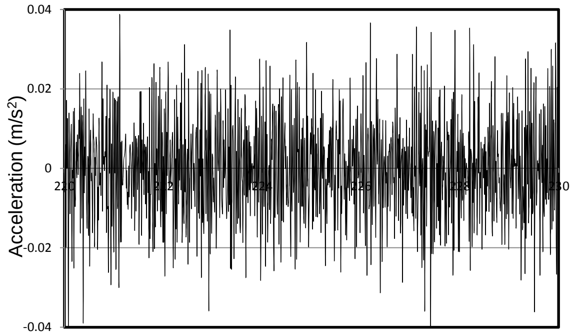

Figure 8 Time history record of ambient vibration of Black Dog Bridge captured by Smartphone

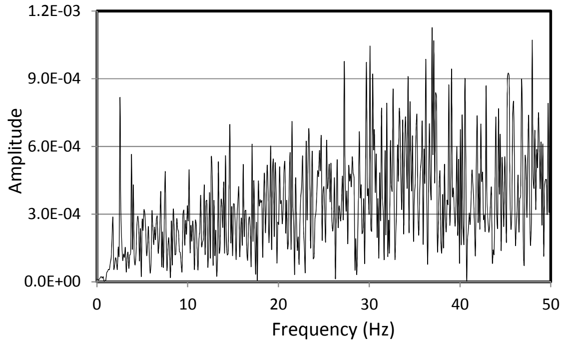

Figure 9 Spectra of time history record of ambient vibration of Black Dog Bridge captured by Smartphone

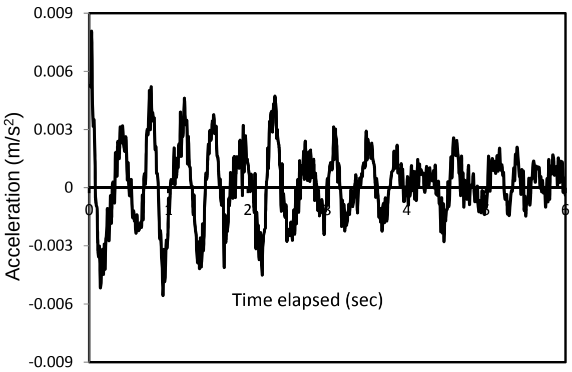

Figure 10 Random decrement signature of accelerometer recorded data

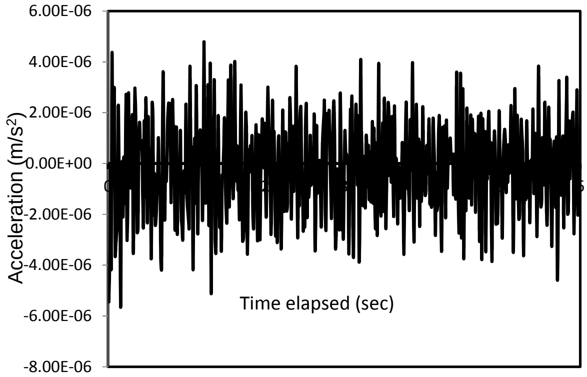

Figure 11 Random decrement signature of accelerometer recorded data

Knight Ferry Covered Bridge

From reuslts at Black Dog Bridge, it can be shown that using Smartphone is a reliable method for system identification, and good agreement has been shown when comparing the results obtained from both accelerometers and the Smartphone. This study also uses the same device to measure the dynamic properties of Knight Ferry Covered Bridge. Two methods were employed to analyse the vibration signal. Frequency domain technique was applied to the forced vibraton record, whereas time domain technique was applied to the random decrement signature of ambient vibration of the bridge. Figure 12 shows the free vibration signal of Knight Ferry Covered Bridge, which was used to identify the natural frequency of the structure in frequency domain. To analyse the ambient vibration of the two bridges, random decrement technique was applied as shown in Figure 13. The results are tabulated in Table 2. Good agreement was found in dynamic properties of Knight Ferry Covered Bridge analysed from both time domain and frequency domain methods.

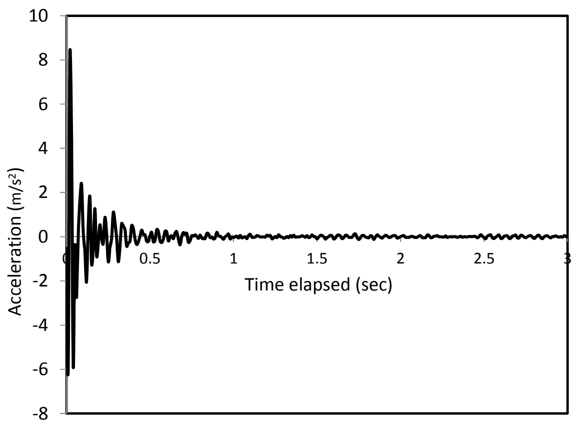

Figure 12 Free vertical vibration decay curve of vertical movement of Knight Ferry cover Bridge captured by the Smartphone

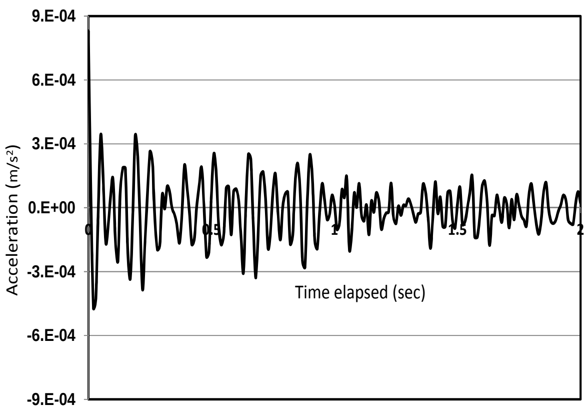

Figure 13 Random decrement signature of vertical movement of Knight Ferry cover Bridge captured by the Smartphone

Table 2 Comparison of dynamic properties analysed from time and frequency domains method

	Natural frequency (Hz)	Damping Ratio
Time domain	3.86	1.8%
Frequency domain	3.71	2.8%

7 Conclusions

In this paper, we propose to use an off the shelf Smartphone as an alternative to the conventional vibration measurement facility, which would normally include a number of accelerometers connected to data acquisition systems by coaxial cables. The ambient and forced vibrations of two bridges were measured, comparisons on the results analysed from both frequency and time domains were made. The following conclusions can be drawn from this project:

(1) When choosing the Smartphones to measure the vibrations, the sensitivity of the accelerometer and maximum sampling rate should be considered.
(2) When an appropriate Smartphone is chosen, it can be a feasible alternative to measure forced vibration.
(3) Time domain analyses are suitable to analyse the ambient vibration of bridges.

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The precursors of timber truss bridges

Clara Bertolini Cestari[1], *Tanja Marzi*[2]

Summary

Structural typologies of ancient timber constructions have offered to modern bridges suggestions of undoubted importance that were transposed, filtered and brought to basic static and geometric forms from new materials techniques and technologies.

The proposals of Palladio for bridge girders had offered reticular schemes unexceptionable under the static and geometric profile: their essentiality, however, was incompatible with a material - solid wood - with relatively limited elastic modulus and with connections typologies achievable at that time, therefore, with scarce overall stiffness that gave rise to structures of large deformation and little reliability. Wood girders then diverged more and more from the schemes of Palladio, strongly increasing (and often disorderly), the number of rafters of reticular scheme in a constant attempt to couple the work of the beam to the work of the arc. Only after 1830 in the United States appeared the first timber trestle railway bridges in which the stiffening elements, longitudinal and diagonal rafters, had substituted the arc, assuming the main load-bearing functions.

The paper describes the various stages of the evolution of trusses, patents, typologies, models, etc.

Key words: timber truss, historical bridges, new technologies

1 Introduction

This paper provides a brief historical overview on truss bridges. It starts from the early sketches of Leonardo da Vinci, trough the achievements of Palladio, until the Venetian bridges. It considers the important transformations of the main typologies and connections of the early 19th century and concludes with a comparison between historical examples. Starting from these assumptions, it comes to the conclusion that thanks to new technologies and new materials, some of the newer timber bridges are re-proposing the simplicity of the Palladian bridge.

2 The "fathers" of wooden bridges

2.1 The inventions of Leonardo da Vinci

Leonardo da Vinci designed wooden bridges that still nowadays we identify for originality of composition and of typology. He can be considered the true "father" of wooden bridges. The drawings of Leonardo show with great rationality the tie elements - mentioned as cables or wires - and compressed elements - identified as real struts-. Some of Leonardo's bridges drawings are in a single span and have a crown that could almost be defined as arch bridges (Fig. 1). These drawings precede that of Palladio, but to Leonardo must be attributed the idea of the geometric support as a reason of the mechanical strength: the triangular mesh, the mesh structure (Fig. 2).

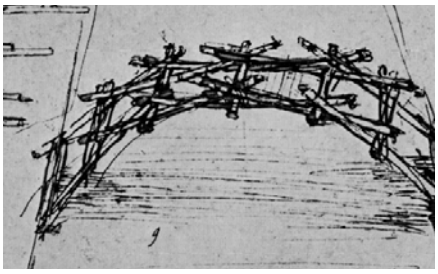

Figure 1 Ponte speditivo, Leonardo da Vinci

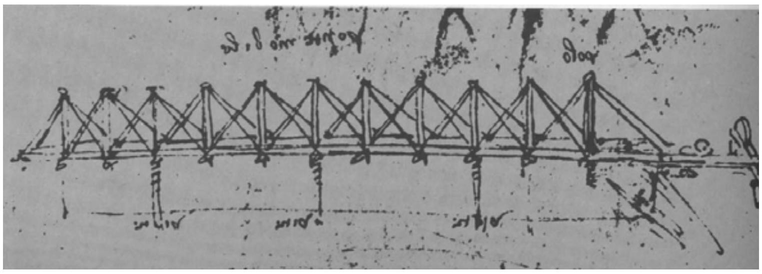

Figure 2 Bridge on intermediate poles, Leonardo da Vinci

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2.2 From the Rialto Bridge to Andrea Palladio

Among the precursors of wooden bridges it must be recalled the Rialto Bridge in Venice. The central parts of the bridge could rise for the passage of boats. Historical documents indicate that a big fire destroyed the bridge in the sixteenth century. The original image of this unique bridge has been handed down from a famous painting by Vittore Carpaccio of 1494 (Fig. 3).

Figure 3a Detail of Rialto Bridge from a painting of Vittore Carpaccio, 1494

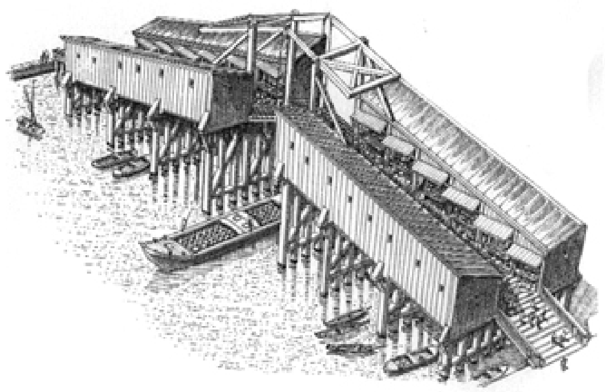

Figure 3b View of Rialto Bridge in Venice from a from a drawing of F. Corni

The kinematic stability, the property of being strictly non-deformable under the graphic statics, is the characteristic of the bridge over the Cismon of Andrea Palladio. Here the trusses and triangular meshes constitute an example of extraordinary conception. The structural scheme is very similar to a truss and is based on a rational net of triangular meshes, therefore non-deformable (Fig. 4).

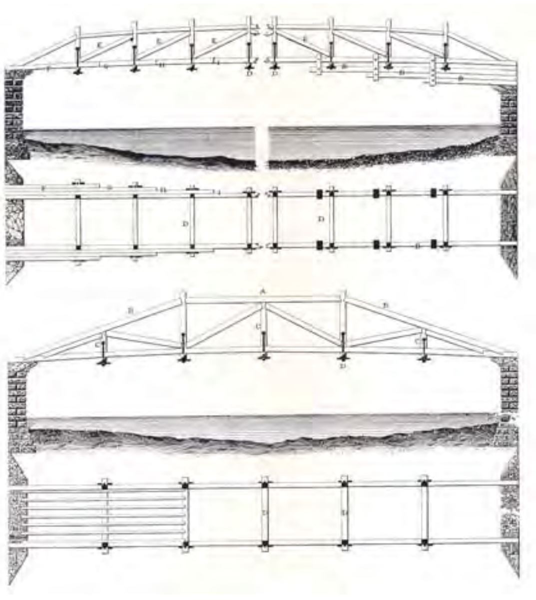

Figure 4a Palladio Bridge: the first invention or the bridge with a variable section (top), the bridge over the Cismon River (bottom)

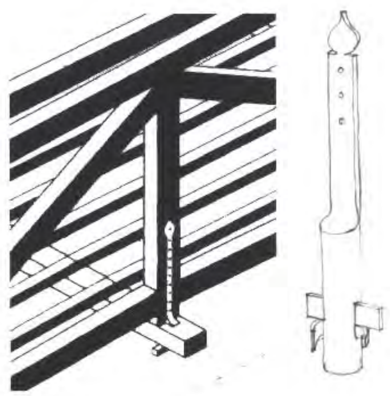

Figure 4b Detail of the connection between a colonnello, the longitudinal and transversal beam by an arpese (from G. Tampone, 2000)

Another wooden bridge designed by Palladio is the famous covered bridge of Bassano del Grappa. Repeatedly destroyed and rebuilt, it presents a much more traditional structural typology with rigid frames on both sides with and trusses placed transversely to support the roof (Fig. 5).

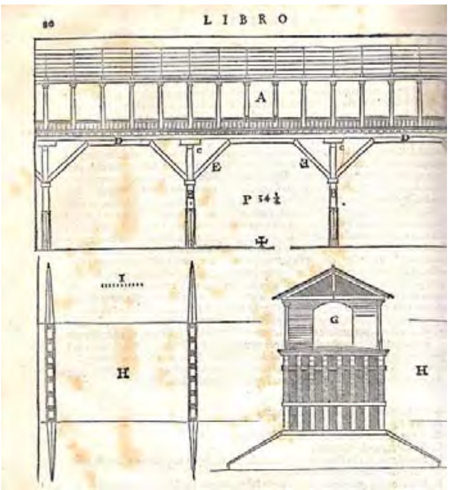

Figure 5 Brenta Bridge in Bassano, Palladio, 1570

Figure 6 Photo of the Brenta Bridge in Bassano nowadays

The functionality of the typology to connect the two sides of the river and at the same time the elegance of the building, are the result of the absence of a number of struts and of the harmonious relationship between the size of the struts forming the structure. In addition, the use of modularity and prefabrication allows a faster construction phase: disassembling and eventual replacements are simplified by such characteristics.

3 Developments and transformations between 18th and 19th centuries

Timber girders then diverged more and more from the schemes of Palladio, strongly increasing (and often disorderly), the number of girders of the reticular scheme in a constant attempt to couple the work of the beam to the work of the arc.

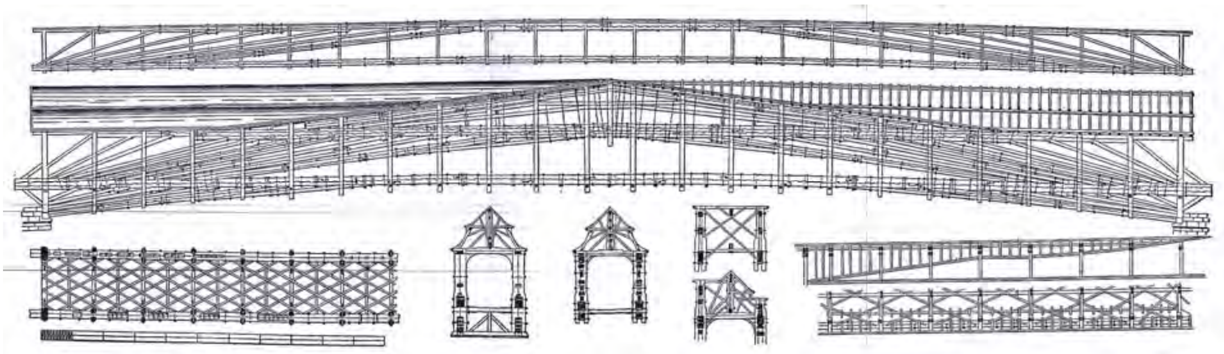

Figure 7 Bridge over Rhine in Schaffhausen by Grubenmann

The bridge over the Rhine at Schaffhausen in 1757 by J. Grubenmann is among the unique experiences realised with this typology. This bridge proposes a hybrid arc-truss structural configuration (Fig. 7).

A special case is the structural scheme of the Bridge of Westminster (1738) in which the main trusses are joined with the extensions of the reticular scheme of the girders, forming a statically indeterminable structure, very stiff, but of very uncertain calculation (Fig. 8).

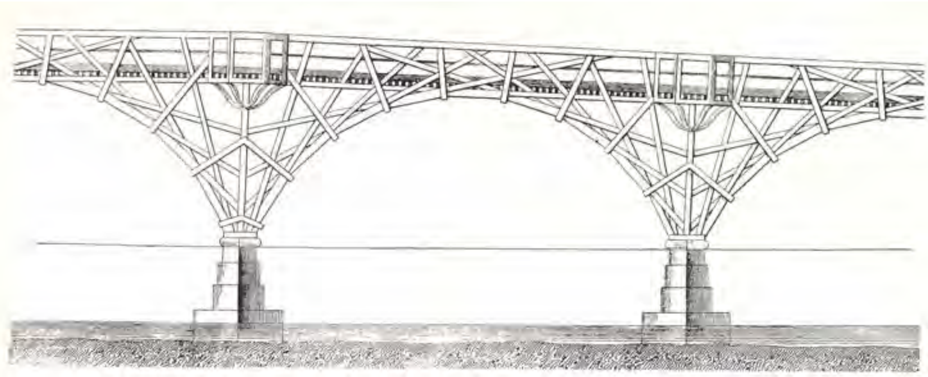

Figure 8 Bridge of Westminster, 1738

Only after 1830 in the United States appeared the first timber trestle railway bridges in which the stiffening elements, longitudinal and diagonal rafters, had substituted the arc, assuming the main load-bearing functions. This typology imposed itself as the conclusion of an evolution of the arch bridges in which the stiffening elements were formed by diagonals strongly interconnected.

In this evolution it stands out the Town Truss patented in 1820, a truss with dense diagonal trestle (Fig. 9). This scheme was realised with struts of relatively small size: a truss of internally highly hyper-static, attenuated or even cancelled by the deformability of the material and of the connections that led to balance of the distribution of loads.

The first realisations (1843) of Town bridges (Long and Howe) are made of steel but are imitations of American timber bridges. The physiognomy of the trestle truss appears completely defined.

Already in the late 19th century steel, for material and typology of connections, has brought back reticular schemes to a simplification and rationalization of the tasks of the various struts (Fig. 10).

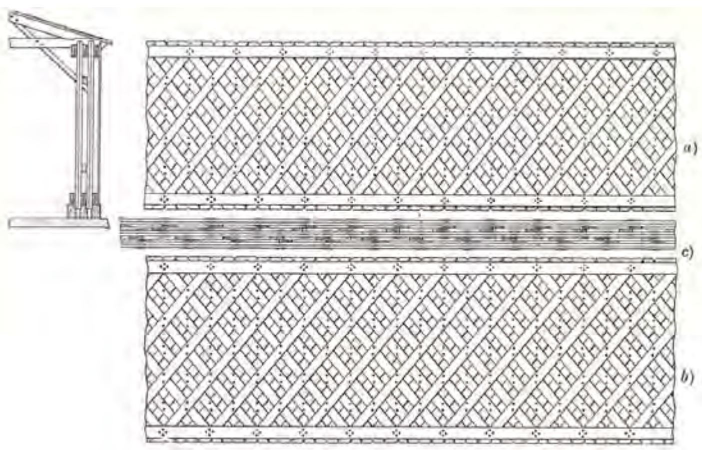

Fig. 9 Town Truss according the original scheme: a. external view, b. internal view, c. top view

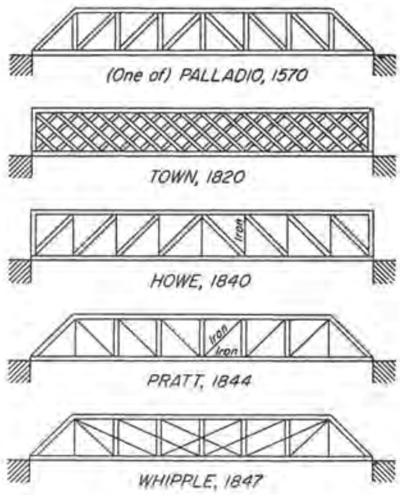

Fig. 10 A variety of truss types employed in bridges

4 Results and discussion

From this analysis it results how the current technique and technology of new wood derivatives, is oriented towards simple and clear truss schemes, with reduced internal statics, until coming back to those isostatic structures proposed by Palladio. In this framework is carried out a comparative analysis with recent bridges that have a structural scheme that can be referred to the Palladian scheme.

An example is the bridge over the river Glomma near Flisa in Norway (Fig. 11). The total span is 181.5 m with the central truss of 71 m. Two bowstring trusses (trusses with triangular mesh with upper arch), are supported at one end by a cantilevered beam (beam with two projecting parts and upper parabolic profile), and together they create a Gerber beam on four supports. This timber road bridge was realised in 2005, replacing a metal one that became obsolete due to insufficient width of the roadway.

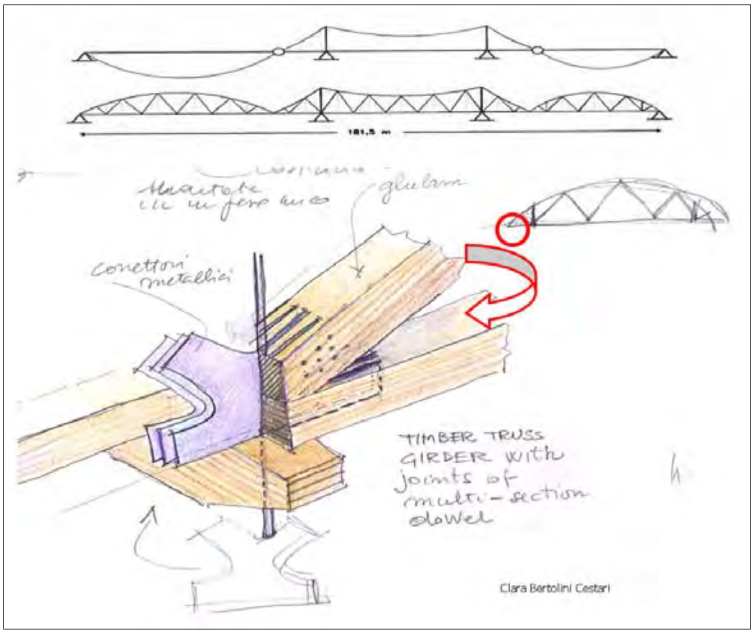

Fig. 11 Scheme of the Flisa bridge (Norway) in glulam (drawing C. Bertolini Cestari)

Glulam technique has re-opened the interest for wooden bridges, truss bridges, arch bridges (i.e. Austria, Switzerland, Finland) and for the latter and with this technology it is possible to cover spans of more than 100 m. In the presence of specific conditions as the need of lightness on the supports, or the urgency of the construction, glulam finds practical application possibilities. Furthermore, the possibility to realise an entire truss in the factory and to transport it and place it in a very short time, are another source of interest. Given the current relationship of needs/performance of glulam, one of the most important innovations of the sector can be found in the possibility of realising a complete work in the factory and place it *in situ* fully assembled.

The technology of glulam and of wood-based products has boosted the use of this material, quickly overcoming design and disciplinary gaps in comparison to other materials. This has involved also some loss of technical rules and standards that can be recovered when it will be established a new practice, constituted also by the critical review of the structures built in those countries where the tradition of building in wood did not have interruptions, i.e. like Switzerland.

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Historical timber bridges in Poland

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Summary

The paper presents some chosen historical bridges from Poland and their basic construction solutions and protection. The authors point methods used to preserve these relicts. The paper shows also Polish guidelines concerning the building of bridges from the beginning of 20th century, including guidelines of building war bridges.

Keywords: Historical timber bridge, timber engineering, timber structure

1 Presentation of Polish timber bridges

The remains of the oldest bridges in Poland were discovered thanks to the underwater archeological research. The dendrochronological dating of the preserved elements of the timber constructions allowed to define the age with an accuracy of up to one year.

In Poland, the oldest known timber bridge is the bridge leading to the settlement in Biskupin. According to the dendrological research made in 90's of the 20th century by Professor Tomasz Ważny, the oak timber used to build the Biskupin settlement is dated to 744-722 BC. The research showed that about half of the timber was acquired during one winter at the turn of the years 738/737 BC.

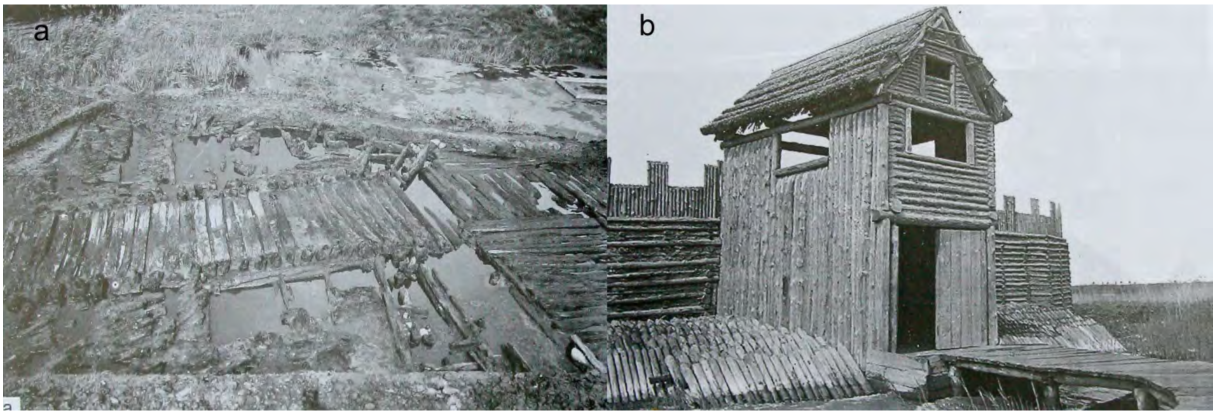

Figure 1: a) Remain of the bridge close to the gate (1946; b) First reconstruction of the gate and bridge, 1948 [1]

Three meters wide and 250m long bridge led through the shallowest parts of the lake to the settlement in Biskupin. The current peninsula was once an island. Zdzisław Adam Rajewski described this bridge in 1938 [2]: "The bridge was built in this way, that oaken columns stuck into the sand of the lake bottom, had heads in a form of rectangular tenon. 2.8m long and 30 cm wide beams with mortises were situated on the tenons." Even back there in the old times obliquely driven piles were used as a security against floe. After the discovery of the settlement, the researchers observed the necessity of maintenance of the timber elements without losing moisture. The drying beams were peeling and losing their construction forms, cuts, etc. Back in 1937 [1.14] the archeologists tried to preserve the wooden parts which were situated above the water by covering them with paraffin colored like wood. About 1000m^2 of the wood was conserved this way, although this was only a temporary solution. The problem of the effective timber protection returned after war. For some time an electrokinetic method was being taken into consideration [14], but this method also didn't offer 100% security.

Two parallel rows of oaken piles were discovered in Gągniewo lake near the town Nętno as the remains of the bridge from 948 –949 and 962–965 [4]. The piles had diameters about 10 cm and they were spread over 6.5–7 m. Some fragments of the superstructure of the bridge construction were found near the piles – beams with mortises (Figure5). About 30–40 m from the place where relicts from 10th century were found, were also discovered

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the remains of much older bridge. Dendrochronological dating stated that the wood for this construction was cut in autumn 673 or in winter 673/674. The examined piles had diameters 10-20 cm.

Another example is a bridge in Przełazy on the lake Niesłysz. 127 piles and the remains of the transverse construction were discovered in the area of location of the bridge. Irregularly situated piles were made of full or cut in half oak and pine logs. Most of the piles were located over the width of 5.2 m, the maximal spacing was 6 m. Figure 2 b) shows the position of elements found on the test pit nr 6 (3x6 m). Description of the elements in the picture contains dimensions and dates based on the dendrochronological tests. The research allowed to define that the wood used for this bridge was cut in 844-847 and in the 80's of 9th century. It is one of the oldest known constructions of the Slavs.

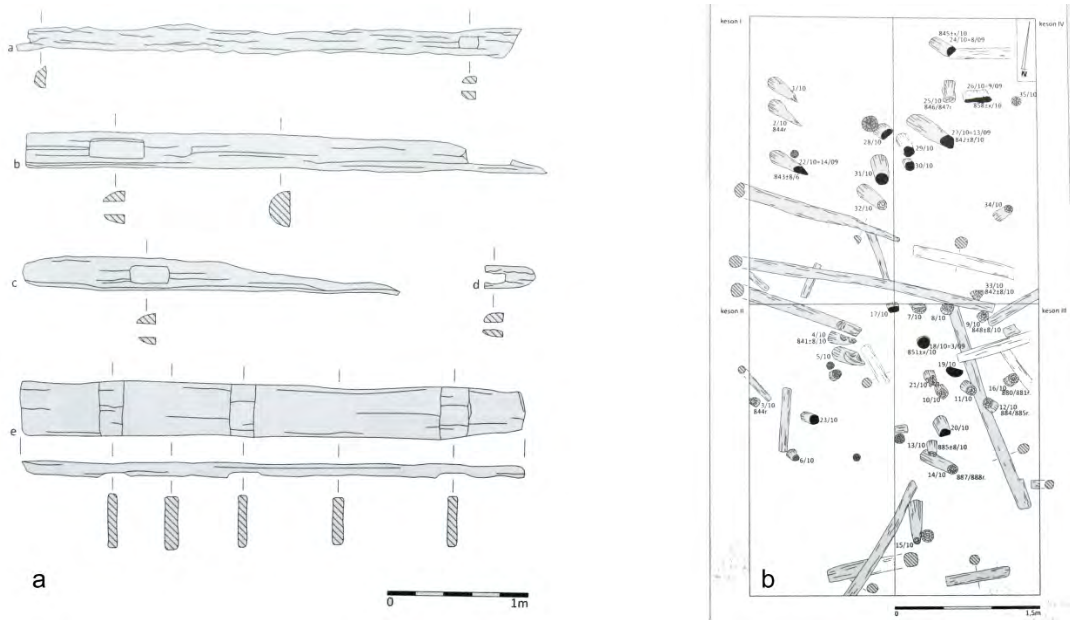

Figure 2:a)Niętno b) Przełazy, relicts of bridge [4]

The longest bridge of those times in Poland was built in Toruń in years 1497-1500. Its length was about 600 m and it made this bridge one of the longest in Europe. The bridge had a simple construction based on the piles and plank deck. The piles driven close to the bridge at an angle (izbice) were installed as a protection from the ice. [13]

The next well known Polish bridge was a bridge in Warsaw built by Erazm Cziotko from Zakroczym. The bridge contained 22 spans (22-24 m) supported on 18 wooden posts and 3 mobile supports. The pillars were made as frames of oak piles with a diameter of 30 cm, anchored at a depth 6-8 m. The whole length of the bridge was estimated to be about 500 m. The roadway was 6 m wide, and the clearance was more than 4 m. Its assembly was done using boats and rafts. The bridgeheads were built as a sheet pilings made of oak piles, filled up with stones up to the level of the roadway. This bridge was also protected against ice. [10; 13]

The first information about the bridge in Szczecin comes from 1283. Due to its length (over 100 m) it was called Most Długi (Lange Brücke, pons longus). The bridge was rebuilt in 1820 as drawbridge - with length 105 m, width 6.6 m and with two additional gangways 2.4 m each. The raised spans of this bridge allowed free sailing. Most Kłodny was a drawbridge already in 1624 (as shown on Figure 3). The drawings and pictures from 19th century show that Most Długi, as well as Kłodny Bridge had a similar construction at that time. The difference was in span of the middle part (The Long Bridge – 11.22 m, Kłodny Bridge 12.20 m) and in the clearance (The Long Bridge – 3.58 m, Kłodny bridge - 3,28 m). It was not until the beginning of 20th century when the timber bridge was replaced by a steel bridge. [5; 6; 7]

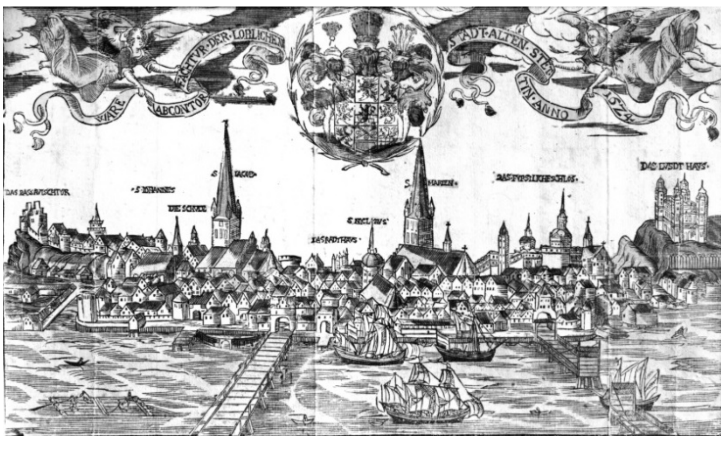

Figure 3 Most Długi (Lange Brücke) left and Most Kłodny (Baumbrücke) right, panorama of Szczecin, 1624 [16]

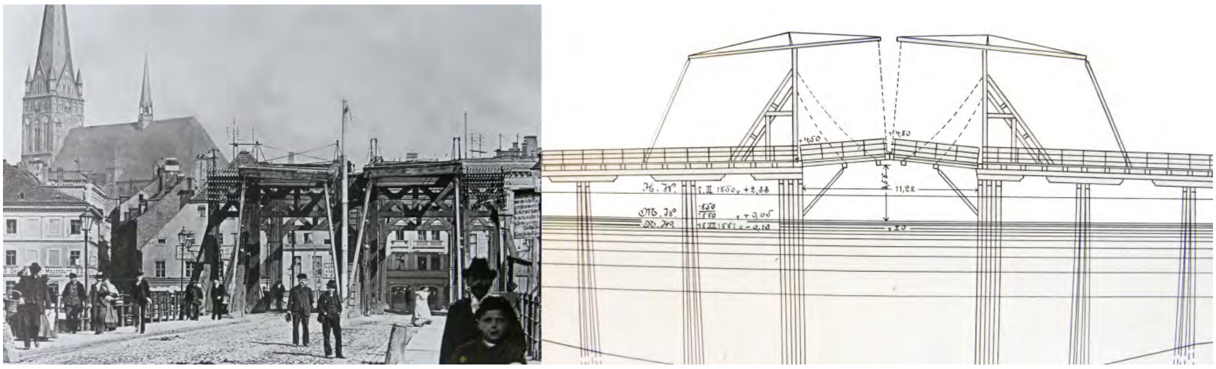

Figure 4 Most Długi (Lange Brücke) Photo ca 1890 [16] and drawing 1891 [15]

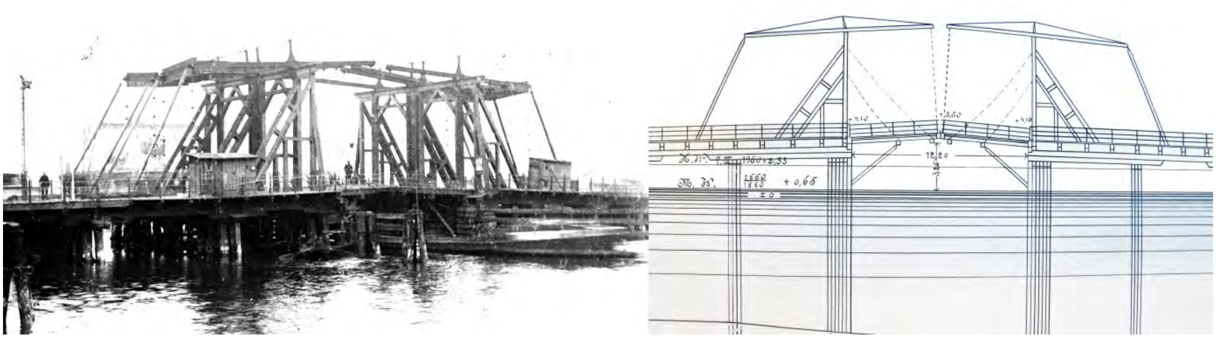

Figure 5 Most Kłodny (Baumbrücke) Photo after rebuilding in 1874 [16] and drawing 1891 [15]

Figure 6: Third of the presented timber bridges of Szczecin is Most Dworcowy built in 1850-1852. This bridge existed as a timber bridge up to 1898. Photo 1890, [15]

The durability of wood in the water might be comfirmed by the fact, that on the drawings of railway bridge over a small Regalica (dated 1911-1912) the foundation is on the wooden piles. The bridge itself is made of steel.

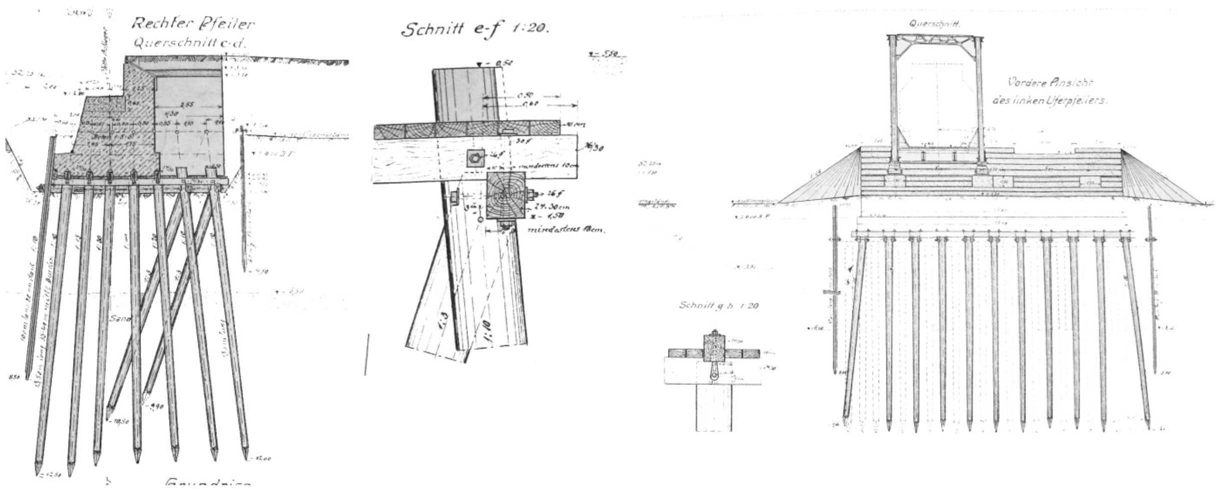

Figure 7: Most kolejowy (Eisebahnbrücke) on Mała Regalica, section and details [15]

The last bridge shown in this paper is the bridge in Wyszogród. This bridge was the longest timber bridge in Europe (with the total length of 1285 m). Built in 1916 as a temporary bridge it successfully served to 1999, due to frequent repairs. Currently only some parts of this bridge remain. The main construction of the bridge was timber and steel but 85-90% was made of wood. The bridge had 60 spans, the main girders were made of steel and founded on timber posts. The wooden floor beams (b=25 cm) situated with spacing 90 cm, deck with thickness 5 cm (upper) +10 cm (bottom). [12]

2 Applied material and rules of construction

Oak, pine, spruce, fir and alder (for poles) wood was used for bridge building. Soft wood was used for temporary constructions. The wood was cut during the winter. When cutting was done at other times the sap was separated and bark was removed. The wood was left in a well aired and sun protected place. The poles had the length of 18-20 m and the diameter 25-30 cm at the end. [3]

Deck was usually made of oak or beech planks (spruce or pine were not recommended). Single planks were 15 – 30 cm x 10 – 15 cm. In a case of deck made of double planks, upper part was recommended to be made of hard wood and lower of soft wood. Substructure used to build of both: logs and rectangular wooden beams. Sample longitudinal and cross section of the timber bridge is shown on Figure 8.

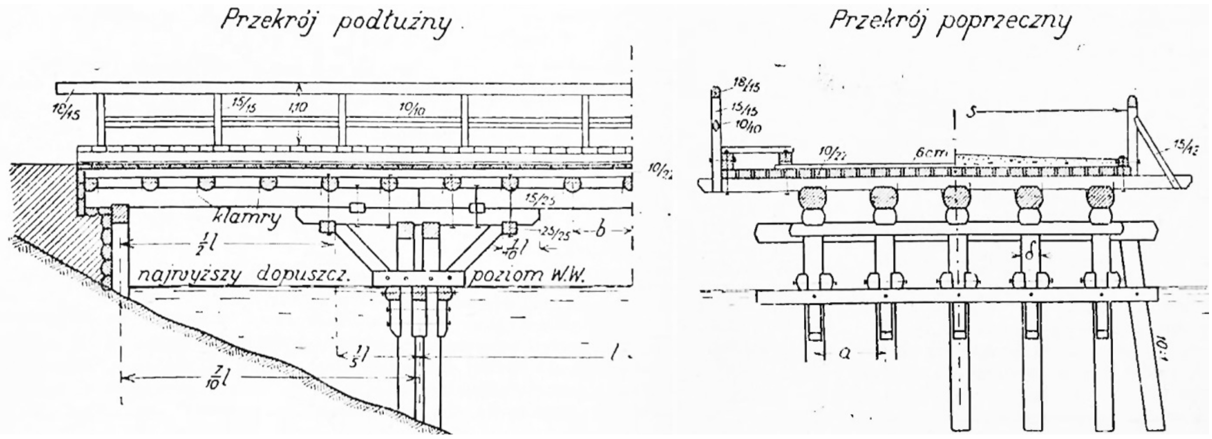

Figure 8: Longitudinal and cross section of timber bridge [3]

The interesting information and solutions content guidelines for the construction of the military bridges dated from 1920 [9]. Below the authors present selected assembly methods showed in mentioned position. Methods showed on Figure 9-11 concern the assembly of bridge supports in the water – using auxiliary beams (Figure 9, 11) or a boat (Figure 10).

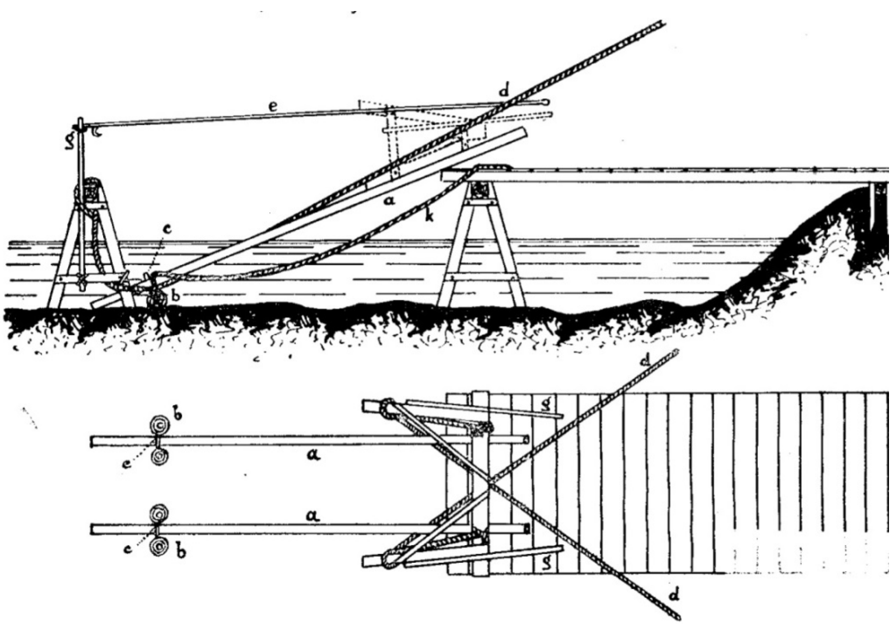

Figure 9: Installation of the support using two leading, auxiliary beams and lines [9]

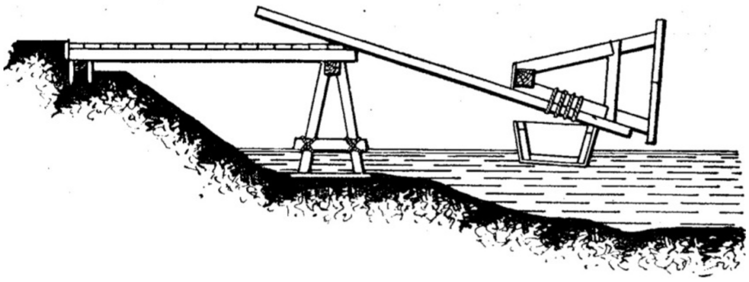

Figure 10: Installation of the support using boot [9]

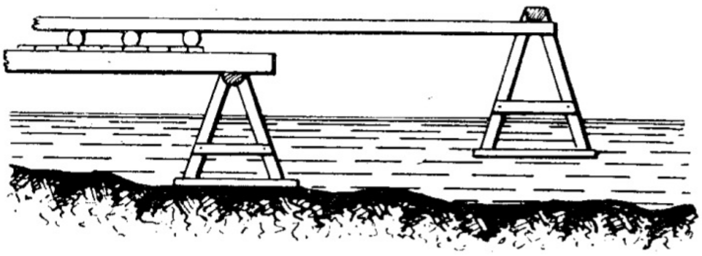

Figure 11: Installation of the support using log beams. Beams shall be minimum 30 cm thick [9]

Another interesting information concerns tabulated rules and data necessary to install piles using hand pile driver. Two pile drivers were used to install the piles, first light ones followed by heavy ones. These necessary data were obtained by tests and included the amount of time and the depth of installation. The mass of the light hand pile driver (Table 1) was 35 kg and the heavy hand pile driver weighed 55 kg. For example to install a pile with diameter of 10 cm in a depth of 84 – 97 cm3 cycles were necessary with 25-30 lighter hand pile driver strokes followed by one cycle of 25-30 heavy hand pile driver strokes. This operation took 15 – 20 min. Time and depth depended on the ground.

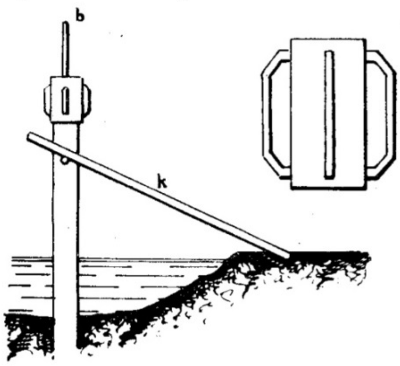

Figure 12: Hand pile driver (made of thick wooden block with a length 1,5m) [9]

Table 1: Rules of installing piles using hand pile driver [9]

Pile diameter [cm]	Number of cycle of 25-30 strokes (light hand pile driver)	Number of cycle of 25-30 strokes (heavy hand pile driver)	Embedment depth [cm]	Time [min]
6	4	---------	84 – 99	6 – 8
8	3	1 – 2	94 – 122	15 – 20
10	3	1	84 – 97	15 – 20
12	4	1 – 2	91 – 102	---
15	3	3	94 – 102	12 – 18

Up to 1900 most parts of bridges were built of untreated timber. Then it was started to protect the wooden structure using zinc chloride, copper sulfate or by the tar coating. Piles were protected by surface charring. Most important was to guarantee a ventilation of the timber elements and to protect the ends of beams against humidity. One of solution was installing of a tar-coated board at the end. The present condition of the remains of historical/prehistorical bridges is very strongly correlated to their storage in a combination of water and mud. The remains of Biskupin and the bridge leading to the village as well as the remains of other bridges survived hundreds of years because they were completely covered by water and mud. [1; 11]

A common way to protect deck was clay coating as it protected the construction against fire. The railway bridge decks were covered by ca 5 cm layer of gravel for fire-protection. In addition there were very strong rules concerning fire safety in the past. Police situated in a gate protected the timber bridges by supervising (for example at the bridge in Warsaw). Water tanks or a minimum number of barrels with water were stored close to the bridge. [8; 9; 11]

3 Conclusion

Often wood is being regarded as a secondary building material with a low durability. In reality it can survive many centuries – like other well-known building materials. Certain regimes and rules should be kept at each stage of the building process. Fire and ice are most dangerous for timber bridges. Fire and ice resistant bridges can be designed today as it was already done in the past.

Weak points of timber bridges are exposed connections and other places, where accumulation of water is possible. The combination of historical experiences, results of archeological tests and modern engineering knowledge led to improve solutions and building safe and durable construction of timber bridges. Professor Andrzej Pszenicki wrote in 1938 that durability of a timber bridge structure can be taken as 15-25 years [8]. Today we discuss much longer periods, amongst others also thanks to the experiences we have made with historical timber bridges and the respective lessons we have learned.

4 Acknowledgement

The author would like to acknowledge Mrs A. Grossman, Manager of the Research Department of the Archeological Museum in Biskupin for all supplied materials. Furthermore our gratitude goes to the National Archiv in Szczecin, National Museum in Szczecin and to: Mrs B. Kozińska, Mr M. Słomiński and Mr G. Solecki.

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The vehicular bridge over the Torrente Branega in the buffer zone of Genova Pra

Giovanni Damonte[1], Gerolamo Stagno[2], Chiara Tartaglia[3], Alberto Carlascio[4], Emilio Costantino[4]

Summary

In the early 2000s the port area to the west of the city of Genoa was the subject of significant environmental recovery and expansion of the docks. In particular, it realized the important environmental intervention of the range of Pra , where in addition to a green built on part of the ' site of the former railway that sea clutter is reduced , it produces the channel of calm division of the island Container depot and in particular the stage of boating that will soon be the subject of an important event in the International boating World. The architectural work of the functional link between area and island green logistics is the wooden bridge at the mouth of the River Branega

Key words: vehicular bridge, buffer zone, Genova Pra

1 The work and his degradation

This is one of the few examples of wooden work carrier realized in the marine environment in recent years so even if modest worthy of particular attention because of its material characteristics. In particular, it is placed in a park area of significant public frequency as in the green area so popular with families especially on weekends.

The structure is of type gantry to a span, given the modest light load to overcome. It is made of laminated fir with metal connections , in particular the nature of the soil makes it necessary reinforced concrete works for the shoulders and indirect foundations.

The maintenance checks begin to thermie course of the work and is already showing the first decline in the handrails of diseases and especially in horizontal surfaces in spite of the protective systems applied during the work. But it is precisely the marine environment to ask questions about the details of the durability of the work. Recently several surveys is evaluated after a phase of more thorough check on the durability of the materials in their various components especially in the points of contact between wood and conglomerate and wood and steel.

This experience has for its object the Diagnostic Imaging performed with particular attention to the evaluation of the durability of the material that has led to a major overhaul of the maintenance protocol. The use of non-destructive methods is part of the diagnosis and the subsequent phase of classification of wood and Testing Structural Durability and also in the light of recent developments in the Statement.

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[2] *Technical Architect Diagnostic Studio Genoa : Diagnosis of the state of conservation*
[3] *Engineer Genoa City Heritage Area Maintenance: Maintenance Project*
[4] *Collaborators of Technical Architect Diagnostic Studio*

2 Brief technical description

The structure is built in the first years of the XIX century, and is inserted into the accommodation at sea with the construction of a tourist port of the promenade of Pra in Genoa and the subsequent international sporting event of the World Rowing. The vehicular bridge is located at the mouth of the river Branega and starts the stilling basin seat of the next race field.

The elements of the bearing structure are laminated wood:

- the struts are constituted by two elements side by side respectively with 7 and 11 laminae in composite system
- the longitudinal beams, each consisting of two elements connected in the center line, are constituted by 15 lamellae
- the secondary elements transverse who bear the deck are glulam too and they are made with 9 elements
- the uprights of the handrail and the one in the middle are solid wood as well as those of vertical and horizontal handrail
- the deck is made from planks lengthwise spaced on two parallel courses. The base plates are made of reinforced concrete and weigh on indirect foundations.

All the elements present signs of a preservative treatment

3 Diagnosis: Methodology Applied

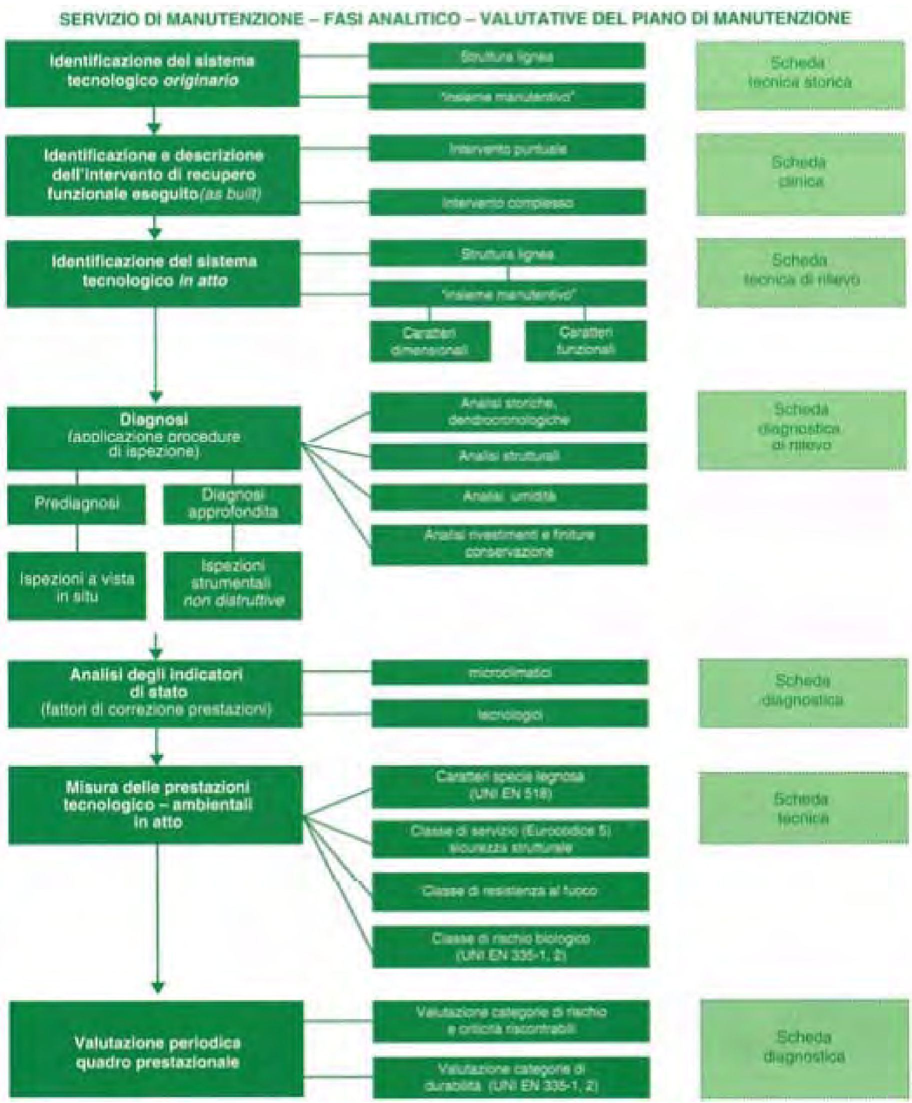
The survey is required as part of the Maintenance Program and developed as part of the plan drawn by the Ente owner Genoa City Council Heritage Department. The methodology is contained in the Scheme of the General Programme of Knowledge act to keep the structure in place and checking the durability is one of the phases currently underway in developing. For the main structure of glued-laminated and solid wood elements in order to determine the diagnosis we had to proceed by considering two possible conditions:

- the possible degradation of the wood
- the conditions of the adhesive connection between the slats

From a visual examination of laminated beams it was found that the lines of bonding between the wooden slats do not exhibit delamination, cracks present, of which in the sample it was measured the depth, are superficial and there are no especially on the outer side of the elements perimeter, it was considered not necessary to run tests on adhesives. For the diagnosis of the wood has been applied to a method comprising the non-invasive diagnostic tests, applying, even in the presence of laminated wood, methodologies commonly performed on solid wood in order to check the status of the material and links between the tables.

We proceeded in detail in:

- technological inspection to ascertain the characteristics and types of structural elements and in order to plan the next phase diagnostics to be applied on critical cases where the inspection does not allow you to make a judgment on the class assignment
- microscopic investigations for the determination of the species with analysis conducted in the laboratory on samples taken
- phase structural diagnostics with non-destructive testing on site for determining the main mechanical parameters



4 Protocol of technological inspection:

Constitutes the step of PRE-DIAGNOSIS and is performed with a careful reading of the elements constituting the bridge site including particular attention to delaminations between the glued elements. Preliminarily we proceeded to an analysis of the design documents and of the documentation available.

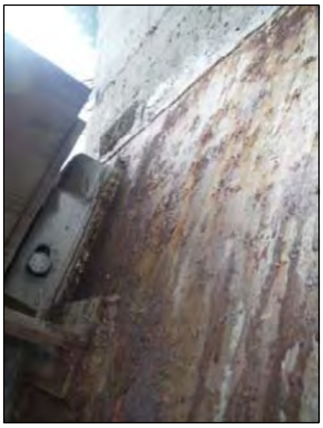
In particular it has been applied to a sheet of Experimental Classification Vista drafted incorporating the provisions of the Regulations UNI 11119 "Protocol Inspection Technology" with those of the UNI EN 11135 1-2 "Classification of Wood River and New USE Trieste which describes the different disorders and diseases of the material The phase of technological inspection is developed to determine the characteristics and types of structural elements and in order to plan the next phase diagnostics to be applied on critical cases where the inspection does not allow you to make a judgment on the class assignment.

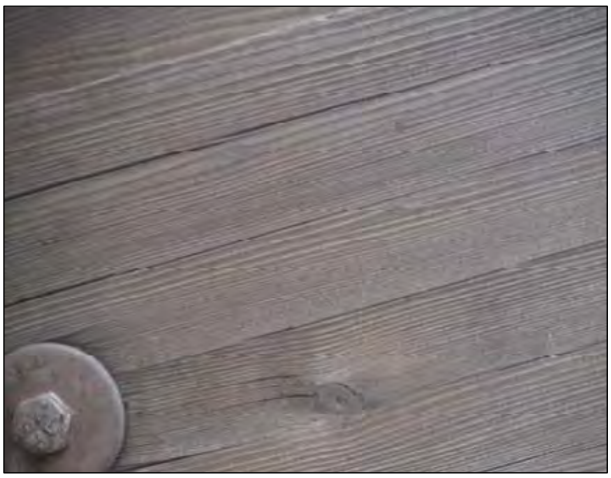
They were then sampled, and subsequently after preparation in liquid and Microtomico cutting of thin sections are then proceeded to the recognition of the wood species with reading the digital microscope on thin sections both radial and longitudinal. (look at photo n. 1-2)

5 Pilodyn testing

The instrument allows to estimate the level of degradation of the surface and the consequent reduction of the resistant section compared to the real section. The test consists in the insertion inside the timber of a metal tip with a constant and predetermined work (6J), the depth reached by the tip is read on a graduated scale placed in the metal casing of the instrument. The tests were performed on selected beams in the sample, the penetration depth of the tip of the instrument are less than 20mm, and can be considered normal for the wood analyzed, as experimentally determined values of the timber elements in a good state of preservation. (look at photo n. 3)



6 Test with wood penetrometer RESI F 400

The instrument allows to evaluate changes in wood density between "healthy" and "degraded" wood and then to make a diagnosis of areas of internal decay of wood material for the entire section. The resistance to drilling of the timber depends primarily on the density of the same and is one of the most important physical parameters of the material, allowing you to draw conclusions about the quality of the wood in a particular section.

In particular, we measure the resistance of the wood to the drilling performed with a spear tip (3 mm diameter) which passes through the entire section of the element. The response in terms of puncture resistance gradually that boring wood is reported to the appropriate cursor highlighted on a strip of paper chemistry (1: 1) that on a special unit to direct automatic storage. (look at photo n. 4-5)

7 Electrical resistivity

With this method we detect both environmental temperature and humidity of the test material in the site as indicated in the specification of the UNI 13183-2: 2003 "Humidity of a piece of sawn timber - Estimation by the electrical method"

The instrument used is the Gann Hydromett RTU 600, powered by a 9V battery and comes with probes with different functions:

- active electrode RF-T-28 for the measurement of temperature and humidity environmental (measuring range: temperature from -10 ° C to + 80 ° C - humidity from 7% to 98%)
- drive-in electrode M18 is equipped with nails that allow measuring deep into the wood, these nails can be long 4-6-20- 30 cm; the instrument must be calibrated on the ambient temperature, measured with a specific probe, and on the wooden species analyzed using the codes on the attached tables; the value obtained is directly the percentage of moisture in the wood
- ET50 temperature probe for measuring the surface temperature and depth in the material (measuring range from -50 ° C to + 300 ° C)

(look at photo n. 6)

Figure 1: photo 1

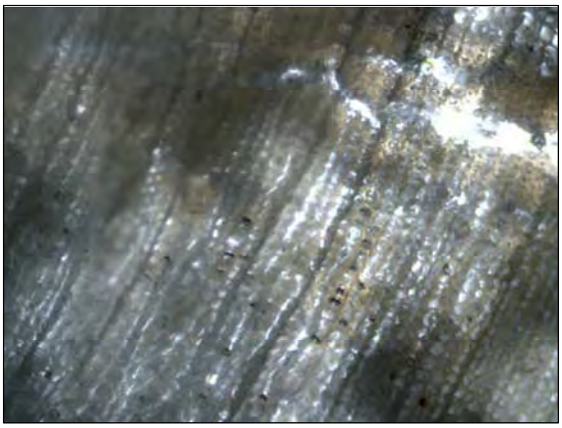


Figure 2: photo 2

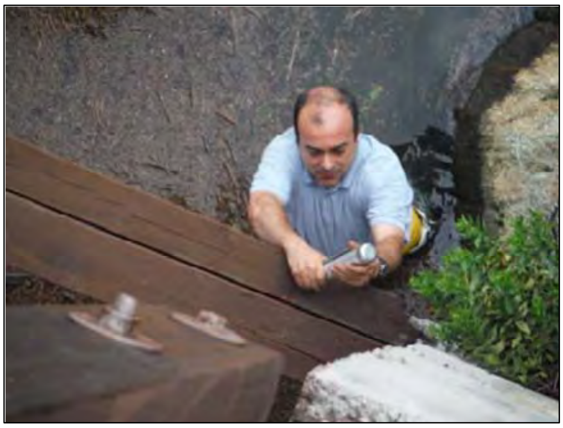


Figure 3: photo 3

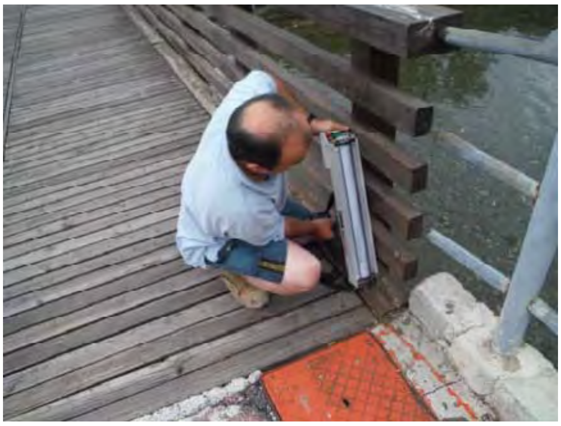
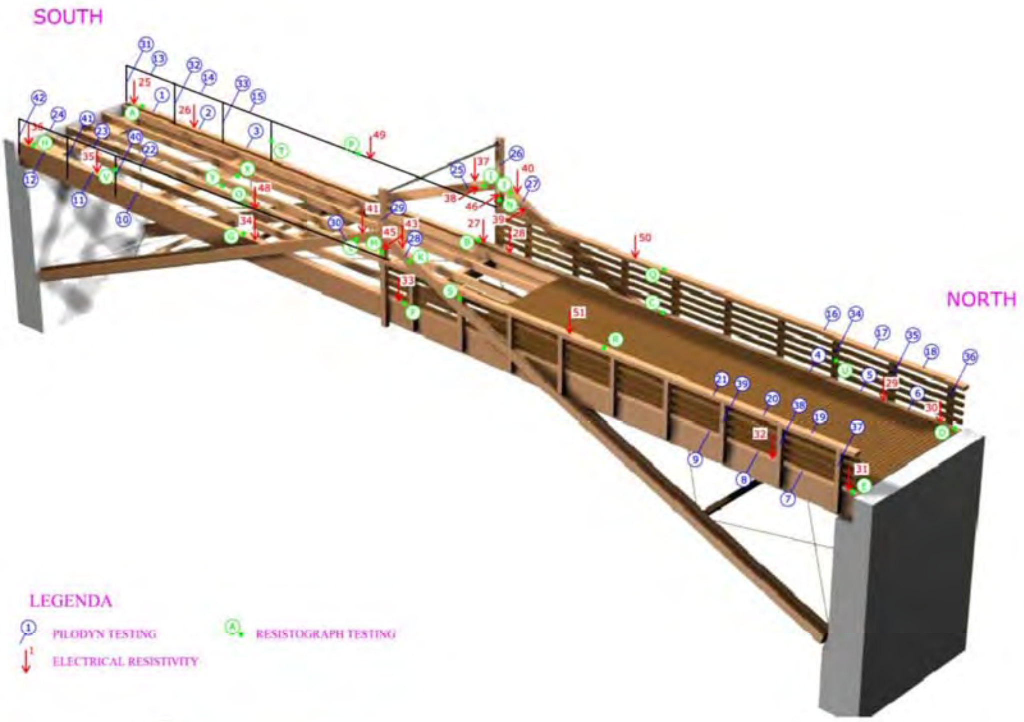
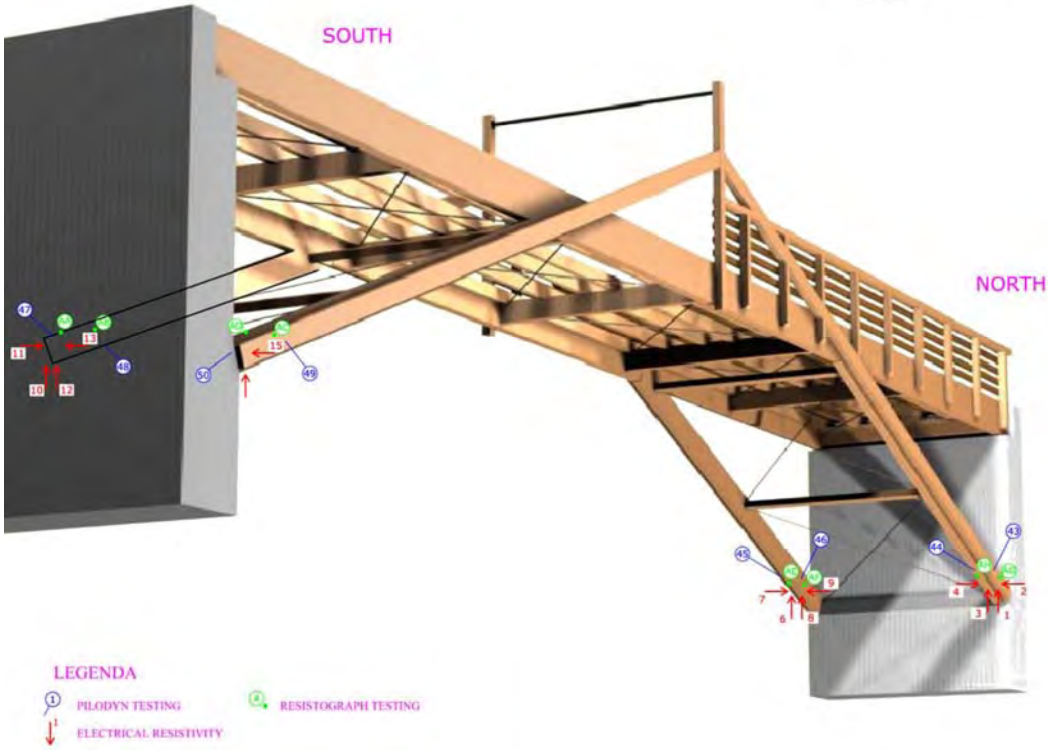


Figure 4: photo 4

Figure 5: photo 5

Figure 6: photo 6





TEST WITH PILODYN 6J STANDARD ON WOODEN ELEMENTS

Object: Bridge over the river Branega (Genova)	**Commissioner :** Comune di Genova	**Date:**
Elements : main beams	**Detectors:** Stagno-Carlascio	**Page:** 1

ELEMENT	WOODY ESSENCE	TIP DIAMETER	PENETRATION DEPTH (mm) TEST 1	TEST 2	TEST 3	TEST 4
1	silver fir	2,5	11			
2	silver fir	2,5	13			
3	silver fir	2,5	15			
4	silver fir	2,5	10			
5	silver fir	2,5	16			
6	silver fir	2,5	15			
7	silver fir	2,5	13			
8	silver fir	2,5	12			
9	silver fir	2,5	15			
10	silver fir	2,5	15			
11	silver fir	2,5	10			
12	silver fir	2,5	15			

TEST WITH PILODYN 6J STANDARD ON WOODEN ELEMENTS

Object: Bridge over the river Branega (Genova)	**Commissioner :** Comune di Genova	**Date:**
Elements : banister	**Detectors:** Stagno-Carlascio	**Page:** 2

ELEMENT	WOODY ESSENCE	TIP DIAMETER	PENETRATION DEPTH (mm) TEST 1	TEST 2	TEST 3	TEST 4
13	silver fir	2,5	internal 12	upper 18	external 11	
14	silver fir	2,5	15	15	14	
15	silver fir	2,5	15	18	12	
16	silver fir	2,5	14	18	19	
17	silver fir	2,5	25	15	16	
18	silver fir	2,5	22	15	15	
19	silver fir	2,5	20	30	16	
20	silver fir	2,5	14	16	40	
21	silver fir	2,5	20	11	17	
22	silver fir	2,5	15	14	11	
23	silver fir	2,5	33	12	15	
24	silver fir	2,5	19	18	14	

TEST WITH PILODYN 6J STANDARD ON WOODEN ELEMENTS

Object: Bridge over the river Branega (Genova)	**Commissioner :** Comune di Genova	**Date:**
Elements : struts and vertical elements	**Detectors:** Stagno-Carlascio	**Page:** 3

ELEMENT	WOODY ESSENCE	TIP DIAMETER	PENETRATION DEPTH (mm) TEST 1	TEST 2	TEST 3	TEST 4
25	silver fir	2,5	internal 15	upper 18	external 18	
26	silver fir	2,5	internal 18	south side 15	external 15	
27	silver fir	2,5	internal 15	upper 16	external 22	
28	silver fir	2,5	internal 18	upper 19	external 21	
29	silver fir	2,5	internal 18	south side 18	external 19	
30	silver fir	2,5	internal 15	upper 17	external 15	
31	silver fir	2,5	internal 18 P	south side 15 M	external 18 P	
32	silver fir	2,5	11 P	18 M	19 P	
33	silver fir	2,5	18 HP	28 M	19 P	
34	silver fir	2,5	18 HP	18 M	20 M	
35	silver fir	2,5	20 HP	18 M	25 P	
36	silver fir	2,5	20 HP	18 M	18 P	
37	silver fir	2,5	internal 15 HP	north side 20 M	external 16 P	
P = HP = M =	polished surface half-polished surface matt surface					

TEST WITH PILODYN 6J STANDARD ON WOODEN ELEMENTS

Object: Bridge over the river Branega (Genova)	**Commissioner :** Comune di Genova	**Date:**
Elements : struts and vertical elements	**Detectors:** Stagno-Carlascio	**Page:** 4

ELEMENT	WOODY ESSENCE	TIP DIAMETER	PENETRATION DEPTH (mm) TEST 1	TEST 2	TEST 3	TEST 4
38	silver fir	2,5	internal 18 M	north side 18 HP	external 20 M	
39	silver fir	2,5	15 HP	15 P	15 M	
40	silver fir	2,5	15 HP	15 M	15 M	
41	silver fir	2,5	15 M	15 HP	17 M	
42	silver fir	2,5	15 P	15 HP	18 M	
43 external strut	silver fir	2,5	lower 15	lateral 15	upper 15	head 35
44 internal strut	silver fir	2,5	15	15	15	
45 external strut	silver fir	2,5	17	15	20	17
46 internal strut	silver fir	2,5	20	15	15	
47 external strut	silver fir	2,5	20	20	18	40
48 internal strut	silver fir	2,5	20	18	20	
49 external strut	silver fir	2,5	15	18	20	22
50 internal strut	silver fir	2,5	18	16	16	
P = HP = M =	polished surface half-polished surface matt surface					

ELECTRICAL RESISTIVITY TEST

Site: Bridge over the river Branega (Genova)	Detectors: Stagno-Carlascio	Date:
Point location:	Material: abete rosso	Page: 1

POINT	AIR TEMP. (°C)	AIR HUMID. (%)	SURFACE TEMP. (°C)	WOOD MOISTURE (%)	SUPPORT HUMIDITY (%)
A	22,3	82,3		12,5	
B	22,3	82,3		25,9	
C	22,3	82,3		63	
D	22,3	82,3		21,8	
E	22,3	82,3		34,2	
F	22,3	82,3		26,8	
G	22,3	82,3		100	
H	22,3	82,3		70,3	
I	22,3	82,3		upper 14,3 int. 14,3 ext. 13,6	
J	22,3	82,3		15,6	
K	22,3	82,3		13,1	
L	22,3	82,3		13,3	
M	22,3	82,3		13,2	
N	22,3	82,3		14,9	

ELECTRICAL RESISTIVITY TEST

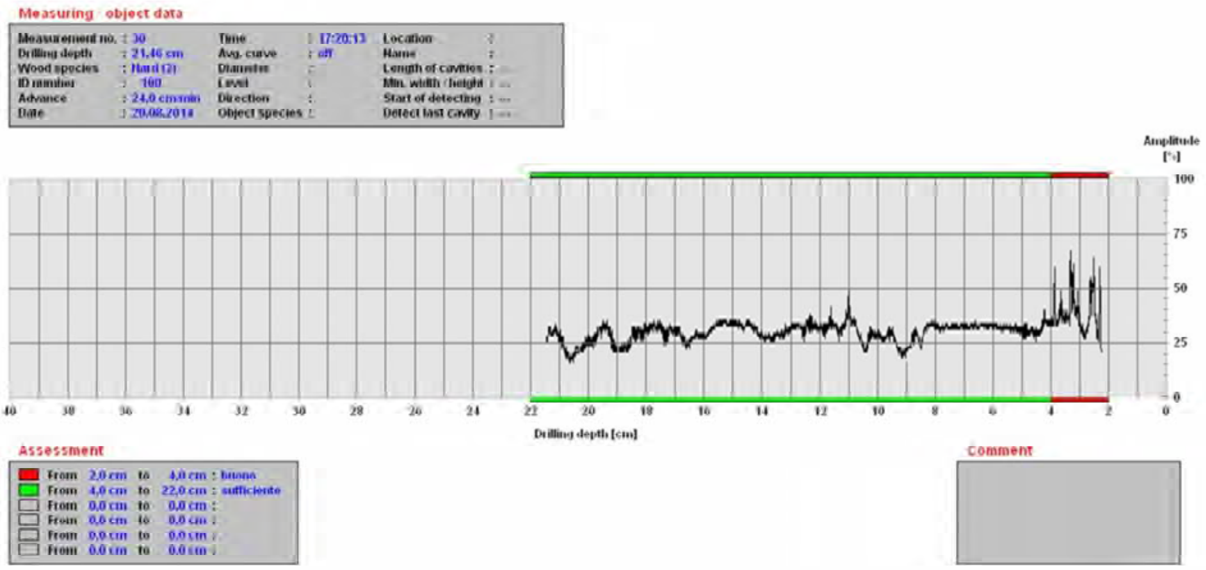
Site: Bridge over the river Branega (Genova)	Detectors: Stagno-Carlascio	Date:
Point location:	Material: abete rosso	Page: 2

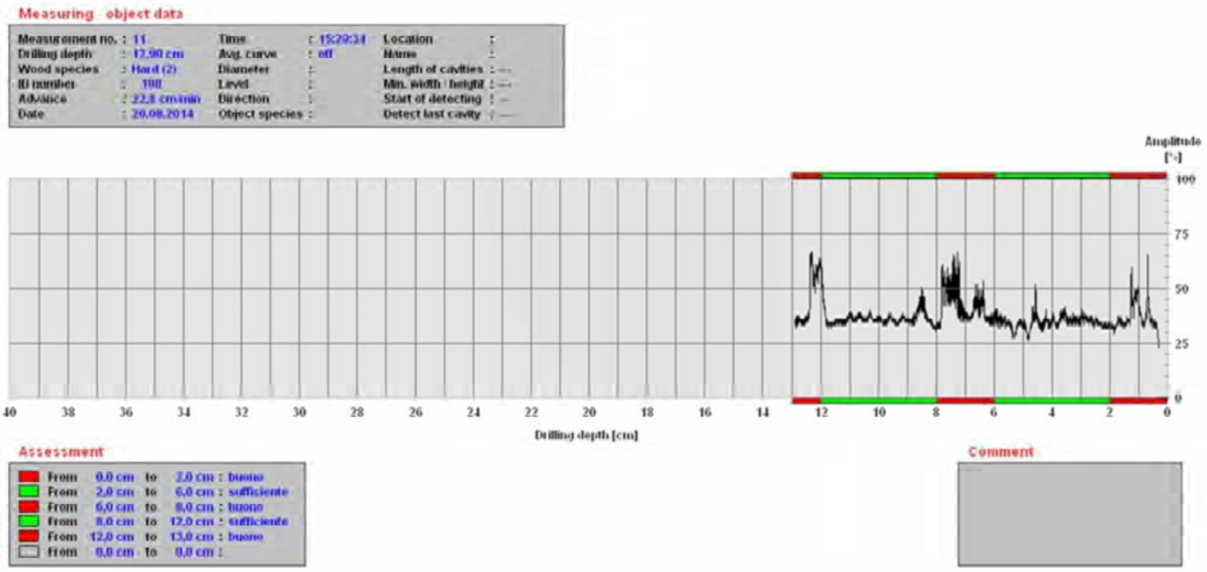
POINT	AIR TEMP. (°C)	AIR HUMID. (%)	SURFACE TEMP. (°C)	WOOD MOISTURE (%)	SUPPORT HUMIDITY (%)
O	22,3	82,3		16,1 unprotected area	
P	22,3	82,3		16,7 protected area	
Q	22,3	82,3		52,4 shrinkage crack	
R	22,3	82,3		62,3 decay	
vert. el. S from ext.	22,3	82,3		21,5 matt	
vert. el. T from ext.	22,3	82,3		17,2 polished	
vert. el. U from ext.	22,3	82,3		15,0 polished	
vert. el. V from ext.	22,3	82,3		16,2 matt	
X floor	22,3	82,3		12,8	
Y floor	22,3	82,3		17,0	
AA ext. Strut	22,3	82,3	21,9	lateral 15,6 upper 15,8 lower 15,3	
AB int. Strut	22,3	82,3	21,9	lateral 16,3 upper 16,3 lower 18,7	
AC ext. strut	22,3	82,3	23,7	lateral 14,3 upper 13,9 lower 16,0	
AD int. strut	22,3	82,3	23,7	lateral 22,1 upper 16,7 lower 17,8	mold

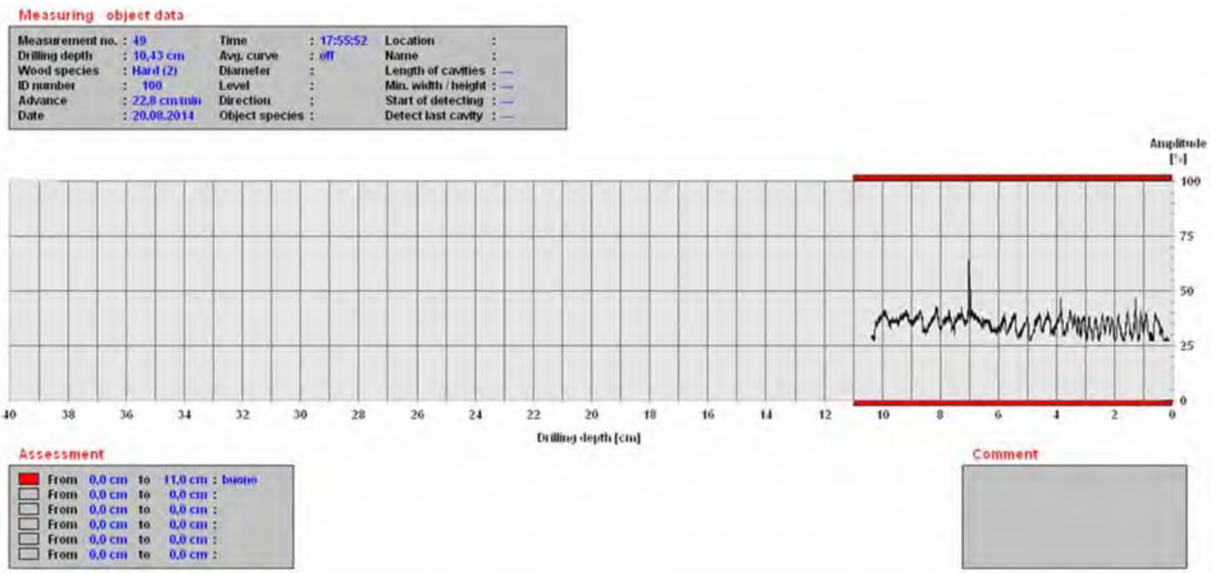
ELECTRICAL RESISTIVITY TEST

Site: Bridge over the river Branega (Genova)	Detectors: Stagno-Carlascio	Date:
Point location:	Material: abete rosso	Page: 3

POINT	AIR TEMP. (°C)	AIR HUMID. (%)	SURFACE TEMP. (°C)	WOOD MOISTURE (%)	SUPPORT HUMIDITY (%)
AE ext. strut	22,3	82,3	23,4	lateral 33,4 upper 21,8 lower 16,8	decay
AF int. strut	22,3	82,3	23,4	lateral 18,5 upper 19,3 lower 17,4	
AG ext. strut	22,3	82,3	23,5	lateral 70,5 upper 15,1 lower 15,5	
AH int. strut	22,3	82,3	23,5	lateral 18,3 upper 19,2 lower 17,2	


Measuring object data
Measurement no. : 30
Drilling depth : 21,46 cm
Wood species : Hard (2)
ID number : 100
Advance : 24,0 cm/min
Date : 20.08.2014
Time : 17:20:13
Avg. curve : off
Diameter :
Level :
Direction :
Object species :
Location :
Name :
Length of cavities :
Min. width / height :
Start of detecting :
Detect last cavity :
Amplitude [%]
Drilling depth [cm]
Assessment
From 2,0 cm to 4,0 cm : buono
From 4,0 cm to 22,0 cm : sufficiente
From 0,0 cm to 0,0 cm :
From 0,0 cm to 0,0 cm :
From 0,0 cm to 0,0 cm :
From 0,0 cm to 0,0 cm :
Comment


Measuring object data
Measurement no. : 11
Drilling depth : 13,90 cm
Wood species : Hard (2)
ID number : 100
Advance : 22,8 cm/min
Date : 20.08.2014
Time : 15:29:34
Avg. curve : off
Diameter :
Level :
Direction :
Object species :
Location :
Name :
Length of cavities :
Min. width / height :
Start of detecting :
Detect last cavity :
Amplitude [%]
Drilling depth [cm]
Assessment
From 0,0 cm to 2,0 cm : buono
From 2,0 cm to 6,0 cm : sufficiente
From 6,0 cm to 8,0 cm : buono
From 8,0 cm to 12,0 cm : sufficiente
From 12,0 cm to 13,0 cm : buono
From 0,0 cm to 0,0 cm :
Comment


Measuring object data
Measurement no. : 49
Drilling depth : 10,43 cm
Wood species : Hard (2)
ID number : 100
Advance : 22,8 cm/min
Date : 20.08.2014
Time : 17:55:52
Avg. curve : off
Diameter :
Level :
Direction :
Object species :
Location :
Name :
Length of cavities :
Min. width / height :
Start of detecting :
Detect last cavity :
Amplitude [%]
Drilling depth [cm]
Assessment
From 0,0 cm to 11,0 cm : buono
From 0,0 cm to 0,0 cm :
From 0,0 cm to 0,0 cm :
From 0,0 cm to 0,0 cm :
From 0,0 cm to 0,0 cm :
From 0,0 cm to 0,0 cm :
Comment

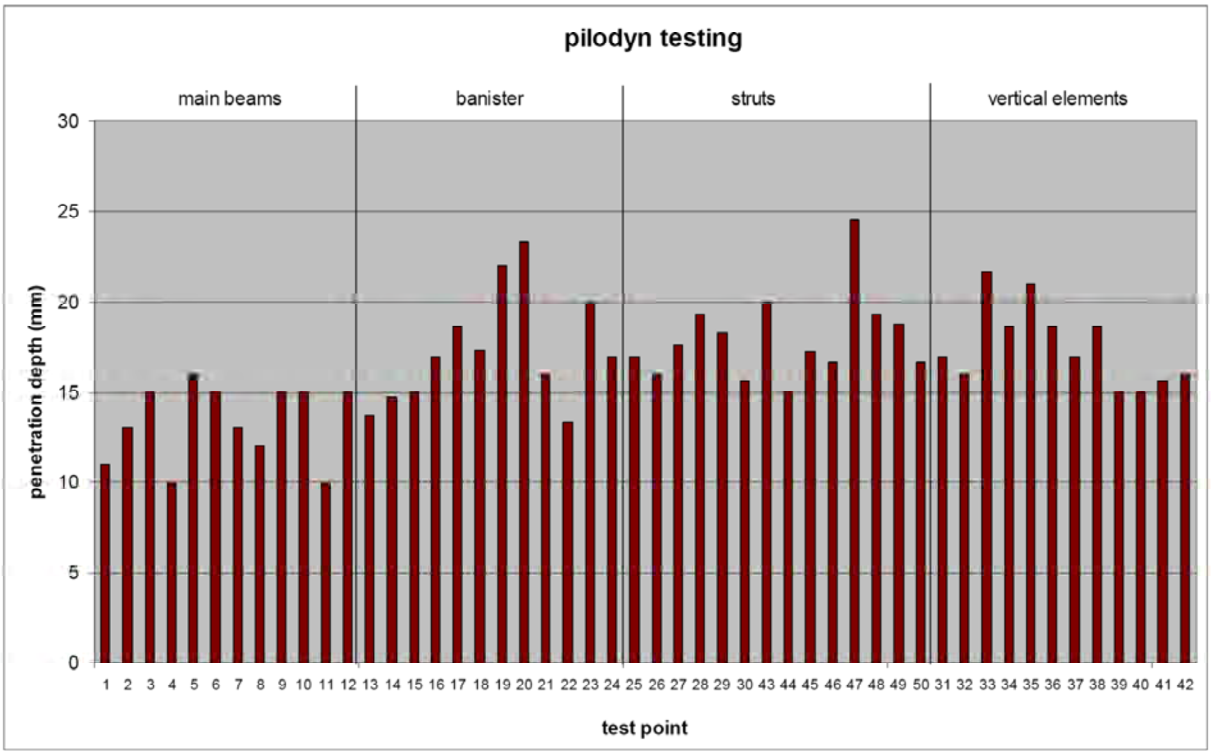
8 Diagnosis

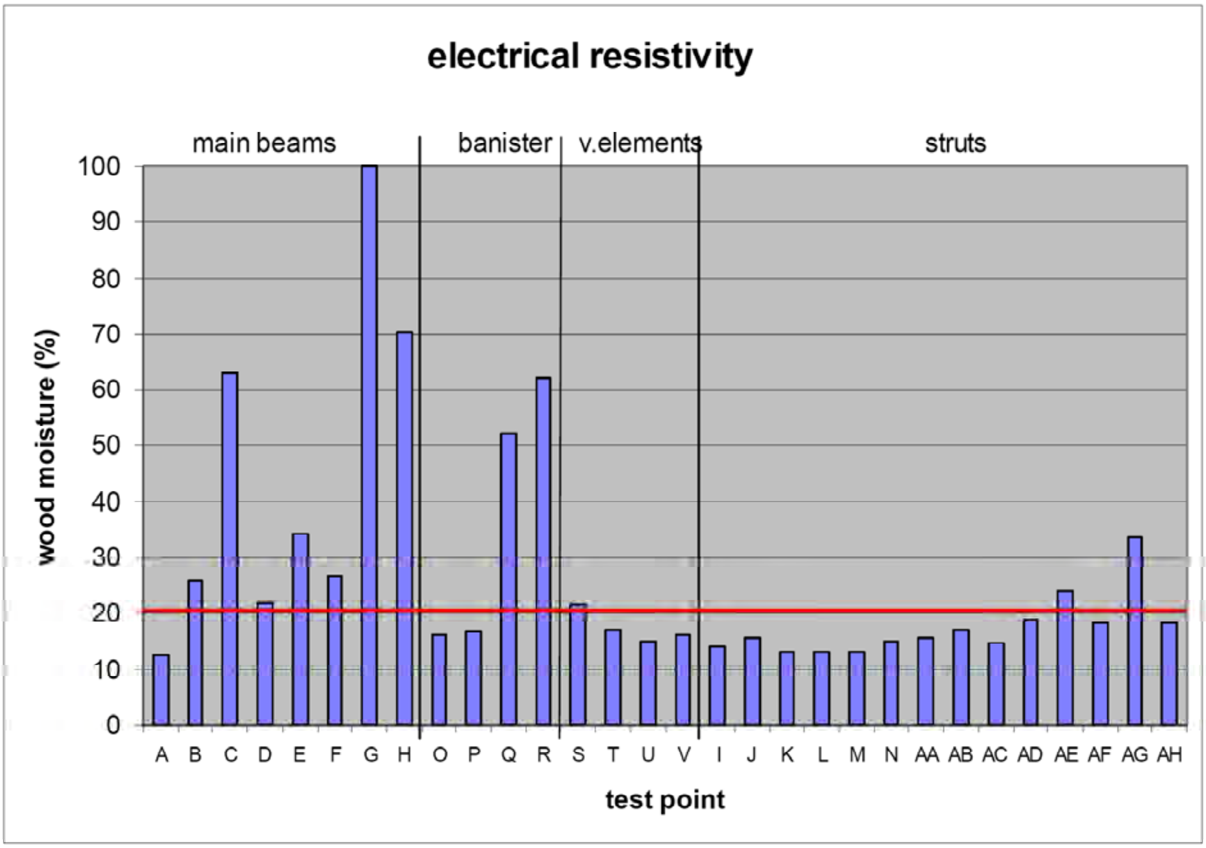
The Technological Inspection showed the onset of diseases mainly due to alterations and not to a defect in the wood; in particular, the layers of protective no longer polished but disintegrated by water and ultraviolet radiaton revealed the activation of mechanisms of biological degradation such as:

- Chromatic alteration: With color phenomena gray (blooming) and subsequent staining fungi surface of green which do not penetrate inside due to surface development of mycelium caused by Penicillium, Glicocladium, Aspergilla, Mucor, Rhyzopus
- Xilofagi attacks: With the presence of anobiids and stemborer as confirmed by the size of the holes in the timber
- Mold Decay: Represent the staining fungi deep a later development in some cases of discoloration that penetrate into the lumen through punctuation aerolatee that stain intensely invaded tissues

In particular it was noted that these diseases (fungi on the surface) are present in the lower sections of the struts and the main longitudinal beams (deep staining fungi) in the upper elements that are arranged in an improper manner and unprotected due to mechanical damage caused by man (repeated removal flashings copper theft vandalism). The instrumental analysis such as histograms show the overall summary of the results for the individual types of testing. The hardness values for wood density determined by the Pylodin piling are, however, acceptable and confirm a reasonable state of preservation except in a few localized spots where there are attacks of Biological level of caries. Moisture measures in the site are contained in most below the 20% threshold trigger of fungal attack with the exception of some struts in the part of contact with the shoulders of the bridge in which are also visible sagging due to oxidation of the plates.

Such situation is not present in the elements of metal nuts and bolts of the links as well as the metal implants. The investigations by penetrometer as shown by the diagrams for different categories of items, respectively, longitudinal beams, struts and handrail, except for some points of the latter have a good state of preservation of the sections examined must be considered that have been carried out in different states both longitudinally and transversely and they are some. From the point of view of Durability refers to the UNI 350 UNI 460 and both the 1996 and EN 335 of 2006, known the wooden species, which is fir (not particularly resistant to several xilofagi including those detected), and the Class of Use, which is 4 if not 5 (as confirmed by the values of moisture crossing the data in the table in EN 460) can be deduced that the treatment preserved is necessary because the species is not durable without it. Other additional instrumental measures are provided to check the acidity and the aggressiveness of the biological degradation.





9 Acknowledgement

The following Technical collaborated in the activities:

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We thank the Municipality of Genoa in the person of Arch. Tedeschi for having granted the experiments on the work in question and Mr. Tuvico of voluntary associations Praesi of Genoa for his collaboration.

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Galata Timber Bridges

Halet Almila Arda Buyuktaskin[1]

Content

Galata Bridges constructed on the Golden Horn were always symbolic links between the old Istanbul at Eminonu neighborhood, the site of the imperial palace and principal religious and secular institutions of the Ottoman Empire, and the districts of Galata, Beyoglu, Sisli and Harbiye where a large proportion of the inhabitants were non-Muslims and where foreign merchants and diplomats lived and worked. In this respect Galata Bridges connected these two distinctive cultures.

It is known that the first bridge, over the Golden Horn, named “Cisr-i Atik or Old Bridge” was constructed in 1836 between Unkapani and Azapkapi because the imperial shipyards were at Azapkapi. Made of wood, this pontoon structure had two arches that provided the necessary height for the passage of small boats. It was about 600 meters in length and 10 meters in width. The first modern Galata Bridge was constructed in 1845, at the time of Sultan Abdulmecid, by the directive of her mother Bezm-i Alem Valide Sultan. She was having particular interest in various building projects in the Dolmabahçe-Yildiz Palace area. She wielded significant power and held that position until her death in 1853. The bridge carried the name “Cisr-i Cedid or New Bridge” to distinguish it from the one at Unkapani. It had also some other names as: “Mother Sultan Bridge”, “Big Bridge”, “ New Mosque Bridge” and “Bridge with Pigeons”. Sultan Abdulmecid was the first person to pass over the bridge and the first to pass below was the French captain Magnan in his ship the Cygne. For the first three days crossing the bridge was free, after which a toll known as “mürüriye” was paid to the Naval Ministry to defray the costs of building it.

The Galata Bridge was made of timber and measured some 500 meters. It was replaced in 1863 with another timber bridge that was stronger than the first one, built by Ethem Pertev Pasha on the orders of Sultan Abdulaziz in readiness for the visit to Istanbul of Napoleon III. Prior to these bridges, the only way to cross between Karaköy and Eminönü had been by rowboats. Apart from their places in fiction, the romantic appearance of the Galata Bridges made them subjects of many paintings and engravings (Figure1).

This paper deals with Galata timber bridges constructed on Golden Horn, emphasising their characteristic features and historical distinctions.

Figure 1: Vittorio Amadeo Preziosi (Maltese, 1816-1882) The Galata Bridge, Istanbul

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TECHNICAL TOUR
Timber Bridges in Switzerland

Technical Tour - Timber bridges in Switzerland

Friday 26th September 2014

Summary

The tour will be a timber bridges tour with the aim to show modern and old Swiss timber bridges. A participation of local experts (engineers/technicians/carpenters) that designed/built the bridges as well of those dealing with these bridges (monitoring and maintenance) is envisaged. The coffee breaks will give the possibility for discussions and also to taste Swiss traditions while visiting an old cheese factory in Affoltern in Emmental and a cookie factory “Kambly” in Trubschachen.

Key words: Aarebrücke, Neumattbrücke, Gohlhausbrügg, Obermattbrücke, Bubeneibrücke

1 Aarebrücke

The Aarebrücke is located in Büren an der Aare and features following specifications:

- Built: 1821
- Covered bridge
- Truss frame
- Max load: 16 tons
- Length:107 m
- Width: 4.8 m
- Height: 3.7 m
- Asphalted roadway

Source: swiss-timber-bridges.ch

2 Neumattbrücke

The Neumattbrücke is located in Burgdorf and features following specifications:

- Built: 2013
- Covered bridge
- Framework
- Span: 59 m
- Width: 3.8 m
- Height: 3.6 m
- Oak decking roadway

Source: n’Holzbau Lungern

3 Gohlhausbrügg

The Gohlhausbrügg is located in Lützelflüh and features following specifications:

- Built: 1584
- Covered bridge
- Max load: 28 tons
- Length: 61.7 m
- Width: 5.6 m
- Height: 5.5 m

Source: swiss-timber-bridges.ch

Photo: S. Franke

4 Obermattbrücke

The Obermattbrücke is located in Obermatt and features following specifications:

- Built: 2007
- Open bridge
- Max load: 40 tons
- Length: 32 m
- Asphalted roadway
- Lifting bridge at high tide

Source: swiss-timber-bridges.ch

Photo: S. Franke

5 Bubeneibrücke

The Bubeneibrücke is located in Signau and features following specifications:

- Built: 1837
- Covered bridge
- Arched girder
- Max load: 28 tons
- Length: 50 m
- Width: 7 m
- Height: 4.5m
- Asphalted roadway

Source: swiss-timber-bridges.ch

Photo: S. Franke

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The main advantages of the new SMART-BITE wood drilling tip of WT-T fastening system at a glance:

- Very good setting behaviour at 45°.

- Fast and effortless setting at 90°.

- Very low setting torque.

- Prevents splitting of the wood.

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