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COST ACTION FP1101

Workshop

HIGHLY PERFORMING TIMBER STRUCTURES: RELIABILITY, ASSESSMENT, MONITORING AND STRENGTHENING

September 23-24, 2014

Bern University of Applied Sciences,
Institute for Timber Construction, Structures and Architecture
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PREFACE

Assessment, strengthening and monitoring are essential activities to extend the life cycle of historic and existing large span timber structures. These encompass the roof structures of important architectural heritage buildings, such as the hammer-beam roof of Westminster hall, UK, to the timber bridges such as the Chapel Bridge in Lucern, Switzerland, to entire timber framed buildings such as the six storey Tamedia building by Shigeru Ban, Switzerland, but also modern laminated timber constructions constituting the envelop of event halls, swimming pools, timber bridges and multi-storey framed buildings.

The large spans and their environmental exposures pose high demands on their performance to meet safety and comfort standards. Advanced tools for assessment and monitoring are essentials to correctly determine the response of these structures in use and optimise any strengthening intervention required to ensure they meet conservation, structural and economic criteria concurrently. In other words they ensure enhanced sustainability of these magnificent structures and delivery to future generations.

But the monitoring and assessment of existing structures is also a unique resource to learn more about their performance and to positively influence future design developments, focussing on best use of current technologies and identifying gaps and current shortcomings needing development of new design solutions.

The COST Action FP1101 "Assessment, Monitoring and Reinforcement of Timber Structures" pulls together these three strands of engineering and collects in a unique network researchers, engineers and practitioners with a wealth of experiences in these field. The COST programme provides the framework for sharing these expertise across all countries in Europe and further afield, in a spirit of peaceful and progressing cooperation.

Within the activities of the COST Action FP1101 the workshop "Highly Performing Timber Structures: Reliability, Assessment, Monitoring and Strengthening" organised by the Bern University of Applied Sciences, Institute for Timber Construction, Structures and Architecture, Biel, Switzerland provides a platform for discussion and interaction by researchers and practitioners from different generations and countries to learn from each other work. The workshop focusses on current timber constructions produced of glued laminated timber. What are here the key issues that affect the long term performance of these structures? What are the performance parameters that need to be monitored over time to determine such performance? How can we use non destructive methods to assess such structures behaviour? What are possible and efficient reinforcement methods? Theoretical and practical experiences, methods and theories will be shared by researchers, engineers and industry partners.

This volume of proceedings contains the papers submitted by the participants to this workshop, forming the basis of reference for the discussions which will enliven the two-days of presentations and debate sessions, hosted in the nice setting at the Biel campus of Bern University of Applied Sciences.

We thank all the participants and COST Action members from research institutes and industry for the great contribution and support. Without their effort, the event would not happen. We are confident to have a successful and fruitful workshop and welcome everyone in Biel.

Editors

Bettina Franke – Organizer and Host

Steffen Franke – Organizer and Host

Andreas Müller – Organizer and Host

Dina D'Ayala - Chair COST Action FP1101

September 2014

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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Maintenance and modification of existing structures

René Steiger [1]

Summary

Designers regularly are involved in the assessment and retrofitting of existing structures. This task, compared to the design of new structures, is completely different and, in order to find suitable and economic solutions should follow a specific format. Decisions on maintenance interventions have to be taken based on all the available information on the present condition of a structure, on its past "experiences" in terms of foreseen and accidental loading and on the intended future use and service life. The assessment of an existing structure starts with a preliminary investigation on site, followed by a general and possibly one or several detailed investigation(s). Eventually urgent or supplementary safety measures have to be taken. Then the condition of the structure is surveyed and evaluated and a prognosis about the further development is given. This step includes the verification of the safety and the serviceability of the structure. If the decision is to modify the structure, different variants of maintenance interventions are planned and compared and their proportionality and effectiveness are verified.

Key words: Significance of an existing structure, Assessment, Maintenance interventions, Updating, design

1 Introduction

Structures are designed on base of design codes and standards reflecting the state-of-the-art at that time with the intent that they shall meet the requirements specified in the performance criteria agreement in terms of load-bearing capacity and serviceability throughout the whole service life (SL). As a matter of time however, due to e.g. changes in use, changes of climatic conditions (indoor and/or outdoor) or changes in material properties, an assessment of existing structures including a condition survey and a verification of the actual load-bearing capacity and serviceability may become necessary.

Maintenance interventions should be economic, ecologic and sustainable, accepted by the public and at the same time fulfil the new requirements and needs in terms of intended future use and expectations in safety of individuals and the whole society. In particular the requirements from the legal and from the user's point of view are to be met together with the structural safety and the serviceability. The significance of an existing structure regarding its monetary but also its intangible values have to be accounted for. Long-term preservation of the value of a building/structure can be the aim of maintaining structures. Furthermore, legal requirements might force the owner of an existing building to invest in maintenance interventions.

Several qualities may contribute to the significance of an existing structure. These consist of tangible but also intangible criteria (e.g. [1]). Examples of intangible criteria are the actual situation of the building with respect to its surroundings, the historic/cultural value, aesthetic qualities, artisanal-technical quality, socio-cultural and emotional quality. The monetary value of a structure/building mainly results from its site (location), type of use, actual state, significance of the building for the society and as part of the surrounding infrastructure, value from the economic point of view, impact on surrounding area and neighbour buildings.

The process of assessing existing structures due to an intended change in use or a marked prolongation of the SL to a considerable extent differs from designing new structures. The design of new structures is based on assumptions with regard to actions, geometrical and mechanical properties. Planning of maintenance interventions, on the other hand, beside assumptions with regard to the change in use can benefit from a lot of available information related to the existing structure. According data and information in any case has to be assessed and updated in order to be able to propose to the owner of the building a well-suited and economic strategy for the further use of the building and for possible maintenance interventions.

2 Motivation for maintenance interventions

The assessment of an existing structure aims at proofing that the requirements associated with the structure are fulfilled over a specified residual service life (RSL) [2]. The need for an assessment of an existing structure origins either from a change of the requirements in the use of the building and/or the requirements related to the

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structure itself and/or from doubts in regard to whether the assumptions underlying the design of the structure are fulfilled or not. Typical situations where the use/purpose of a structure is changed are [3]:

- Increased level of loading (e.g. higher live loads, revised models for snow loads),
- Increased intended service life time (The structure is still needed after the planned service life.),
- Request of increased reliability (e.g. due to a change in importance of the structure for the society),
- Modification/strengthening of the structure to accommodate changes in use.

Situations where doubts may be raised in regard to the integrity of the structure are e.g. [3]:

- The structure has not been inspected for an extended period of time (Damages and unforeseen degradation might have taken place.).
- Adverse results of a periodic investigation of the structure's state took place (Unexpected degradation e.g. due to rot, decay, increased moisture, corrosion, etc. has been observed.)
- Deviations from the original project layout are observed. An inadequate serviceability is noticed. Construction or design errors are becoming aware.
- The structure has been subjected to foreseen accidental or non-foreseen extreme loads (excessive load, earthquake, impact of vehicles, fire, etc.)
- Similar structure(s) exhibit unsatisfactory performance.
- New knowledge on the structural behaviour and/or revised design codes are available.

3 Main elements in the assessment and maintenance of existing structures

The main elements in the assessment and maintenance planning are shown in Figure 1 [4, 5]. Before starting the assessment, it is essential to take down the aspects of the further use of the structure (purpose) as well as the intended RSL (which might be associated with a certain inspection and maintenance program) together with the owner of the building. The owner's and engineer's agreement on the expected performance criteria of the building has to address appropriate safety measures to defined hazard scenarios as well as a list of accepted risks.

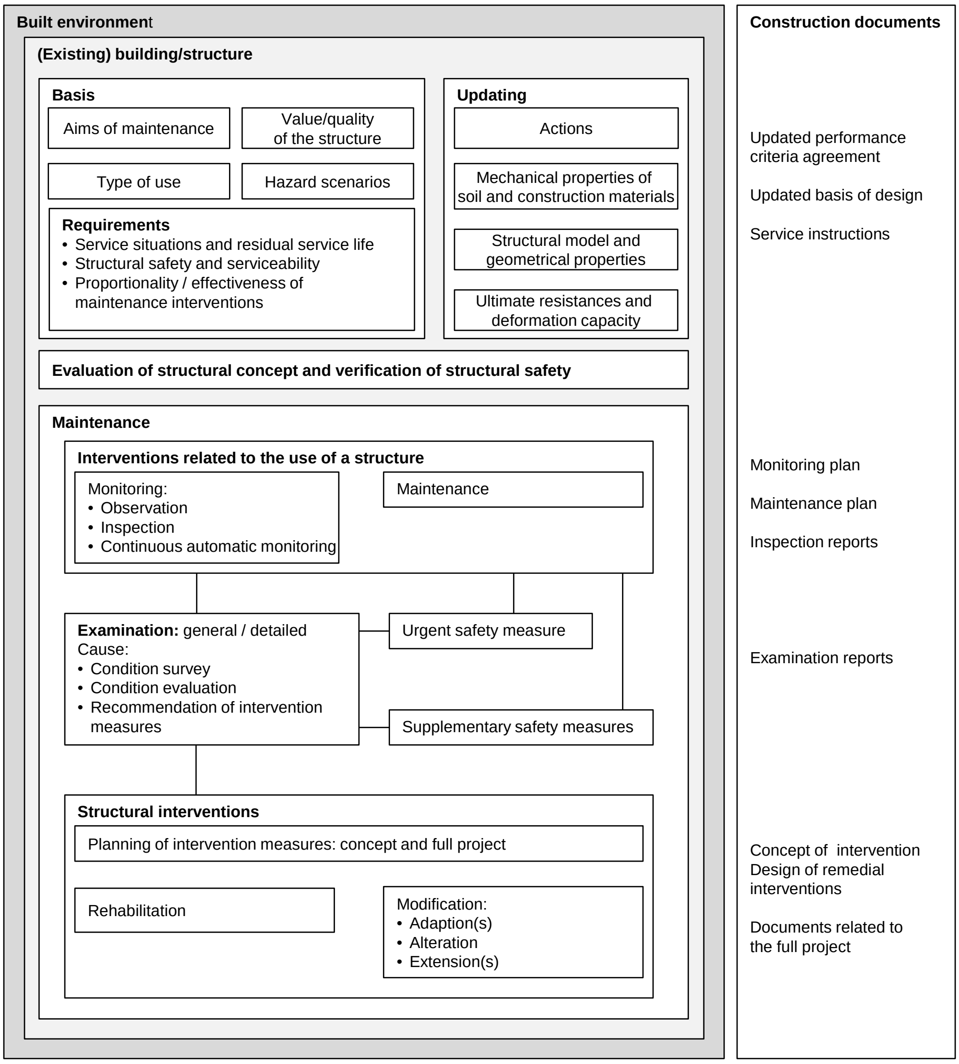

Figure 1: Activities and key documents in the assessment and maintenance of existing structures (adapted from [4, 5]).

If the requirements regarding the present and future use of a structure are specified, the assessment is a decision process with the purpose of identifying those measures which lead to the most economical fulfilment of these requirements with regard to the RSL. Hereby a subordinate decision has to be taken in regard to the level of detail of the analysis to be performed. It is essential, that the committed engineer can make his decisions based on codified standards which reflect the present best practices in regard to the assessment of existing structures.

Guidelines for the use and the maintenance of existing structures exist in many countries. At least in the USA, Canada, Switzerland, UK such guidelines have been prepared at a detailed level [2]. So far however, only a few countries (for example the Netherlands [6] and Switzerland [1, 3, 7-13]) have worked out generally applicable code-type documents for the assessment of existing structures. In 2001 the first edition of an ISO-standard [14] on the assessment of existing structures has been approved. Eurocodes for the assessment of existing structures are not available yet but the responsible technical committee CEN/TC 250 recently set up a special Working Group dealing with this topic (http://eurocodes.jrc.ec.europa.eu).

4 Basic procedure in surveying the condition of a structure

It is suggested to perform the assessment in different steps with increasing deepening, the degree of deepening thereby depending on the amount and the quality of information being at disposal as well as on the importance of the building. This can be reached by breaking down the assessment into different phases (Figure 2) [2, 3, 15]. The assessment starts with a preliminary examination, including a site visit with simple, mainly visual checks. Special attention in this step is paid to the overall state of the structure and to its critical parts. Typical hazard scenarios, which could endanger the situation, have to be identified and as a result of this, immediate measures eventually are needed.

If doubts are confirmed, the assessment is continued with a general examination, in the course of which the whole structure as well as single structural elements (including non-load-bearing parts, if these parts are relevant with regard to the safety of individuals, valuables and environment) are examined. The general examination has to be based on a study of the existing documents and on an agreement on performance criteria or on similar documents like utilisation and safety plans. Then the concept of the existing structure has to be evaluated, paying special attention to the robustness of the system (serial and parallel systems, static redundancy, etc.) [16]. Visual

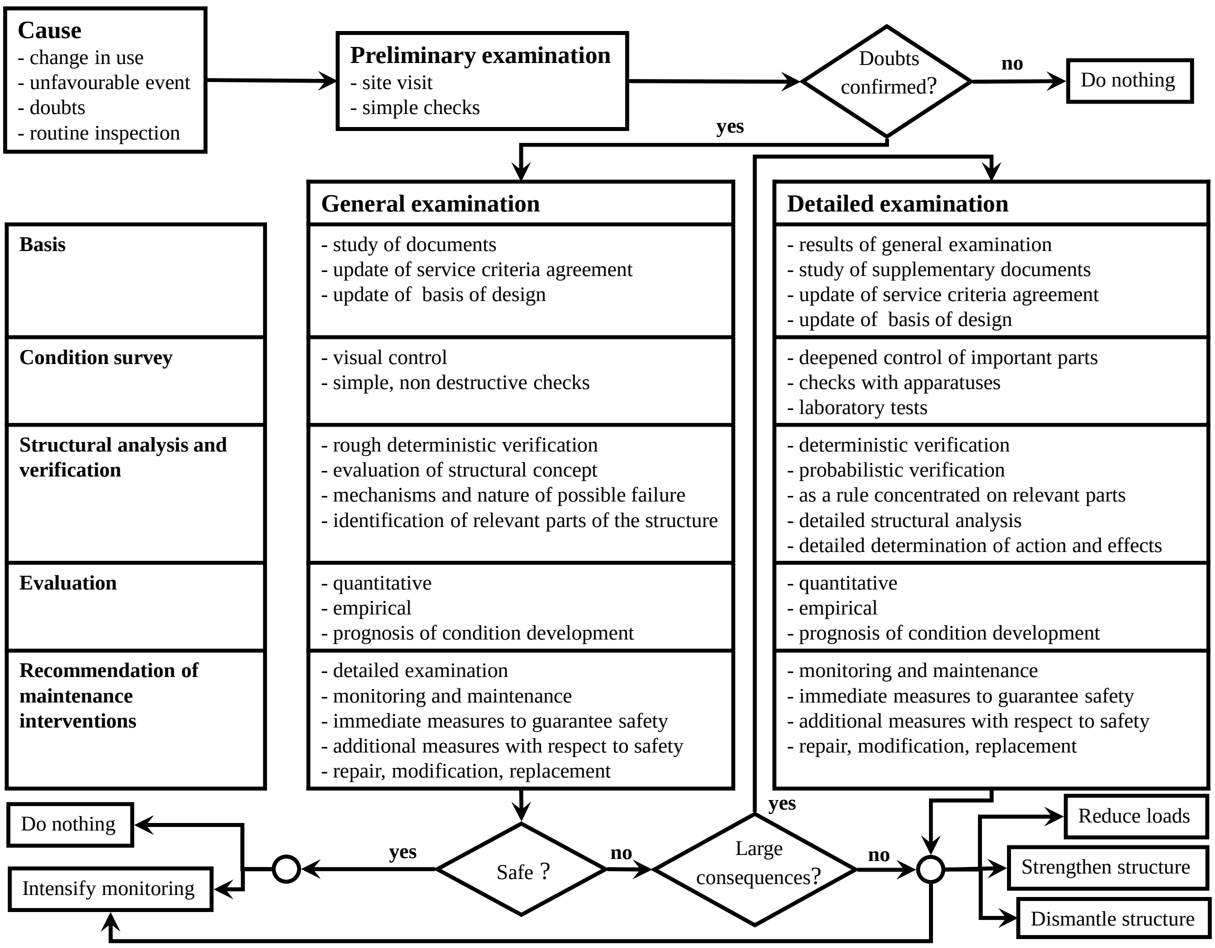

Figure 2: Stepwise procedure when assessing existing structures [2, 3, 15].

and simple, mostly non-destructive examination on site serve at identifying deterioration and deficiencies. Finally a rough limit states (LS) analysis (serviceability and ultimate limit state, SLS, ULS) is performed, in order to identify zones and members of the structure which are of decisive influence on the structure's integrity.

If the general examination did not confirm the building's adequate state with regard to its intended further use, a single detailed examination or perhaps several of them are needed. Key zones and members of the structure are examined in detail (again with increasing deepening) using more sophisticated methods (expert's opinion, proof loading, destructive and non-destructive tests of representative samples, etc. [15, 17]). Causes of deterioration and deficiencies of the key zones and members have to be found. The load-bearing behaviour of the structure is analysed in more detail and in the course of the LS analysis the actions and the resistances are determined with higher accuracy, using more sophisticated models. Regarding the resistance, possible failure modes (brittle failure, ductile failure with/without strength capacity from strain hardening) of the load-bearing members and connection have to be evaluated, taking into account the deformation development (past) as well as the deformation behaviour/capacity (future). The current mechanical properties of the construction materials have to be assessed and prognosis about the future development of these properties have to be given.

If the gathered information about the load-bearing behaviour and capacity of a particular structure is still insufficient or especially in cases where a structure shall be subjected to higher live loads, static or dynamic proof loading may be an appropriate tool to assess the performance of the structure.

Each condition survey consists of an evaluation of the reliability and plausibility of the results of the examination!

5 Condition evaluation and prognosis of the condition development

After having assessed a structure or parts of it, the condition has to be evaluated in terms of structural safety, serviceability and durability. The first step in evaluating the condition of a structure is a condition survey, which usually bases on a quantitative assessment, where the structural safety and the serviceability have been numerically verified [3]. Only in special cases it may be sufficient to evaluate the state of a structure empirically. Important parts of the structural system which cannot be assessed directly have to be evaluated on base of indirect indications.

It is recommended to study the impact of decisive examination values and model uncertainties by means of a sensitivity analysis. In case where it is not possible to meet the requirements in load-bearing capacity or serviceability, the key parameters shall be checked again. This is of big importance especially for structures which might collapse due to sudden brittle failure of structural members. Eventually an additional detailed examination can provide more information or revised data.

Not only is the actual state of interest but also the possible future development. The prediction of the future development of the condition of a structure encloses a statement on the expected future condition of the structure itself and of its structural safety and serviceability. The prognosis has to account for possible changes in actions, climatic conditions, ultimate resistance and load-bearing behaviour including unfavourable impacts of damages and deterioration. Based on the condition survey and on the experiences made with similar structures, qualitative statements concerning the condition development can be made. Finally the prognosis of the condition development has to be benchmarked to the residual service life.

6 Recommendation of intervention measures

The owner of the investigated building will take the final decision on how to proceed on base of the engineer’s recommendation of intervention measures. The latter may consist of [3]:

- Accepting the actual state of the structure,
- Immediate intervention, possibly accompanied by urgent safety measures,
- Supplementary safety measures,
- General and/or detailed assessment of the structure,
- Changes in monitoring or maintenance intervals of the structure,
- Rehabilitation or modification of the structure or parts of it,
- Replacement of the structure or parts of it,
- Decommissioning of the building,
- Dismantlement of the building.

7 Updating

When assessing existing structures and planning maintenance interventions information about the structure is a key issue. The main point being different compared to the design of new structures is that the amount and the quality of available information are different: For new structures we have more or less general and in some sense imprecise information about the critical characteristics of the structure to be designed, e.g. intended geometrical properties, mechanical properties of soil and building materials as indicated in design codes and actions on the structure to be accounted for in the design. The latter depend on the building site (location), size and shape of the building and on its use.

For existing structures there is much more information at disposal and it is important to use this information when assessing existing structures. The respective process is called updating and involves the actual structural system, geometrical properties, mechanical properties of building materials and soil, the load-bearing and deformation capacity of the structure, the local climate (indoor and outdoor!) as well as possible damages and deterioration. The information to be collected can be of very diverse nature e.g. in type of measurement data, subjective information, etc. Some information is purely qualitative (no, minor, severe damage) whereas other is of quantitative type (crack lengths and depths, displacements, etc.). Updating can also make use of indirect information, which does not origin from the structure itself, but which is correlated to the structural performance, e.g. common loading, correlated material properties or correlated degradation processes. Qualitative information ("The structure looks fine.") has to be formalized in order to be able to use it in calculations.

The Swiss assessment code SIA 269 [3] uses a semi-probabilistic approach to derive updated design values from known (and updated) probability distributions of basic random variables (BRV) (effects of actions, ultimate resistance and stiffness properties). More detailed information related to this topic together with a numeric example can be found in [18].

8 Analysis of the structural concept and verification of structural safety

Both the analysis of the structural concept and the verification of structural safety can start from the assumption that the structure has been monitored and maintained since there is no safety margin foreseen in the design codes which would cover missing or insufficient monitoring and maintenance as well as overlooking/ignoring of damages, deterioration and deficiencies.

Compared to the design of new structures, when assessing existing structures the engineer's thinking should be more analytical in terms of for example a differentiation between processes/actions driven by load ($F = m \cdot g$), by force ($F = m \cdot a$) or by deformation (settlement, deviation in temperature, etc.).

8.1 Analysis of the structural concept

In the course of analysing the structural system the structure is assessed with regard to relevant design situations together with respective key parameters. The strategy is not only to examine single structural elements but the whole structure including the (past) deformation behaviour and the perspectives in (future) deformation behaviour/capacity as well as possible failure modes (brittle, ductile, progressive failure, etc.). Hereby one has to take into account that there is a close relation between deformations and actions structures are subjected to! Impacts of damages and deterioration on the load-bearing and deformation capacity have to be identified, quantified and accounted for when updating geometrical and mechanical properties. In addition the robustness of the structure shall be evaluated [16].

8.2 Verification of structural safety

The verification of the safety of existing structures usually is carried out similar to the design of new structures, i.e. deterministically. Hence, also existing structures have to meet the requirements in terms of level of safety specified for the design of new structures. The quite frequent opinion that the requirements in level of safety for existing structures were laxer is completely wrong [2, 4]. The deterministic verification of existing structures in partial safety factor format however, differs from designing a new structure in terms of available information. As already mentioned, all BRV have to be updated accordingly. Compared to the designing of new structures, where design situations are examined using design values of action effects and material resistance, in the assessment of existing structures examination situations described by examination values of action effects and resistances are evaluated. In order to be able to better get aware of immanent deficiencies and reserves, the format of verification is changed

from $E_{d,act} \leq R_{d,act}$ to $DC = \dfrac{R_{d,act}}{E_{d,act}} \geq 1$

where DC stands for degree of compliance and E_d and R_d are the design values of the verified hazard scenario and ultimate resistance respectively. Properties, values and terms which had been updated based on new information should be clearly distinguished from those used for the design of new structures, i.e. they are "actual" and should be marked by an index "act".

If DC < 1 the structure has to be strengthened or the loads have to be reduced. In cases where the proof of an adequate resistance did just not pass, a semi-probabilistic or a probabilistic verification might be useful before arranging tedious and costly strengthening measures [19, 20].

The bigger the DC is, the higher are the reserves in terms of load-bearing capacity. The numerical value of the DC provides valuable information about the potential of the structure to face future changes in use [4]. For $0.4 \leq DC \leq 0.6$ the structure will either suffer from marked overloading or the structural verification has been carried out on base of too conservative mechanical properties. If $DC < 0.5$, urgent safety measures have to be taken.

9 Maintenance interventions

It has to be differentiated between structural interventions and interventions related to the use of the structure. The latter enclose monitoring and maintenance according to the respective plans drafted when designing the structure. Structural interventions consist of rehabilitation and modification (adaption, alteration, extensions). Maintenance interventions resulting from the assessment or the monitoring of a structure may also enclose urgent and supplementary safety measures.

When planning maintenance interventions, similar to the assessment of a structure, a stepwise procedure appears to be appropriate. The examination should provide recommendations of interventions. If structural interventions were recommended, the initial task is to suggest a concept of where and how to intervene. In terms of level and contents this concept corresponds to a pre-project and consists of a description of possible variants of maintenance intervention including a verification of their feasibility, a comparison of the different variants and a well-founded proposal of the most appropriate (best) variant. Together with the owner of the building the variant finally to be followed is chosen and the engineer in charge starts more detailed planning activities resulting in full project documents.

10 Verification of proportionality/effectiveness of maintenance interventions

When deciding about starting maintenance interventions, it is a key issue to have an idea about the proportionality and effectiveness of such measures. In the Swiss Code SIA 269 [3] the proportionality of maintenance interventions related to safety is assessed on base of their effectiveness and taking into account the following aspects:

- Safety requested by individuals and by public
- Availability of a building or a facility
- Consequences of failure for human beings, valuables and environment
- Preservation of monetary and cultural values.

The expenses for maintenance interventions can be expressed in terms of costs for granting the requests of structural safety and serviceability of a structure. As a benefit of maintenance interventions, the increase in monetary and cultural values, the reduction in costs for inspections/monitoring and maintenance as well as the reduction in risk because of restoring the required structural safety and serviceability can be revealed. Finally the proportionality of maintenance interventions can be assessed by comparing the costs of safety measures with the efficiency of interventions. The according procedure is codified in [3] and described in [18].

11 Conclusion and future work

For designers the assessment of existing structures and planning of maintenance intervention has become daily business and will even gain importance in future. Hence, commonly accepted guidelines and codes (e.g. in the form of additional sets of Eurocodes) should be made available as soon as possible. The series of Swiss codes SIA 269 “Assessment of existing structures”, available since 2011 provide a valuable basis for drafting according codes with a broader acceptance (e.g. Eurocodes).

References

[1] Schweizerischer Ingenieur- und Architektenverein (2000) SIA-Merkblatt 2017: Erhaltungswert von Bauwerken. SIA, Zürich, Schweiz.

[2] Diamantidis D. (2001) Probabilistic assessment of existing structures. RILEM Publications S.A.R.L., Cachan, France.

[3] Schweizerischer Ingenieur- und Architektenverein (2011) SIA-Norm 269: Grundlagen der Erhaltung von Tragwerken. SIA, Zürich, Schweiz.

[4] Brühwiler E. (2011) Verständigung, Grundsätze und Grundlagen beim Umgang mit bestehenden Tragwerken. In: Dokumentation D 0240: Erhaltung von Tragwerken - Vertiefung und Anwendung, Schweizerischer Ingenieur- und Architektenverein, SIA, Zürich, 9-31.

[5] Swiss Society of Engineers and Architects (2013) SIA standard 260: Basis of structural design. SIA, Zurich, Switzerland.

[6] Nederlands Normalisatie-instituut N. (2011) NEN 8700: Grondslagen van de beoordeling van de constructieve veiligheid van een bestaand bouwwerk - Gebouwen (Assessment of existing structures in case of reconstruction and disapproval - Basic Rules). NEN, Delft, The Netherlands.

[7] Schweizerischer Ingenieur- und Architektenverein (1994) SIA-Richtlinie 462: Beurteilung der Tragsicherheit bestehender Bauwerke. SIA, Zürich, Schweiz.

[8] Schweizerischer Ingenieur- und Architektenverein (1997) SIA-Empfehlung 162/5: Erhaltung von Betontragwerken. SIA, Zürich, Schweiz.

[9] Schweizerischer Ingenieur- und Architektenverein (1997) SIA-Norm 469: Erhaltung von Bauwerken. SIA, Zürich, Schweiz.

[10] Schweizerischer Ingenieur- und Architektenverein (2004) SIA-Merkblatt 2018: Überprüfung bestehender Gebäude bezüglich Erdbeben. SIA, Zürich, Schweiz.

[11] Schweizerischer Ingenieur- und Architektenverein (2011) SIA-Norm 269/1: Erhaltung von Tragwerken - Einwirkungen. SIA, Zürich, Schweiz.

[12] Schweizerischer Ingenieur- und Architektenverein (2011) SIA-Norm 269/5: Erhaltung von Tragwerken - Holzbau. SIA, Zürich, Schweiz.

[13] Swiss Society of Engineers and Architects (2003) SIA standard 260: Basis of structural design. SIA, Zurich, Switzerland.

[14] International Organization for Standardization ISO (2001) ISO 13822: Bases for design of structures - Assessment of existing structures. ISO, Geneva, Switzerland.

[15] Brüninghoff H., Kreuzinger H., Srpcic J., Steiger R., Köhler J., Tannert T., Dietsch P., Hösl M., Fink G. (2010) Assessment of timber structures. Shaker Verlag GmbH, Aachen, Germany.

[16] Sorensen J. D., Dietsch P., Kirkegaard P. H., Munch-Andersen J., Cizmar D., Neves L., Branco J., Zhang B., Fink G., Steiger R., Köhler J., Rajcic V., Turk G., Winter S. (2010) Design for robustness of timber structures. Shaker Verlag GmbH, Aachen, Germany.

[17] Kasal B. (2010) In situ assessment of structural timber state of the art report of the RILEM Technical Committee 215-AST. Springer, Dordrecht, The Netherlands.

[18] Steiger R., Köhler J. (2008) Paper CIB-W18/41-102-2: Development of new Swiss standards for the assessment of existing load bearing structures. In: Proceedings of the CIB-W18 Meeting Forty One, St. Andrews, Canada.

[19] Köhler J. (2011) Die Aktualisierung als zentrales Element in den Erhaltungsnormen - Aspekte der Probabilistik. In: Dokumentation D 0240: Erhaltung von Tragwerken - Vertiefung und Anwendung, Schweizerischer Ingenieur- und Architektenverein, SIA, Zürich, 33-36.

[20] Köhler J., Sørensen J. D., Faber M. H. (2007) Probabilistic modeling of timber structures. Structural Safety, 29(4):255-267.

Assessment - Introduction, Procedure and Methodology

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Summary

The examination of the structural condition is one of the most important parts of the assessment of an existing building. Examinations have to be done with great care, using the appropriate testing methods and tools. The analysis of a building's condition determines its cohesion in terms of construction and evaluates the safety and serviceability of the structures. It also detects damage to structural elements and lays the groundwork for reinforcement, renovation or monitoring measures. An advantage of timber structures is that discolouration or cracks make it relatively easy to visually recognise problem areas; these can be defined early when visual inspection is combined with measuring the humidity of the wood. Additional methods are available for analysing a building's condition more comprehensively and making more detailed measurements. The testing methods used in assessing a building's condition are divided into three categories: non-destructive, semi-destructive and destructive. The different testing methods are well described in current literature e.g. [1], [3], [6].

Key words: Examination, Visual (hands-on) inspection, Structural condition, Testing methods, Documentation

1 Introduction

In Europe and elsewhere in the world, timber is being used for bigger and more ambitious structures. The engineer has to bear greater responsibilities in the design, the supervision of construction and the monitoring of the performance of the structure while it is in use.

Assessment is becoming increasingly important to safety issues. Part of examining an existing building consists of assessing its condition, as well as making a thorough evaluation of the load-safety factor or - if necessary - the remaining load-bearing capacity of its structure. Regular assessments prevent a building's users from being endangered.

When a building's condition is analysed, an evaluation is made of its cohesiveness in construction terms, its load-bearing capacity and serviceability are assessed, and any damage to structural elements is detected. The groundwork for reinforcement, renovation or monitoring measures can thus be laid. Experience has shown that defects and damage to the load-bearing structure that are recognised early, can be corrected at comparatively little cost. The methods used in assessing a building's condition allow for precise appraisal of the measures, needed to plan reinforcement or change of use. It also forms an important basis for calculating the cost of both. In Switzerland, the assessment of a building's condition is based on the utilisation agreement and the project basis, made by the civil engineers in the planning process. Additionally, any existing inspection and assessment reports have to be taken into account [2].

2 Procedure and Methodology

Assessments must be carried out with great care, using the appropriate testing methods and tools. They must always be conducted regularly at set intervals, using various specialist competencies and testing techniques. All statements and results must be verifiable and, furthermore, be so comprehensibly documented and substantiated that anyone can understand them in, for example, a court case. The most important fundamental rule is that each structural element must be closely checked. Random spot-checks of the load-bearing structure are not adequate. A competent and substantiated report on the condition of the structure as a whole can be made only if it has been extensively examined; this provides both the owners and users of the building with the necessary security. In the event of damage occurring later, a court of law would classify a remote diagnosis as gross negligence. The engineer or expert who had conducted the examination would be held fully liable.

A wide range of non-destructive, semi-destructive and, if needed, destructive investigative methods is available for assessing the condition of timber constructions. There are acoustic, electro-magnetic, thermal and optical methods, as well as mechanical techniques. Some are destructive. Table 1 provides an overview of typical pro-

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cedures for assessing the condition of timber constructions. Non-destructive and semi-destructive testing procedures are mainly used. Each procedure has specific advantages and disadvantages, making it either more or less suitable for use in particular situations. It is therefore important to use the most suitable procedure for each inspection. Combining different procedures often provides better results with greater information.

Table 1: Overview of typical assessment methods for timber constructions

Method	**Process**	**Condition**
Visual (hands-on) inspection	photographs, magnifying glass, microscope knocking and tapping moisture content measurement (resistance method) mapping of cracks core drill simple chemical tests	damage to cross-section wood type and quality fungal and insect infestation cavities, cracks surface treatment type of adhesive exposure to chemicals, deformations
Current testing procedures	taking core samples drill resistance measurement penetration resistance measurement testing quality of glue lines mechanical determination of resistance	damage to cross-section wood density rigidity of wood quality of glued joints mechanical characteristics
Special testing techniques	endoscopy ultrasound x-rays	concealed components cavities connections fungal and insect infestation
Load tests	in-situ measurements	deformation, rigidity of entire structure

The testing procedures are tools with which a building can be assessed, allowing the condition of its materials, elements and entire load-bearing structure to be measured quantitatively. The property being tested is generally not damaged when it is measured, and its intended purpose is also not influenced. Evaluating and interpreting the results of the measurements requires highly developed professional knowledge and many years of experience. Obtaining additional, more detailed, information about the condition of an element often requires the use of destructive investigations or taking samples, with subsequent laboratory testing. It is therefore advisable to first consider the consequences of the proposed action in relation to its benefits.

What damages a timber structure?

With buildings and structures made of timber or wood materials, priority should go to checking for moisture and possible cracking. Changing climatic exposure causes moisture levels in wood to fluctuate, making structural elements shrink and swell, which in turn can cause cracks to appear. Furthermore, the mechanical characteristics of timber change with fluctuations in the moisture content, and a marked increase in moisture leads to damage by fungi that destroy wood. Detecting cracks can – depending on their extent and causes – have a decisive effect on the assessment of a building's load-bearing capacity (see Fig. 1, Fig. 2 and Fig. 3). Any delamination of glued structural elements needs to be differentiated from cracks with other causes (such as shrinkage cracks).

Figure 1: Shrinkage cracks in glulam

Figure 2: Delamination in the glued joint

Figure 3: Shear cracks

Figure 4: Fungi attack due to a high moisture content

Figure 5: Degradation depends on high moisture content

Damage caused by wood-destroying insects has become something of a rarity in central Europe. Drying wood technically modifies its ingredients. Recent studies [4] show that the danger of infestation in such wood by wood-destroying insects can be ruled out. This includes all solid wood products used in construction. Nonetheless, any visual inspection of components of this type should also include a check for exit holes or signs of active infestation (sawdust).

The premises to be assessed should be visited beforehand in the company of the person responsible for the building. All available data relating to the building, such as plans, building specifications and utilisation agreements, should be made available for the assessment. Additional information about the ways the building has been used since its construction is often helpful. This can, for example, make it easier to evaluate the demands the climate and other effects have had on the building over the years.

Familiarity with the load-bearing characteristics of the structure as a whole is of great importance for assessing the condition of a building and for understanding any deficiencies in its structure. Problems or deficiencies in the overall stability of a building are often recognised only as a result of this entire essential knowledge. Furthermore, important stabilisation and bracing elements for structural members that are at risk of twisting or buckling have often been severed during later installation work or, because of ignorance were not even installed in the first place (Fig. 6). A responsible expert must be able to recognise these failings and call for the corresponding measures to be taken. Complete access needs to be ensured, so that load-bearing elements can be inspected individually. Accident and labour regulations must be observed during assessments. Hydraulic platforms or mobile scaffolding towers are preferable to long leaning ladders for longer periods of work at great heights, such as when the supporting structure of a sports hall is being assessed. Specialised equipment for inspecting the underside of bridges is generally necessary if a bridge is to be inspected thoroughly.

Figure 6: Buckling of compression-loaded members of a trussed girder

Structural elements must be systematically numbered or named while they are being assessed, so that the test measurement data can later be evaluated and attributed correctly. The individual elements should be named either according to position or by the axes on existing planning documentation, or, should the existing plans be lacking or incomplete, on plans drawn up in advance. In practice, self-adhesive numbers have also proved their worth in marking testing points and structural elements (Figure 7). This makes recording measurement results, photos and assessment documentation considerably easier, while also ensuring good orientation in larger and more complex structures. It is, furthermore, worthwhile using scaled plans of the most important structural elements, and preparing protocols in advance for measuring wood moisture and cracks, and for photographic documentation.

Figure 7: Systematically numbered investigation point

Regular monitoring of timber structures usually makes it possible to detect problem areas early. That is all that is required to avoid serious damage. This is why it is important that an assessment of a building's condition should always start with a meticulous visual and hands-on examination, and an inspection of every structural element. Any problem areas that have been found can, in combination with wood moisture measurements, be defined and examined with the use of additional testing methods. The cost of using specific measurement and testing equipment should, however, remain reasonable in relation to the resulting benefit. It makes sense to take an iterative or incremental approach with regard to methods and to the number of measurement points. At least 15 to 20 measurement results or samples are needed for a statistically reliable outcome. Experience shows that this cannot be justified in terms of cost, and because of restrictions on site. For this reason, an incremental approach is also advisable with regard to taking samples. Often, if an initial testing sequence of six to eight samples shows minimal variation in results, there will be no need for any further samples to be taken. Should that not be the case, it is essential for more samples to be taken in a second step.

For further modelling of load-bearing structures, it can be advisable to carry out additional load tests to investigate the load-bearing and deformation behaviour, and to record any deformations [3] (Appendix C). In contrast, using established methods to conduct strain measurements of wooden structural elements is difficult and often produces fairly uninformative results.

Assessments should be repeated at a later date in special cases, such as in buildings used for very different activities, or in indoor ice-rinks, where climatic conditions can vary greatly and differ according to season. Problems such as cracks or delamination – undetectable in the winter season when wood cross-sections are very humidified – become detectable once the wood has dried during the summer.

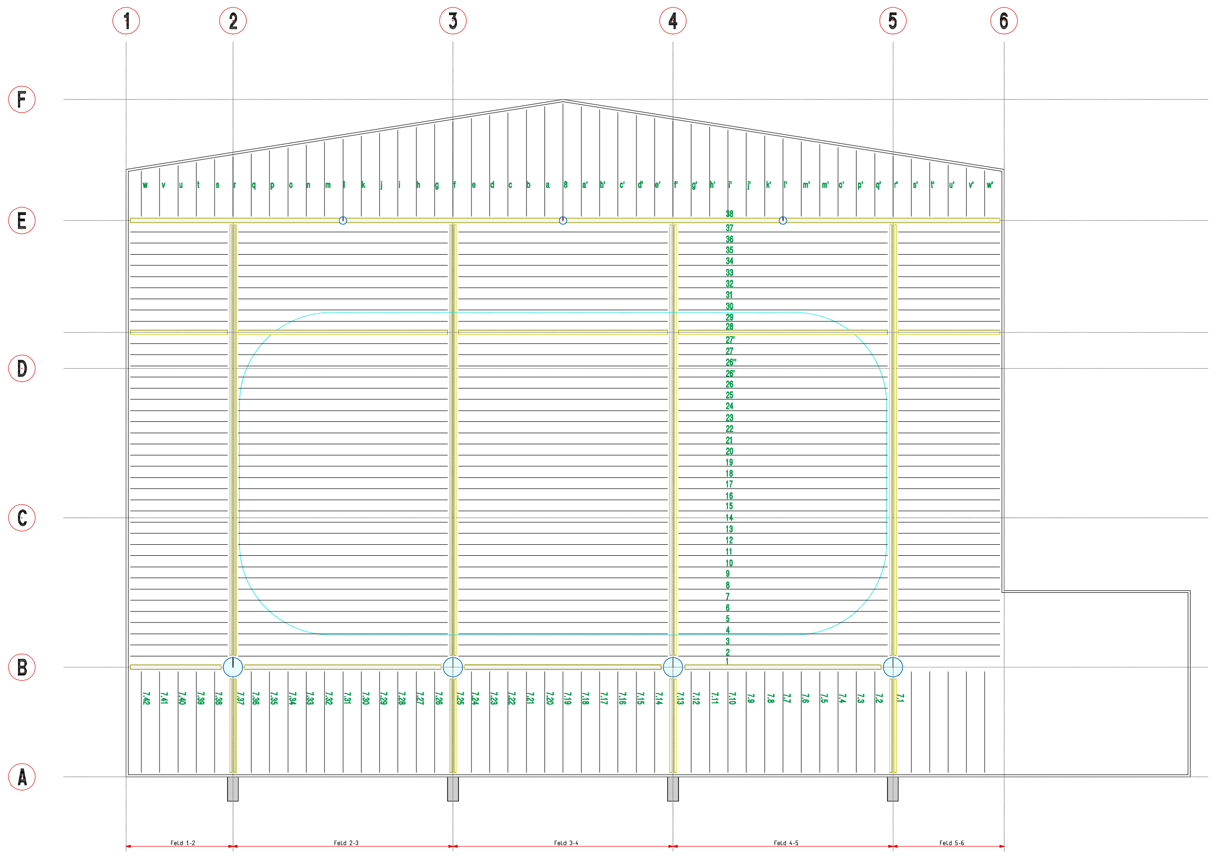

Figure 8: Planning documentation with the definition of the main axes

3 Conclusion

An advantage of wooden structures is that problem areas are relatively easy to see because of discolouration or cracking; in combination with wood moisture measurement, this means that problem areas can be well defined at an early stage. A good grasp of the load-bearing characteristics of a structure as a whole is of great importance during an assessment, so that any defects can be properly understood before any details, individual load-bearing elements or connections are recorded. Additional assessment methods are available for more comprehensive analysis and detailed measurement results. Each of the various procedures has its advantages and disadvantages. Combining different procedures often increases the value of the information.

References

[1] P. Dietsch, J. Köhler (eds) 2010: Assessment of Timber Structures, Shaker-Verlag, Germany, ISBN 978-3-8322-9513-4

[2] SIA 269:2011, Grundlagen der Erhaltung von Tragwerken, Schweizerischer Ingenieur- und Architektenverein, Zürich

[3] SIA 269/5:2011, Erhalten von Tragwerken – Holzbau, Schweizerischer Ingenieur- und Architektenverein, Zürich

[4] S. Aicher, B. Radovic, G. Folland, 2001: Befallswahrscheinlichkeit durch Hausbock bei Brettschichtholz, IRB-Verlag, Deutschland, ISBN 978-3-8167-5977-5

[5] Empa/Lignum 2001: Richtlinie Holzzerstörende Pilze und Insekten; Analyse, Prognose, Bekämpfung, Lignatec 14, Lignum, Zürich

[6] B. Franke, R. Widmann, 2012: Zustandserfassung und Verstärkung von Brettschichtholz, In: SAH-Tagungsband - Mechanische Verbindungen im mehrgeschossigen Holzbau, 44. Fortbildungskurs 2012, S. 193-202

[7] H.-J. Blass, H. Brüninghoff, H. Kreuziner, B. Radovic, S. Winter, 2006: Guideline for a First Evaluation of large-span Timber Structures, Council for Timber Technology, Wuppertal

Causes, assessment and impact of cracks in timber elements

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Summary

The increasing number of large timber structures and particularly of public buildings requires the assessment and monitoring of timber structures for safe, reliable timber constructions. Within the assessment, one obvious sign are cracks, which occur due to the natural behaviour of wood but also due to physical and/or mechanical excessive stresses. The paper present, categorize and discuss the causes of cracks, explains test standards for the quality control of glulam members and shows possibilities for the assessment.

Key words: Timber, Assessment, Cracks, Impact

1 Introduction

Impressive large span constructions are more often realized in timber or engineered wood products over the past years. For example, glulam beams provide the possibility for the use of timber in halls, bridges and multi storey buildings. The limitation in span of solid timber members was lifted due to glulam production. But the environmental conditions are different for these constructions. A quite high humidity with constant temperature is prevailing e.g. in roof constructions for swimming pools whereas low temperature and dry conditions are prevailing in ice sport arenas. The climate changes have an influence on the long term behaviour or can even lead to cracks and thus to a weakening of the timber members. With an increasing number of large timber structures, and particularly of public buildings, the assessments and monitoring of timber structures has becomes a crucial topic.

For the design of the structure, various materials as single-material, engineered or composite product are available but again with different mechanical and physical properties or manufacturing processes. Figure 1 gives an overview of external and internal influences on the structural system and the possible failures during the life cycle of timber structures. Reasons for failures are e.g. design errors, climate situations different from those at the time of initial design of the structure or (intended or accidental) increases in load.

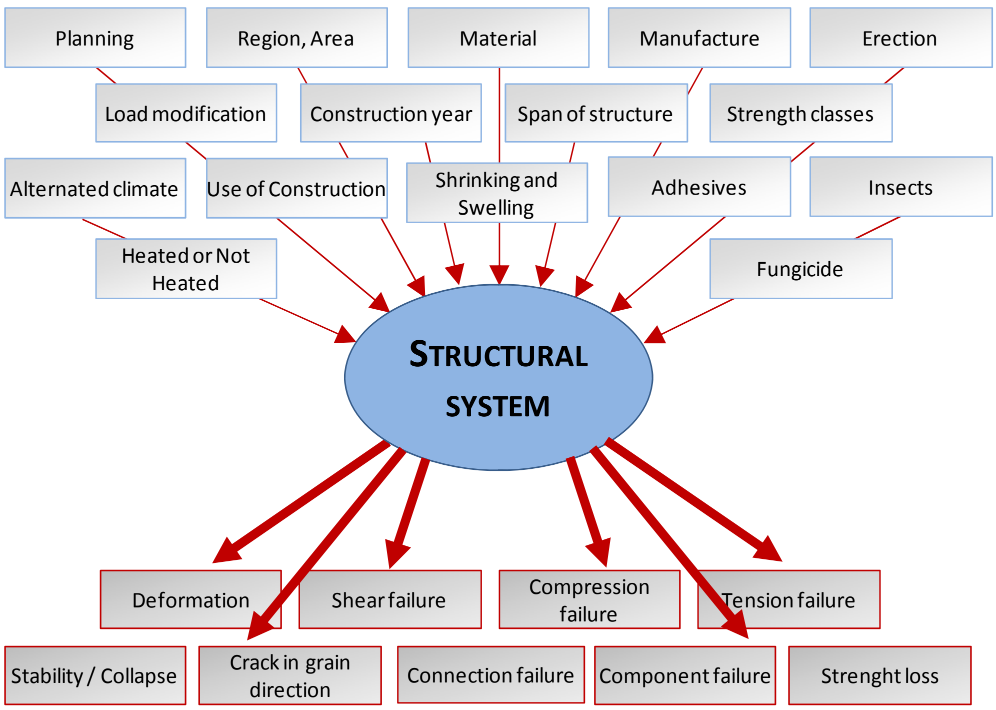

Figure 1: Impacts and reaction on the structural system and their action, according to Blass & Frese [1]

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When assessing a timber structure, one of the first and most obvious sign of damage is the presence of cracks in the elements. Indeed, cracks are clearly visible in dry conditions. Such cracks are often noted in large span timber elements which are more impacted by climate variations. Indeed, large span elements usually correspond to large cross sections. And, while moisture diffuses across the timber section, large cross sections tend to develop severe moisture gradient and thus severe moisture related stresses. These stresses are often released by the creation of cracks. These cracks propagate in the grain direction as a consequence of the wood low resistance in tension perpendicular to the grain. Both standards and literature agree on the major relevance of these cracks.

2 Crack situations and causes

2.1 Crack characteristics

The failure analysis on timber structures in Germany done by Blass & Frese 2010 [1] gives an good overview of the distribution of main failures classified according to the construction, usages and region, Figure 1. With regard to their type, they can be categorized in (i) crack failure in grain direction and (ii) shear and tension failures, as shown in Figure 2. The category “others” includes failures such as decay, critical distortion or change in colour of the timber. The cracking in grain direction can lead to a very brittle failure of the structural member and thus to the collapse of the complete structure, this mainly due to the anisotropic material behaviour of wood. The strength of wood in tension perpendicular to the grain is small compared to the tension strength in grain direction or even zero at the area of natural defects. Therefore first signs of delamination in glulam members or cracks respectively are of major importance for the assessment of the residual load-carrying capacity of timber structural elements.

The information gathered in [1-4] as well as in internal assessment reports from Bern University of Applied Sciences were statistically analysed. Two sets of results were obtained: the characteristics of timber elements and the types of crack distributions.

Most assessment reports state that the timber structures have been built using glulam beams of quality GL28h (see Table 1). Their shape, however, is more distributed with the most commons being, by order: straight 36 %,

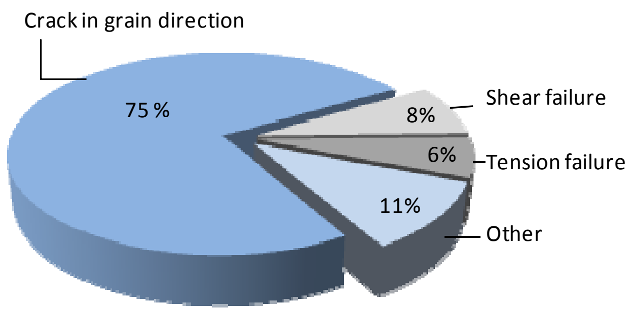


Figure 2: Distribution of type of failures, according to [1], [2]

Table 1: Most frequent characteristics of the timber structures assessed

Characteristic	**Main result**	**Corresponding number of assessments**	
Material	Glued laminated timber	594	80%
Quality (or equivalent)	GL28h	68	72%
Loading situation	Bending	470	80%

Table 2: Characteristics of crack distributions

Cracks cause	**Location / Amount**	**Length / Depth ratio**	**No.**
Stress concentration (Restrained shrinkage, notches...)	From the Singularity/Solitary	1 - 10 m / 1	35%
Normal climate changes	Random / Numerous	0.1 - 1 m / 0.1 to 0.4	33%
Element quality (Glue line or finger joints)	From the defect/ Depending on its extend		17%
Overloading (Shear or bending stresses)	Mid span/ Solitary to numerous	1 m / 1	15%

tapered straight 29 % and pitched cambered 21 %. The spans of the timber elements follow a normal distribution with a mean of 23 m. The distribution of the widths, though, is less clear as the number of assessments providing this information was much lower. The width of 140 mm corresponds to both the median and the mode of the distribution. The height to width ratio of the elements was also obtained for 37 assessments and results in a mean value of 4.3.

Four main causes of crack initiation have been found; see Table 2. Cracks due to the concentration of stresses and to an overloading of the elements show similar properties: they are long cracks (more than 1 m) and are going through the elements (depth ratio of 1). However, there locations as well as their number vary depending on the cause of the crack. The climate changes to which a timber element is subjected during its service life produce small and shallow cracks which are often numerous. Finally, the cracks due to poor quality of manufacturing are obviously directly linked to the extent of the defective material and therefore vary from case to case.

2.2 Moisture induced cracks, due to hygroscopic behaviour of wood

The hygroscopic behaviour of wood describes the adsorption and desorption of moisture to maintain equilibrium depending on the surrounding climate in particular relative humidity and temperature. The adsorption of moisture occurs in two steps in the range from 0 % to 30 % where the moisture is transferred into the cell walls of the wood. Above 30 % moisture content, the cell walls are completely saturated and the moisture is transferred into the cavities of the cells. The moisture content of 28 - 30 % is called fibre saturation point. The fibre saturation point varies depending on the wood species. Changes in the moisture content below the fibre saturation point affect the physical, mechanical and rheological properties of wood, like the shrinking and swelling, the strength values or the modulus of elasticity or rigidity, [6], [7]. The dimension changes are different in the three material axes (longitudinal, tangential or radial) as principle shown in Figure 3.

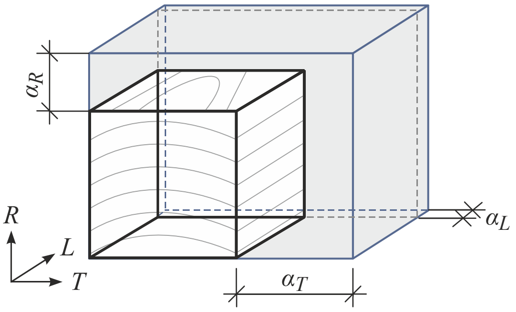


Figure 3: Differential shrinking or swelling depending on the material direction

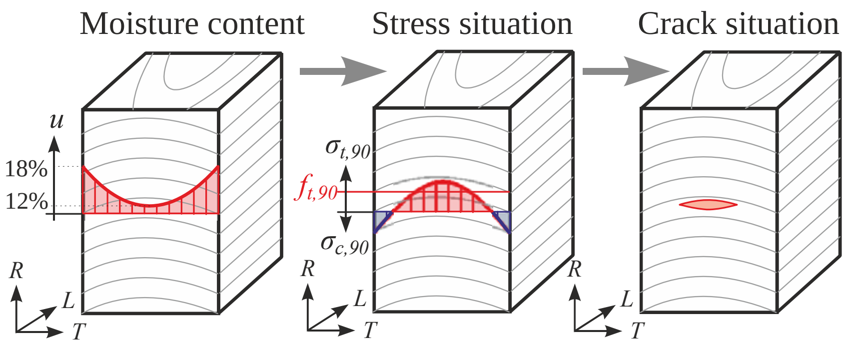


Figure 4: Gradual increase of the moisture content and the stress reaction respectively crack growth

To reduce the initial change of moisture content in wooden members, they should be conditioned in such way that they meet the average moisture content which is expected in service. However, glulam and also block-glued glulam is been produced with a moisture content of 8 % to 15 % according to EN 386:2001 [8] and will then mostly be installed with this moisture content, but the moisture content in service can be much higher depending on the application. Members in bridges in normal European climate conditions are expected to have a moisture content of around 15 % to 20 %, [9]. The moisture content within service class 2, for example, is allowed to vary of 8 % according to EN 1995-1-1:2004 [10]. It has to be noticed that a gradual increase of the moisture content of less than 1 % moisture content can theoretically already lead to excess of the material strength perpendicular to the grain (applying the characteristic properties for GL 24h). A gradual increase or decrease of the moisture content happens easily within the constructions and results in dimensional changes of the cross section and also in internal stresses, as shown in Figure 4.

2.3 Stress induced cracks, due to the mechanical performance

Several failure cases respectively the appearance of cracks are stress related. If the strength of the material, mainly bending, tension or shear, is exceeded, cracks can develop in the cross section which can significantly reduce the capacity, see Figure 5 and Figure 6. Further information can e.g. be found in [11].

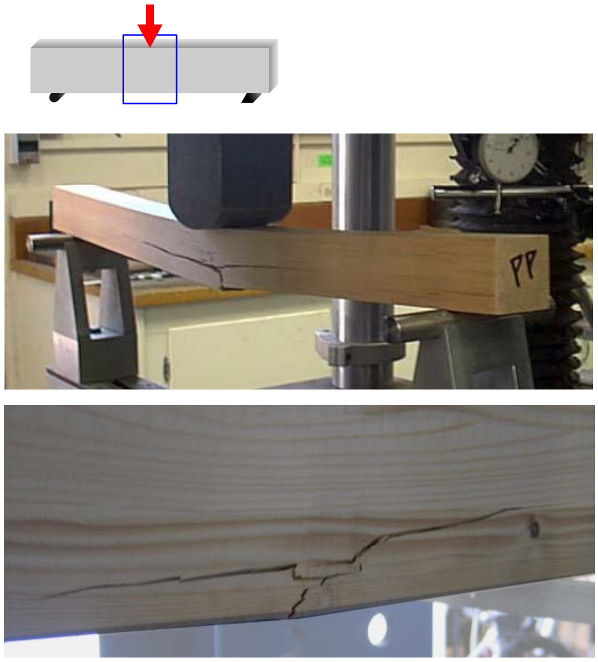


Figure 5: Tension failure under bending

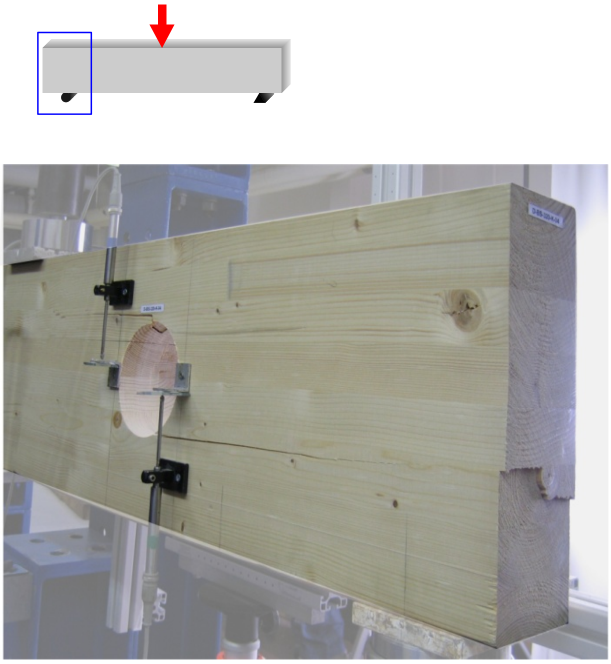


Figure 6: Shear failure at holes or at end supports

3 Material - Glulam

3.1 General

Wood is natural grown material which limits the available cross sections and lengths for large span constructions, bridges or even for multi-story buildings. However, glulam can overcome this limit. It is a well established engineered wood product of single lamellas of solid wood glued together. The lamellas are finger jointed in length and glued together in the depth of the cross sections. Glulam is commonly produced of European spruce, fir or larch and nowadays also of hard wood like ash, beech or oak, [12], [13], [14]. The adhesive used for the lamination of the lamellas ranges from casein (historical) to a wide range of modern glues like polyurethane (PUR), resorcinol-formaldehyde (RF) and melamine-urea-formaldehyde (MUF).

3.2 Quality control

For the quality control of the glulam production, the requirements for a delamination test according to EN 391:2001 [15] and shear strength test according to EN 392:1995 [16] are given in EN 386:2001 [8]. The delamination test setup has been used to proof the resistance against the climate exposure during the life time of the glulam member. The test begins with a fully adsorption of water of the specimen by applying controlled vacuum and compression cycles while the specimen is under water. Finally the specimen has to be dried climate controlled using an oven with air circulation. The wet and dry cycle results in tension stress in radial direction respectively transverse to the glue line in the specimen. For the assessment, the sum of the openings along the glue line and at the end grain developed during the tests has to be taken in relation to the total glue line length at the end grain of the specimen. To respect different service classes according to EN 1995-1-1:2004 [10] the delamination test standard differs between three methods for the wet/dry cycles. The block specimen, shown in Figure 7, needed for the quality control of the production can easily be extracted. But for the assessment of existing structures, the block specimen defined in EN 391:2001 [15] cannot be extracted from the structure. Figure 9 shows as example one test specimen after the delamination test where the openings are clearly marked. Figure 11 shows as example the assessment of the results of a delaminations test series observed.

The second required test for the quality control of glulam is a shear test of the glue line loaded in longitudinal direction. The test standard provides two specimen shapes, on the one hand the block or bar with a cross section of 50 mm by 50 mm including all glue lines of the member depth and on the other hand a drill core including only one glue line as shown in Figure 8. The core specimen is here commonly used for the assessment of existing glulam structures. Parallel to the shear strength, the percentage of wood failure (PWF) at the failure plane after testing is visually examined, as shown in Figure 10 and Figure 12.

Please note, that no correlation between the delaminations and shear test of the standards could be observed within recent research projects, [17]. Therefore the results of the observed shear resistance have to be used and assessed carefully for the assessment of existing timber constructions.

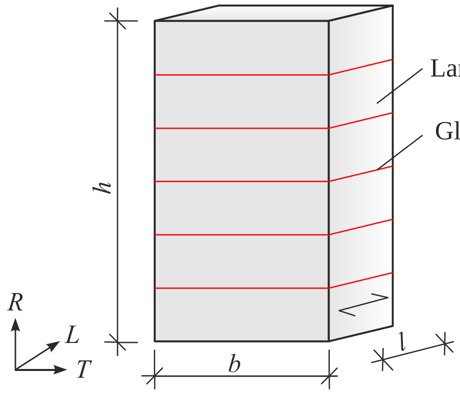


Figure 7: Test specimen for delamination at a block

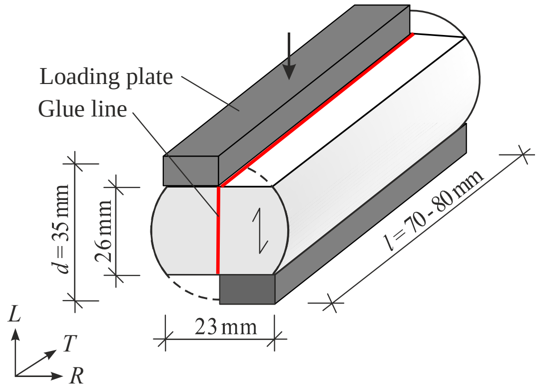


Figure 8: Test setup for core, EN 392:1995

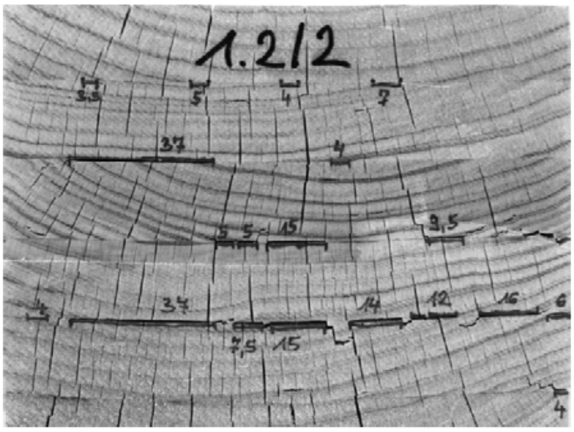


Figure 9: Standard test specimen after the test with marked delamination

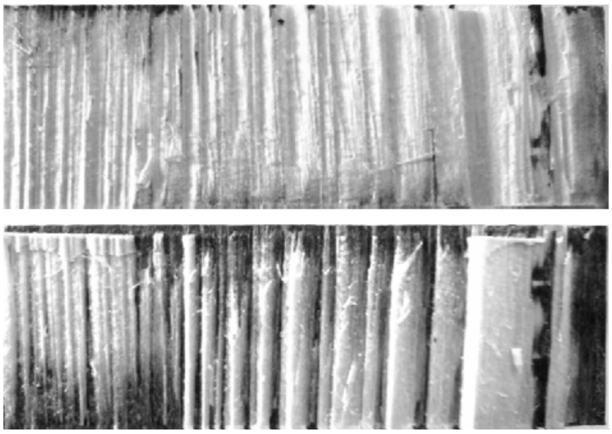


Figure 10: Fracture surface with wood failure of a tested drill core

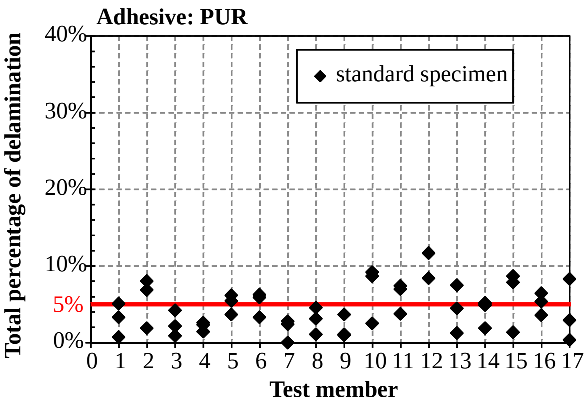


Figure 11: Example of delamination test results for adhesive PUR

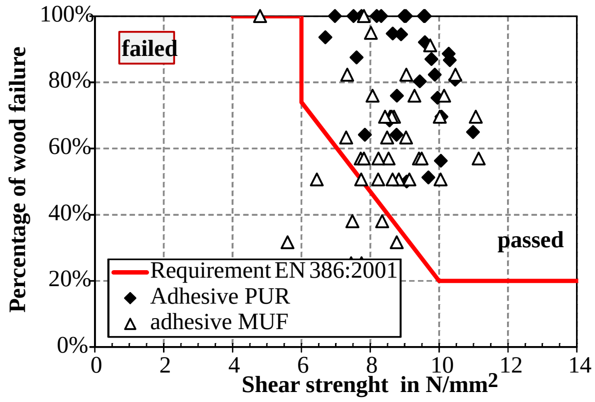


Figure 12: Example of shear test results for the test series with PUR and MUF adhesives

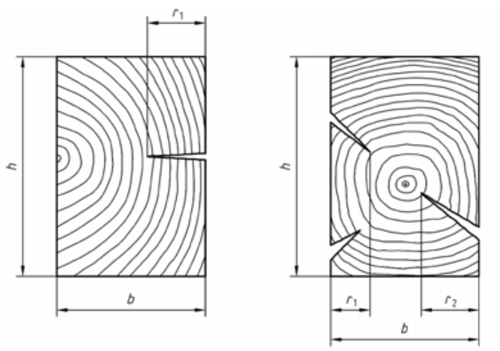
4 Assessment and impact of cracks

4.1 Current regulations

For the assessment of cracks the crack kind, the measured sizes as well as the number has to be judged individually for each case. Particular the type of structure, the structural system as well as the function of the member within the complete structure have to be considered, e.g. main or second girder. Further the current as well as the future condition of use e.g. the climate conditions are essential for the assessment. For the limit state design, existing cracks reduces the effective cross section of the member for tension and bending tension stress under an angle or perpendicular to grain, torsion stress, shear stress and notches or holes.

The weakest point in timber structures is often the low tensile strength perpendicular to grain which leads to cracks in the cross-section and along the span of the member. For the assessment of such cracks the standard SIA 269/5 gives some guidelines how to consider existing cracks in the evaluation of timber structures. Furthermore the international standard DIN 4074-1 for strength grading of solid softwood provides some specifications on maximum crack depths as well. In this standard the maximum crack length is restricted to 1 m. In research papers by Frech [18] and Radovic & Wiegand [19], further specifications are given for the consideration of cracks. However, as shown in Figure 12, there is no consistency in the restrictions in the crack ratio. The published criteria in the standards and the research reports vary in a wide range from R = 0.125 to 0.5 and neither the actual stress situation is always considered nor the stress combinations or the position of the crack along the span which is an important criterion, nor the crack length.

It can be summarized, that the current practice in the assessment of existing timber structures including the influence of different crack situations on the load carrying capacity may not be considered suitable whether to facilitate confident decisions about the reliability of the structure nor to evaluate the residual load-carrying capacity of structural elements [20].



$$R = \frac{r_1}{b}, \qquad R = \frac{r_1 + r_2}{b}$$

Figure 13: DIN 4074-1:2003 Determination of crack depths r for square-cut solid timber beams

4.2 Development of reliable calculation methods

A recent research project at the Bern University of Applied Sciences is dealing with the impact of cracks regarding the stiffness and load carrying behaviour of timber elements. The residual load carrying capacity is analysed by numerical and experimental investigations. This numerical investigation consisted in simulating a three-point bending test of a glued laminated beam presenting one initially opened crack. The numerical model of the glued laminated timber beam used was defined in agreement with the results of the first step of this research. The numerical simulation results reveal that the distinction between through cracks and not through cracks should be made. Indeed, both in terms of stiffness and load-carrying capacity, these two cases have a different impact on the beam performance.

First the simulations revealed that not-through cracks could be neglected regarding the global stiffness of the beam. Moreover, a calculation model was proposed to estimate the reduction of stiffness for a beam presenting a crack. This simple calculation model showed a good correlation with the results obtained from the numerical simulations. However, further investigation would be necessary to define more precisely.

Then, several trends have been highlighted to correlate the residual load-carrying capacity of a cracked beam with the crack characteristics. In the case of through cracks, the residual load-carrying capacity of the beam has been correlated to the crack height in the cross section and to the crack length using a parabolic and a power law, respectively. In the case of not-through cracks, a trend has been identified; however, further research would be needed to effectively correlate the crack characteristics to the beam residual load-carrying capacity.

Overall, the project is continuing with experimental investigations in order to validate the model and the results of the model and furthermore in order to develop calculation methods for the residual load carrying capacity and stiffness.

5 Conclusion

There are several impacts and influences on a structure which can lead to cracks in timber members. The research aimed at characterizing the situations where cracks are found in timber structures. Two types of result were obtained from this investigation. First the characteristics of the timber elements where cracks have been found were collected. This showed that cracked large span timber structures mainly deal with straight glued laminated timber beams loaded in bending. Moreover, information regarding the dimensions of such beams was obtained. Thus, these elements have an average span of 23 m, a medium width of 140 mm and a corresponding average height-to-width ratio of 4.8.

Then the first step of this study showed that cracks can be characterized and categorized according to their causes of origin. The two main reasons for the development of cracks, due to moisture and stresses, were highlighted and explained.

The assessment of cracks goes along with the quality control of glulam members where delaminations tests have to be performed. Both delaminations and cracks can be tested with a shear test setup and analysed by inspection of the shear failure surface. Either delaminations or cracks can have a significant influence on the stiffness and load carrying capacity. While through cracks influence the stiffness, not through cracks influence the load carrying capacity. Investigation in order to determine reliable methods for the prediction of these are currently carried out at the Bern University of Applied Sciences.

6 Acknowledgement

The research work is within the COST Action FP 1101 – Assessment, Monitoring and Reinforcement of timber structures. The Swiss State Secretariat for Education, Research and Innovation (SERI) proudly supports the research work. The project work was supported by the Federal Office for the Environment (FOEN). Some investigations have also been done by Noëlie Magniere during her master thesis.

References

[1] Blass, H. J., Frese, M. (2010): Schadensanalyse von Hallentragwerken aus Holz. Band 16 der Reihe Karlsruher Berichte zum Ingenieurholzbau, KIT Scientific Publishing, ISBN 978-3-86644-590-1, Germany.

[2] Vogel, M. (2008): Überwachung von Bauwerken. Proceedings, Holzbautag in Biel, Switzerland.

[3] Frühwald E. et al.: Design of safe timber structures - How can we learn from structural failures in concrete, steel and timber?. Lund University, Division of Structural Engineering, Lund, Sweden, 2007.

[4] Colling F. and Müller T.: Lernen aus Schäden im Holzbau [Ursachen, Vermeidung, Beispiel]. Deutsche Gesellschaft für Holzforschung, München, Germany, 2000.

[5] Dröge G. and Dröge T.: Schäden an Holztragwerken. Fraunhofer IRB Verlag, Stuttgart, Germany, 2002.

[6] Neuhaus, F. H. (1981) Elastizitätszahlen von Fichtenholz in Abhängigkeit von der Holzfeuchtigkeit. Technical Reports, Ruhr-University of Bochum, Germany.

[7] Niemz, P. (1993) Physik des Holzes und der Holzwerkstoffe. DRW-Verlag, Leinfelden-Echterdingen, Germany.

[8] European Committee for Standardization (CEN), EN 386:2001, Glued laminated timber, Performance requirements and minimum production requirements, Brussels, (2001).

[9] Scharmacher, F., Müller, A. (2012) Erfahrungen und Konsequenzen aus der Langzeitüberwachung von Holzbrücken. Bridge Symposium - Grünbrücken aus Holz, Stuttgart, Germany.

[10] European Committee for Standardization (CEN), EN1995-1-1:2004, Eurocode 5, Design of timber structures - General - Common rules and rules for buildings. Brussels

[11] Franke, S., Franke, B., Harte, A. (2014 in publication) Reinforcement of timber beams, J Civil Struct Health Monit, Springer Link, Heidelberg

[12] Blass, H. J., Denzler, J., Frese, M., Glos, P., Linsemann P. (2005) Biegefestigkeit von Brettschichtholz in Buche. Publisher University of Karlsruhe, Germany.

[13] Frühwald, A., Ressel, J. B., Becker, P., Pohlmann, C.M., Wonnemann, R. (2003). Verwendung von Laubhölzern zur Herstellung von Leimholzelementen, Research report, University Hamburg, Germany.

[14] Information on http://www.grupo-gamiz.com/en/0202.html, 13/02/2013

[15] European Committee for Standardization (CEN), EN 391:2001, Glued laminated timber, Delamination test of glue lines, Brussels, (2001).

[16] European Committee for Standardization (CEN), EN 392:1995, Glued laminated timber, Shear test of glue lines, Brussels, (1995).

[17] Franke, B., Scharmacher, F., Müller, A. (2013) Assessment of the glue-line quality in glued laminated timber structures. Proceedings, Shatis 2013, Italy.

[18] Frech, P. (1986): Beurteilungskriterien für Rissbildungen im konstruktiven Holzbau. Research Report T1885, Frauenhofer IRB-Verlag, Germany.

[19] Radovic, B., Wiegand , T. (2005): Oberflächenqualität von Brettschichtholz. bauen mit Holz 7, pp. 33-38.

[20] Dietsch, P., Kreuzinger, H. (2011): Guideline on the assessment of timber structures: Summary, Engineering structures 33, issue 11, pp. 2983-2986

[21] Schweizerischer Ingenieur- und Architektenverein (2011): SIA-Norm 269/5: Erhaltung von Tragwerken – Holzbau (Standard SIA 269/1: Existing Structures – Timber Structures). SIA, Zurich, Switzerland.

Measuring Methods for Moisture Content of wood

Ulrich Berger[1]

Summary

Measuring the wood moisture content is important to avoid moisture damages. This article illuminates different measurement methods for determining the wood moisture content, exemplarily for resistance measuring instruments, dielectric constant measuring instruments, sorption isotherms and the gravimetric (oven-dry) measurement method.

It shows the influence of factors like temperature, density, equilibrium moisture content, type of used electrodes. Also the additional measurement of moisture in building materials will be touched.

Key words: Resistance measurement, Dielectric constant, Sorption isotherms, Gravimetric measurement

1 Introduction

Measuring the moisture content of wood and wood products gets more and more important to avoid damages in buildings, floors, furniture, bridges, halls. Also important is the measuring of the water content of the surrounding building materials of the wooden products, otherwise the moisture can run over to the wood, for example in parquet. Depending on the climate in a room or building, which is described by the relative humidity and the temperature of the air, wood adapts to a so called equilibrium moisture content, if it is long enough exposed to these surroundings.

2 Methods of measuring wood moisture

Electrical wood moisture measuring instruments

Resistance measuring instruments

Dielectric constant measuring instruments

Sorption isotherms

Gravimetric (oven-dry) measurement method

2.1 Resistance measuring instruments

The measurement of electrical resistance in wood is still the method most frequently used for the determination of wood moisture.

Absolutely dry wood has a very high electric resistance. The presence of moisture in wood reduces the electric resistance. As the moisture content of the wood increases, the electrical resistance decreases, so that a greater current can flow in the wood.

If this change in resistance is considered in relation to the wood moisture content, it can be seen, that there is a regular relationship that, with a few exceptions, is similar for almost all species of wood.

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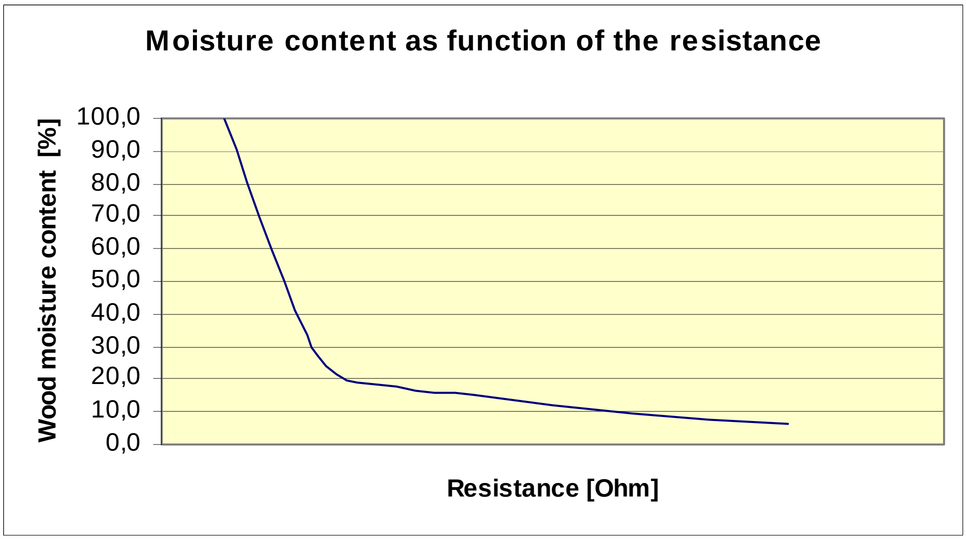

Figure 1: Moisture content as function of the resistance

Meters with built-in electrodes:

Figure 2: Typical meters with built in electrodes

Meters with external drive-in-electrodes, or with external ram-in-electrodes for thick woods or hardwoods:

Figure 3: Typical meters with external drive-in-electrodes

2.1.1 Moisture distribution inside the wood

The moisture content of the wood can be inhomogeneous as shown in the left part of the board, or equalized through the whole thickness as in the right part of the board. If the wood is equalized it makes no difference, in which depth you measure the moisture content. That means, the electrodes can be some millimetres or some centimetres inside the wood.

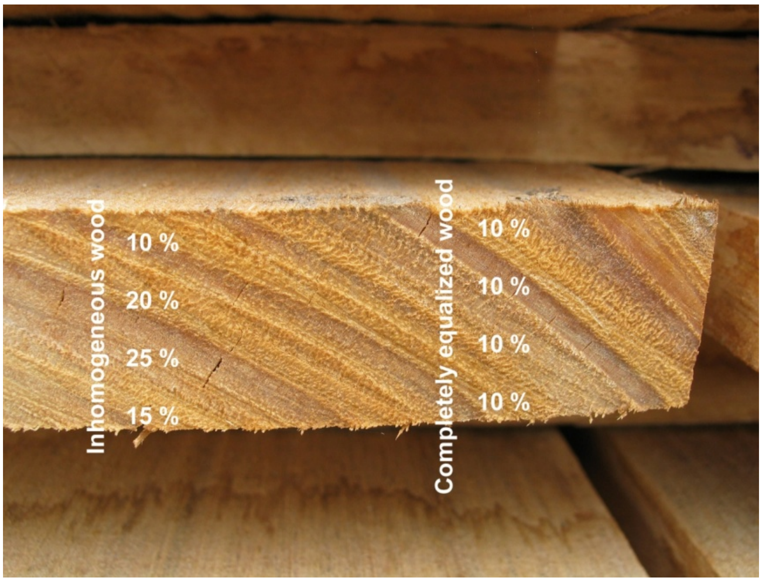

Figure 4: Moisture distribution inside the wood

2.1.2 Insulated electrodes

If you use insulated electrodes, the measuring area is only at the non-insulated tip of the electrodes (see white bar). If you bring in the electrodes into different deepnesses you can establish a moisture profile through the piece of wood.

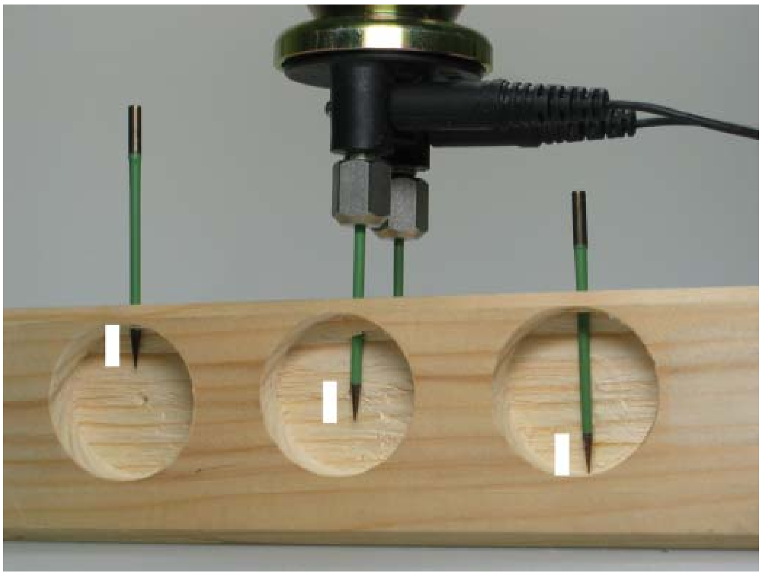

Figure 5: Insulated electrodes

2.1.3 Non-insulated electrodes

If you use non-insulated electrodes, you measure always the wettest part of the wood along the electrodes (see long white bar), but you do not know, in which depth this value is.

Surface electrodes are suitable primarily for measuring the moisture content of veneers and thin boards.

In the case of very thin veneers it is best to measure several layers of veneer, to prevent the measurement from being affected by the layer beneath. Long electrodes should always be preferred for the measurement of sawn timber, squared timber and lumber. The appropriate selection of penetration depth permits the determination of average or maximum values.

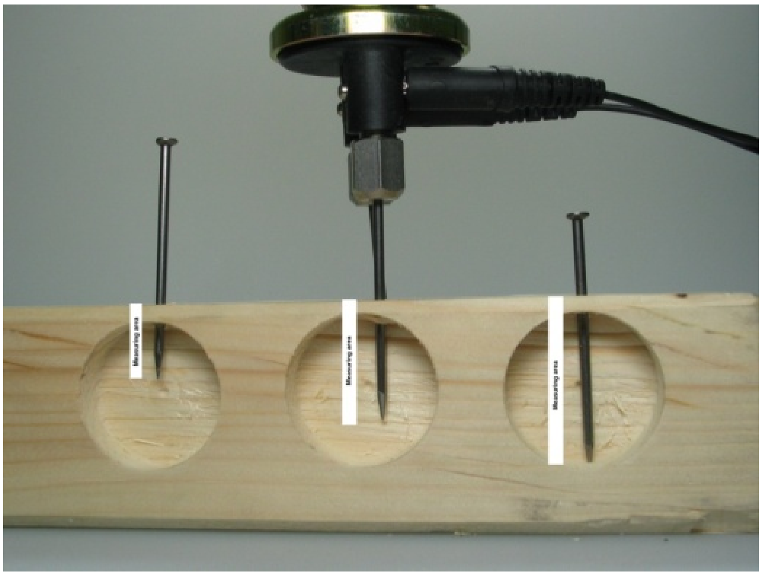

Figure 6: Non-insulated electrodes

2.1.4 Temperature influence

The electrical measurement of wood moisture is significantly influenced by the temperature of the wood to be measured. The electrical resistance of wood varies not only with the moisture content, but also with the temperature. With a constant moisture content, the resistance decreases with increasing temperature. When the temperature falls, the resistance increases.

This temperature dependence is not constant, but becomes more pronounced as the wood moisture content increases. In simple wood moisture measuring instruments, the scale is generally designed for a wood temperature of 20 °C. This means that if the temperature deviates from 20 °C, the value displayed by the instrument will no longer correspond to the actual wood moisture content.

For temperatures less than 20 °C, the moisture content displayed will be too low.

For temperatures greater than 20 °C it will be too high.

Correction of the values obtained is therefore necessary, with the aid of an appropriate correction table.

Other wood moisture measuring instruments are equipped with an appropriate temperature compensation. Here the wood temperature can be set directly on the measuring instrument and is then automatically taken into account when the wood moisture content is displayed. In the case of measuring instruments which lack such temperature compensation, a wood moisture content deviation of approximately 1% can be assumed for each 10 °C to 20 °C deviation in temperature, provided that the wood is relatively dry .

2.1.5 Species correction

The wood moisture value displayed is dependent not only on the temperature, but also on the species of wood to be measured. Instruments of higher quality, therefore, have a wood type selector, which has settings for several different types of wood.

A table can be used to determine which wood type corresponds to the species of wood to be measured.

The wood type selector permits additional automatic correction of the measured value, thus improving the measurement accuracy.

In general, for most applications, the electrical resistance procedure is the most accurate measurement method for determining wood moisture content.

However, a certain amount of damage to the material is associated with the use of deep electrodes.

2.2 Dielectric constant measuring instruments

Dielectric constant measuring instruments are non-destructive, and only need to contact the surface of the wood. The dielectric constant of absolutely dry wood (0% moisture content) is compared with that of the moist wood:

The dielectric constant of dry wood is very low, while that of water or wet wood is very high.

However, the magnitude of the dielectric constant is decisively influenced by the bulk density of the wood as well as by its chemical composition.

The bulk density is known to vary considerably, even within a single species of wood.

However, this variation is reflected linearly, i.e. directly in the measurement result.

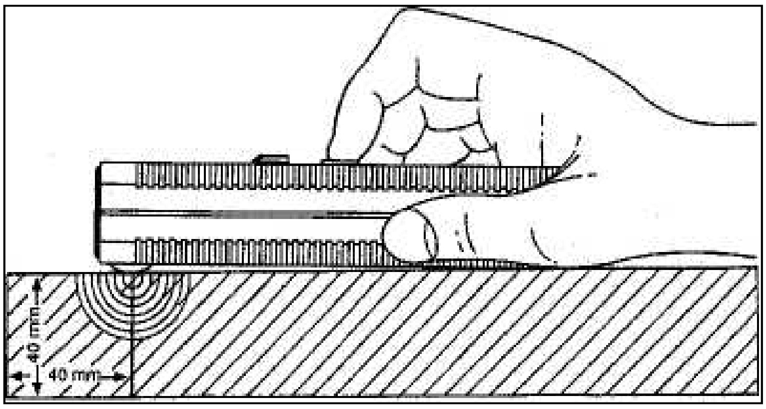

Figure 7: Dielectric measuring methode

2.2.1 Influence of the density, species correction

Instruments which operate according to this measurement procedure therefore have a correction switch for different bulk densities (which of course presupposes an estimate of the bulk density which is as accurate as possible), or they are provided with several scales to which the various wood species are assigned in accordance with their average bulk densities.

However, the attenuation of the electric field with increasing depth, from the outer zone towards the centre of the wood, is not linear. The electric field decreases logarithmically with depth, so that the deeper moister layers contribute only fractionally to the measurement result.

In the case of wood with completely uniform equilibrium moisture content, with no difference between the core and the outer zone, the dielectric constant measurement procedure yields accurate measurement results, since the outer zone, which decisively influences the measurement result, has the same moisture content as the core zone.

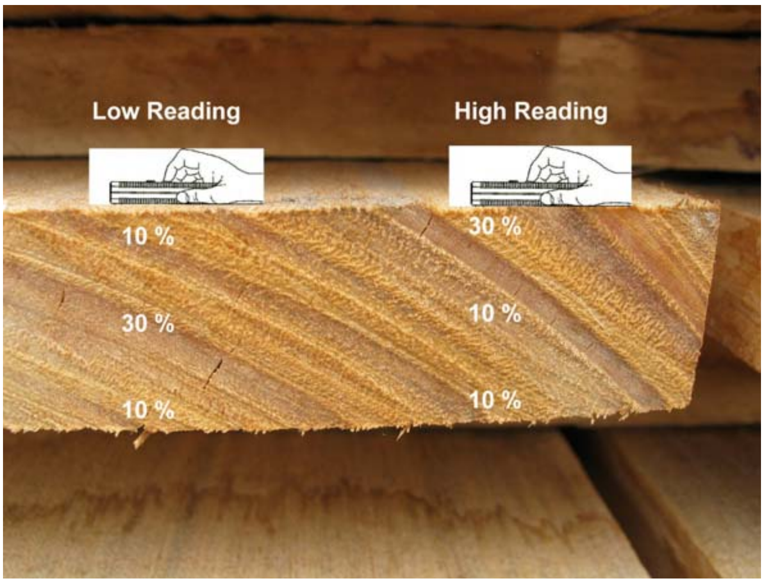

Figure 8: Influence of the density, species correction

However, in the case of boards e.g. 60 mm thick with a high core moisture content, it must be kept in mind that the measurement result is only partially influenced by the higher moisture content at the centre of the wood:

The value displayed by the instrument is a combined value which primarily reflects the moisture content of the outer zone and only partially reflects the higher moisture content at the centre of the wood.

Advantages

No damage to the material, simple and fast measurement.

Disadvantages

The value displayed by the instrument is a combined value which primarily reflects the moisture content of the outer zone, and only partially reflects the higher moisture content at the centre of the wood.

Furthermore, the roughness of the surface can affect the measurement result.

Only in exceptional cases (e.g. wood with a completely uniform equilibrium moisture content or thin boards) the measurement accuracy equals that of electrical resistance measuring instruments.

It is not possible to obtain information concerning differences in moisture content between the core and the outer zone of the wood.

2.3 Sorption isotherms

An indirect method to determine the wood moisture content is the measurement of the relative humidity of the air in a drill hole in the wood. Via a sorption isotherm curve the wood moisture can be found out. This sorption isotherm curve can be stored in a meter designed for relative humidity of the air, to show directly the wood moisture content.

Some meters show also an equilibrium wood moisture content (EMC) which is calculated from the relative humidity and the temperature of the air.

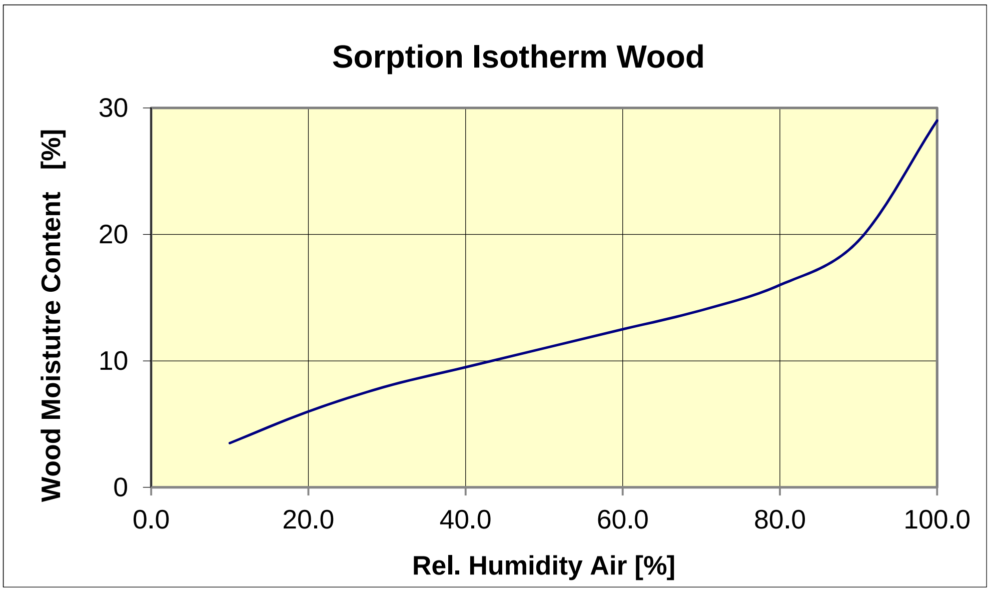

Figure 9: Sorption isotherm of wood

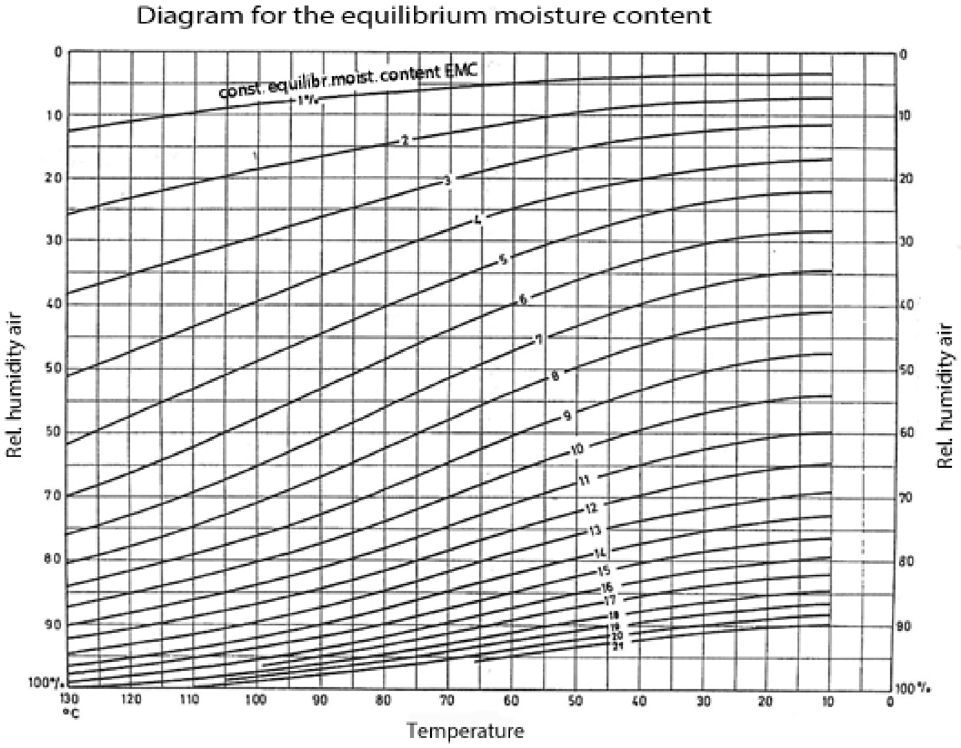

Figure 10: Diagram for the equilibrium moisture content

2.4 Gravimetric (oven-dry) measurement method:

A very exact method to determine the wood moisture content is the oven-dry method.

A sample is taken of the material to be measured, the sample is weighed and dried for an extended period (up to 24 hours or more), until a constant weight is achieved. Afterwards the sample is weighed again.

The difference in weight is then used to calculate the original moisture content according to the following formula:

$$\frac{(\text{wet weight} - \text{dry weight}) \times 100\ \text{dry}}{\text{dry weight}} \qquad (1)$$

Advantages of this method: Theoretical accurate results

Disadvantages: Takes a lot of time; during sampling, transport and evaluation of the moisture results, errors may be introduced from a variety of hidden sources.

3 Results and discussion

Resistive measuring is still the most used method. Fast measurements with good accuracy below the fibre saturation point are possible. With insulated electrodes the moisture gradient within the material can be determined.

Capacitive measurements, with dielectric constant measuring instruments offer a non-destructive fast result. If the moisture content is equalized over the profile of the wood, the result is quite accurate.

4 Conclusion and future work

Electrical moisture meters are necessary in the building sector to avoid damages in buildings. The most important part of a meter is a stable amplifier, which has a good accuracy in the low moisture range.

The meter should be easy to use, the moisture content, the selected species and the correction temperature should be seen on the display or via a switch position.

A lot of new wood based construction materials will have to be examined to determine the moisture curves, that are stored in qualitative measuring devices.

Failure cases of mechanical fasteners in timber connections

Pierre Quenneville[1]

Summary

Connections are an important part in a timber structure. Their design can account for a very significant portion of the overall design effort of the structure, yet not every designer is familiar with all the particularities of their design rules. On the potential failures of connections, some of them are the result of inadequate detailing combined with the effect of in-service conditions. Some of these are illustrated in this paper. On the issue of failures related to inadequate resistance, recent research efforts have identified some of the more significant parameters influencing resistance, either for ductile or brittle failures. One principle which is well accepted is that connection can be designed to fail in a ductile manner and this is usually achieved by using small diameter fasteners. The other principle that is also accepted is that connections with mechanical fasteners, usually larger ones, will result in brittle failures. Brittle failures for large and small fasteners are identified in this paper.

Key words: Timber, Wood, Connection, Fastener, Failure, Ductile, Brittle

1 Introduction

Connections are an important part of any structure. Their design can be a significant portion of the design effort and the knowledge required to cover all the particularities does normally occupy a great portion of any design standard. Over the years, their design has mainly been either covered by a series of simplified design equations with the addition of modification factors to attempt to catch all situations or through lengthy design tables listing resistance values for very specific configurations.

The failures of mechanical connections are usually a result of; a design blunder, inadequate detailing or inadequate strength due to poor knowledge and control of their behaviour. Design blunders occur when a specific issue has been verified but a mistake in a calculation has been done. This is common not only to connections and there is not much that research can do in order to diminish their occurrence. Inadequate detailing usually result from the absence of the specific knowledge of timber construction, usually for building elements that are exposed to water, and a resulting absence of the details required to protect timber. Again, for this second one, research is not the right action where efforts should be concentrated in order to minimise their occurrence. On the issue of inadequate strength, this is where research has been making progress and knowledge of the effects of connection variables is being acquired.

Since the early days of timber research, the resistance of connections, usually for bolts, has been given through design rules covering their ductile behaviour but with the addition of modification factors to cover cases where brittle failures are observed, usually associated to the group effect phenomenon. In the last 50 years, much of the efforts on connection of mechanical fasteners have concentrated on the resistance of ductile and brittle failure modes, identified as separate issues.

The topic of design blunders is a delicate one and not specific to timber or to connections, usually attributed to a mistake during the design process. For this reason, it will not be covered in this paper.

2 Connection detailing issues

On the issue of inadequate detailing, knowledge is usually available in some form but has been discarded to reduce costs or is unknown by the person in charge of the design.

The most common issue of inadequate detailing associated with connection design is one where the occurrence of shrinkage or swelling stresses has not been taken into account. Usually, this is observable in structures that have been constructed using green timber and allowed to reach an equilibrium moisture content that forced a significant amount of shrinkage. This shrinkage, if restricted, will normally result in tension stresses perpendicular-to-grain, forcing checks or splits. These checks or splits can significantly alter the resistance of a fastener if located at a critical location or can be insignificant if located away from the fastener potential failure planes. Figures 1 and 2 show two different situations where splits resulting from shrinkage stresses have occurred.

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In the two situations shown in Figures 1 and 2, the Douglas fir timber in the structure was used green due to the requirement to use un-dried timber in the quick erection of large hall during WWII. In these structures, timber members are connected with 102 mm split rings. Shrinkage splits plague the connections in these structures and they need to be inspected regularly and repaired if necessary. Fortunately, repair techniques have evolved as a result of the advent of the new screws that are now available and the negative effect of splits can be overcome easier. In the figures, threaded rods have been used to limit the lengthening of splits or to reinforce the longitudinal shear strength in the area of the connection.

Nevertheless, it may be sometimes unavoidable to use green timber in a structure and details to prevent shrinkage splits can be included in a design, such as the use of slotted holes in timber or steel plates. Other issues with connections inadequate detailing normally occur as a result of inattention to details during the drawing/detailing phase. Usually, these mistakes can be taken care later. Figures 3 and 4 show two situations where bad details have been allowed to be present well after completion of the project.

Figure 1: Shrinkage split with no effect on connection resistance

Figure 2. Shrinkage split resulting in significant reduction of connection resistance

3 Connection resistance issues

On the issue of connection failures due to inadequate resistance, knowledge available through on-going research has expanded to a point where modes of failures, both ductile and brittle, are better known.

The issue of ductile failures of mechanical fasteners is well known and documented. Johansen theory (1949) is the cornerstone of all design equations related to ductile failures for all dowelled-type fasteners. Its acceptance is world-wide.

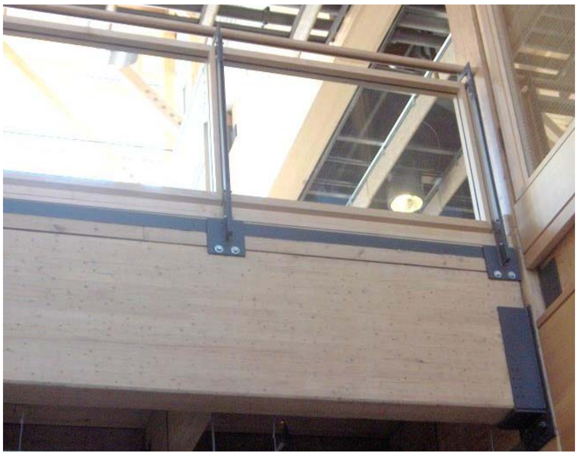

Figure 3: Railing connection detail with lag screws at beam-to-floor interface

Figure 4. Steel knife plate slot extended to bottom of member. The slot in all other members is from the top only for better appearance

Brittle failures associated with failure of the wood surrounding the fastener group, on the other hand, is only starting to be understood within the engineering community even though their occurrence have been observed in many laboratory experiments. Typically, brittle failures are different for connections loaded parallel or perpendicular-to-grain. Figure 5 shows the main situations for bolted connections (Quenneville and Morris, 2008).

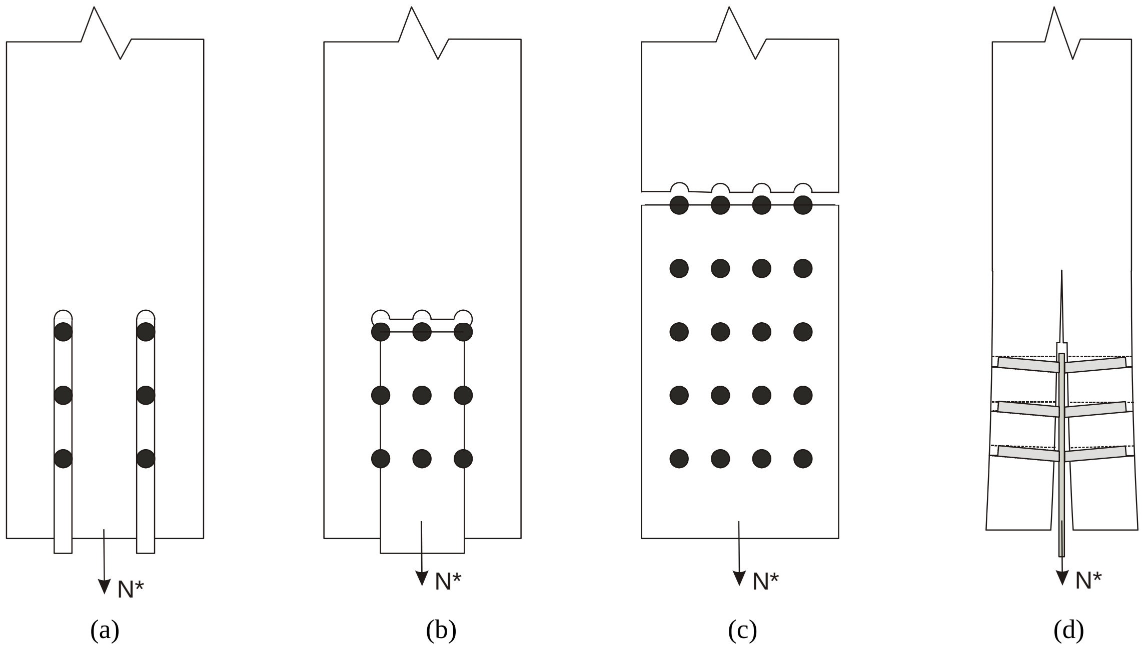

Figure 5: Potential brittle failure modes for dowelled and bolted connections; (a) row shear (b) group tear-out (c) net tension (d) splitting.

Figure 6 shows the potential failures of small-dowelled fasteners that do not protrude through the thickness of the connected member (Zarnani and Quenneville, 2014).

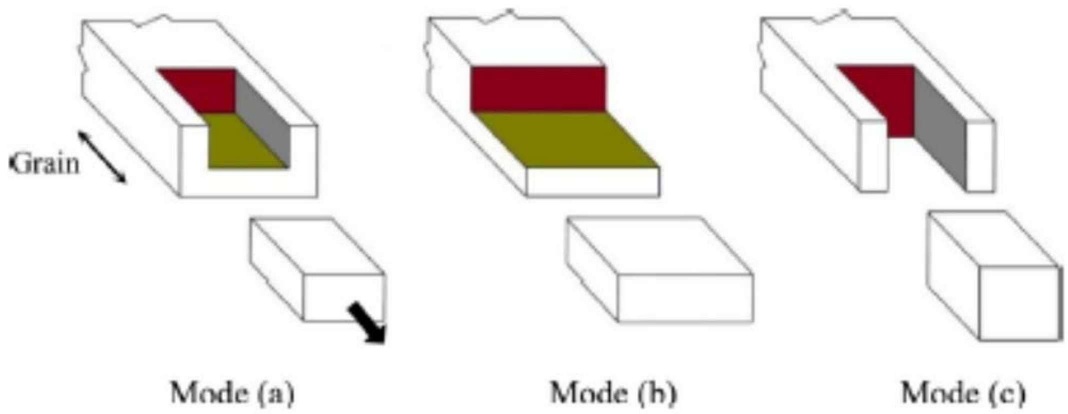

Figure 6: Different potential wood block tear-out failures for small dowel-type fastener groups (Zarnani and Quenneville, 2014).

In all of these cases, the connection resistance is a function of the timber longitudinal shear strength, the tensile strength, the stiffness of the timber surrounding the failure planes and configuration parameters (end distance, spacing, etc…). Design equations that best predict the resistance of these failure modes are unfortunately not simple.

However, in this era of computer-assisted design and calculations, it is believed that design tools that best describe a given behaviour should be made available to provide accurate resistance calculations.

4 Conclusion

A description of potential issues leading to the failure of connection using mechanical fasteners is presented. Some of these issues are within the control of the design team and knowledge to avoid these is well accepted and disseminated. Issues related to failure of timber connections associated with inadequate resistance are not as well known in all parts of the world. Issues and parameters that lead to brittle failures specifically have only been identified lately and design equations are being developed and codified. Work on the prediction of resistance of connections failing due to the resistance of timber in shear or in tension is progressing but there are still significant gaps. Such a gap is the behaviour of mechanical fasteners in timber of higher density and strength.

References

[1] Johansen KW. (1949) Theory of timber connections. Int Assoc Bridge Struc Engr, (**1949**)9, 249-262.

[2] Quenneville, P. Morris H. (2008) Proposal for a mechanics-based bolted connection approach for AS 1720.1. Australian Journal of Structural Engineering, (**9**)3, 1-12.

[3] Zarnani P., Quenneville P. (2014) Improved design procedure for timber connections. Construction and Building Materials, 60 (2014), 81-90.

Experimental evaluation of a dovetail joint

Karel Šobra[1], *Jorge M. Branco*[2]

Summary

An experimental campaign was defined within a STSM of COST action FP1101. This experimental work performed in the University of Minho, Portugal, was divided into two parts: the behaviour of different species under compression was evaluated in the first one, while in the second one, smaller models of dovetail joint were tested. After the experiments aimed to evaluate the behaviour of different wood species in compression, scaled models of the dovetail joint in the scale ratio 1:2 were tested. Overall 17 specimens were tested under monotonic compression and tension. Then, the rotation response (bending moment) of the joints were evaluated under monotonic and cyclic loading.

Key words: the dovetail joint, experimental campaign

1 Introduction

Wood is a material which is traditionally used in constructions for a long time. Even though fires, which significantly reduced number of wood construction in the middle ages, few historical structures with wooden trusses remain in the Czech Republic. One of the most interesting ones is trusses of wide construction as churches, storehouses, etc.

During the lifetime, trusses degrade, they could be mechanically damaged or suffer to neglected maintenance, influence of wood-rotting fungus and wood-destroying insects, etc. Since historical trusses frequently belong to buildings of the Cultural Heritage their reconstruction is usually under supervision of Monument Care Department of Ministry of Culture of the Czech Republic. Therefore all interventions must preserve the originality of the construction. Due to this traditional carpentry, usually all wooden, joints have to be used.

During reconstruction detailed diagnosis and analysis of the existing structures are necessary to support the decision if it is sufficient only to strengthen construction or other more serious intervention is necessary. In this phase, the assessment of the behaviour or carpentry joints is crucial. Joints play an important role in the stress distribution within the structure as they represent the key elements in terms of strength and ductility.

It is fundamental to assess the joints behaviour to detect and repair defect which causes damage in the construction of current joint. It is possible to model all designed intervention using numerical modelling. After evaluation of their impacts to the construction they can be apply to the construction without any doubts of their effectiveness.

In spite of that historical joints are constructed in the same way during the ages, just by routine, there are not many studies, nowadays, focused to historical joints [1-9].

The experimental campaign was established to contribute to this need by presenting an experimental evaluation of one of the most widespread carpentry joints, the dovetail.

2 A dovetail joint

Collar beam truss (Figure 1) is one of the most commonly used types of truss construction in Gothic and Baroque periods. This type of the truss construction is a result of an evolution of trusses used for high roof which were used especially during Gothic period. Construction of the truss for high roofs had to be stiff enough to resist increased wind loads and in the same time light enough. Utilization of collar beam in the truss construction minimizes rafter's span what provides use of smaller cross section of used elements, and in the same time increases stiffness it the cross direction of the truss as a whole.

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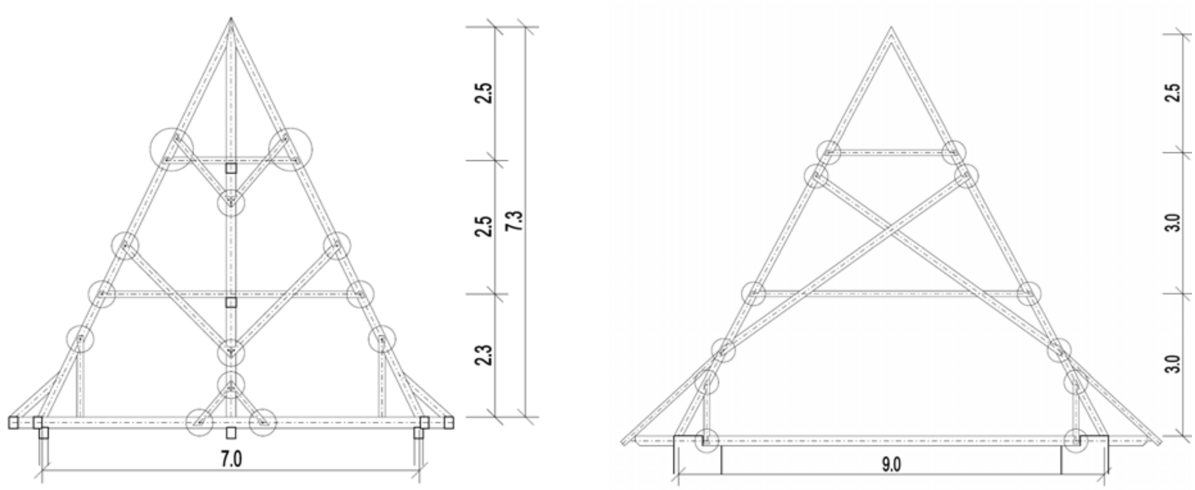

Figure 1: Collar beam truss: chapel in Kozojedy (Jičín, Czech Republic) in the left and in right truss of Church of Saint Anna in Prague (Czech Republic) – the dovetail joints are in circles

The dovetail joint is the most commonly used traditional carpentry joint, which can be found in collar beam trusses. It is complete wooden joint which connects two elements in a general angle. In the joint, both elements are weakened of 1/3 of the width of thinner one to fit together and whole joint is held together by the key.

Forces in the joint are transferred through direct contact of the contact surfaces (further called compressive areas). Even the highly deformed key can still hold whole joint together, however, due to gaps which appear in the joint, compressive areas are smaller and local stresses increase. For different types of loading different compressive areas are involve to the force transfer as can be seen in Figure 2.

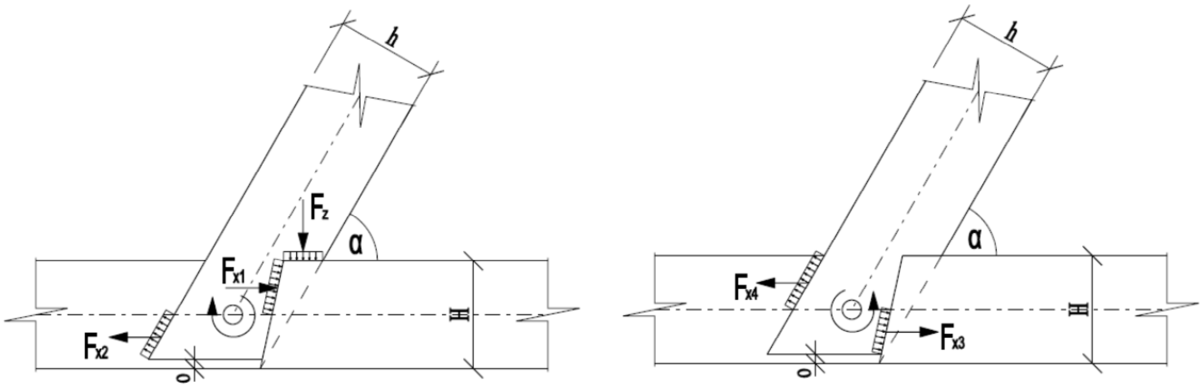

Figure 2: Compressive areas of the joint

3 Experimental tests of scaled joints

During the experiments scaled models made from Scots pine (*Pinus sylvestris*, CV) and from Maritime pine (*Pinus pinaster*, PB) were tested. In all specimens the key was made from tropical wood species Massaranduba (*Manilkara huberi*), since it is very hard, avoiding undesirable shear deformation that could compromise understanding deformations of the connected elements.

For the experimental tests of the scaled joints, a steel frame allowing changes of the joint skew angle (seen in Figure 3) was used to accommodate and fix the specimens during tests. All joints were made with a skew angle $\alpha = 60°$ and were placed over the steel frame in the way, where loaded element is in vertical position. In terms of load procedure, all tests (tension, compression and bending) were made in displacement control.

Figure 3: Steel frame used during the experimental campaign of scaled joints

At the beginning two tests of Scots pine (*Pinus sylvestris*, CV) and two specimens of Maritime pine (*Pinus pinaster*, PB) were made during monotonic compression tests. Deformation of upper surface of skew element due downward motion of vertical element was measured on both sides of skew element and displacement in the key surroundings was measured with LVDT placed on the front surface of skew element as can be seen at Figure 4. Velocity of displacement during the test was 0.015 mm/s.

Using carbon paper was verified that whole assumed compressive area is uniformly used during the monotonic compression and its deformation was significant, what is possible to see at Figure 5a).

At weaker wood species CV small stress damage of the key was observed, on the other hand at PB, which is harder than CV, noticeable shear deformation of the key was observed (see Figure 5b))

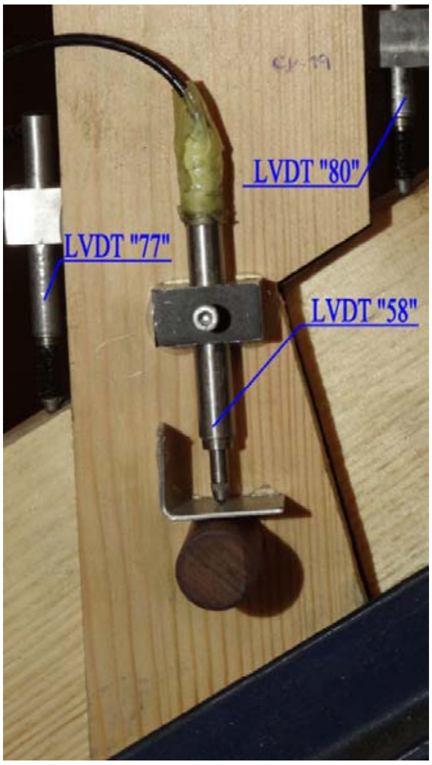

Figure 4: Position of LVDTs during compression monotonic tests

a)

b)

Figure 5: Deformation of elements of the joint observed at compression test of Maritime pine (Pinus pinaster, PB) specimens, a) deformation of skewed element, b) shear deformation of the key

Measured values of F_{max} evaluated for both CV and PB specimens are shown at Table 1. Coefficients of variance till 5.5% show good homogeneity of the results.

Table 1: Elastic stiffness (K_e) and pos-elastic stiffness (K_p) [kNm^{-1}] and maximal force (F_{max}) [kN] obtained from compression tests

	LVDT no.	F_{max}	K_e	K_p
CV	80	24.06	30045	1492
	77	22.64	28968	1572
	Avg.	**23.35**	**29507**	**1532**
	CoV	**4,3%**	**2,6%**	**3,7%**
PB	80	26.66	49792	3950
	77	25.96	53834	3131
	Avg.	**26.31**	**51813**	**3541**
	CoV	**1,9%**	**5,5%**	**16,4%**

In Table 1 F_{max} is the maximal force obtained according European standard EN 408 [10], K_e is the elastic stiffness and K_p is the pos-elastic stiffness, *Avg.* is the average value of column and *CoV* is the coefficient of variation.

One of the aims of the compression experimental test was to verify the assumption that the compressive areas in the same direction suffer the same deformation, what is essential for the theoretical solution of the dovetail joint behaviour [11]. At Figure 6 measured force – displacement relationships for Maritime pine (PB) are shown. Labelling of curves at Figure 6 corresponds with used LVDTs shown at Figure 4. Good match between the lines for the same specimen shows uniform deformation of upper surface of skew element.

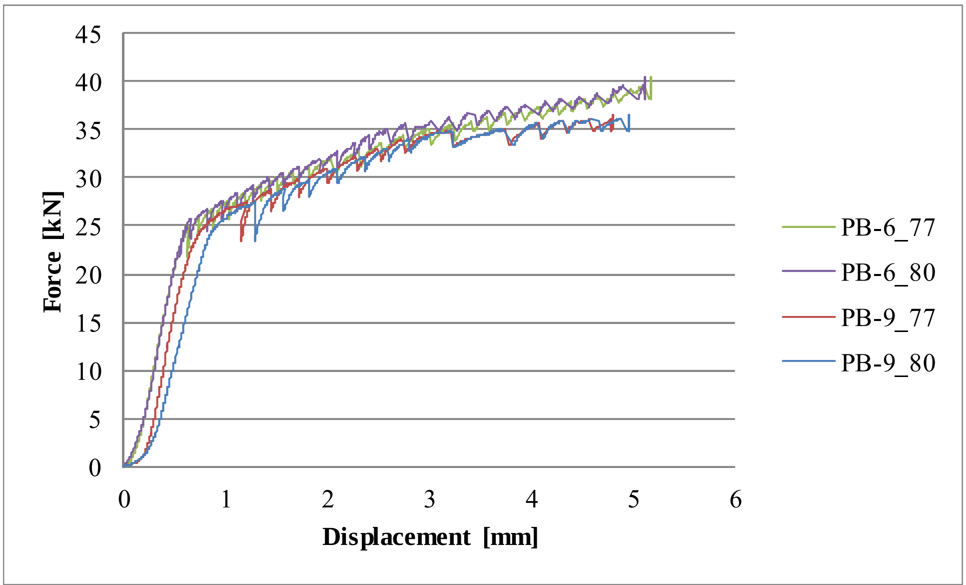

Figure 6: Force – displacement relationship for LVDTs 77 and 80 in the case of PB specimens

Since no significant deformation of the key was shown by the experiments, all the rest of deformation can by divided between front and back side of skew element, on half of the rest for each side. Behaviour show at Figure 7c) and d) can be considered.

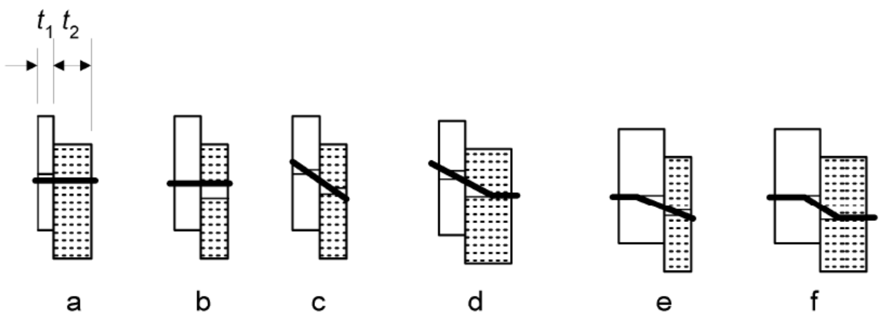

Figure 7: Types of deformation for one shear wooden joint [12]

In Table 2 comparison of measured data with data obtained using theoretical model is show. As it is possible to see at Figure 6, at the beginning of all curves some initial slip (part of curve with small stiffness) can be observed, displacement obtained in this area can be assigned to the initial state of the joint, where all gaps close and joint fitting together. Due to this, it is possible to remove this initial deformation from the force – displacement behaviour and adjusted reached values as well. For PB specimens, 0.12 mm displacement was cut off and for CV specimens, 0.24 mm displacement was removed. Good match of the force at proportional limit and the elastic bearing capacity can be point out.

Table 2: Comparison of measured and calculated data for compression of the joint

	LVDT no.	F_{pp}	R_F	δ_M	δ_T	$\delta_{M,a}$
		[kN]	[kN]	[mm]	[mm]	[mm]
CV	80	24.06	27.26	1.517	0.409	1.277
	77	22.64		1.227	0.206	0.987
PB	80	26.66	29.76	1.041	0.405	0.921
	77	25.96		0.88	0.202	0.76

In Table 2 F_{pp} is the force at proportional limit, R_F is the elastic resistance of the joint from the theoretical model, δ_M is the measured displacement, δ_T is the displacement obtained from theoretical model and $\delta_{M,a}$ is the adjusted displacement obtained from test. For calculation of the displacement and the forces, material characteristics obtained in the first part of the experimental campaign were used.

Bending moment tests

Main objective of bending moment tests was to establish experiments which simulate behaviour during cycling bending moment loading. To set proper values of displacement for cycling tests one CV specimen was tested to positive (counter clockwise direction) bending moment and one CV specimen was tested to negative (clockwise) bending moment. Velocity of the test for both positive and negative bending was 0.03 mm/s.

During both types of the tests, horizontal force which simulated monotonic bending moment was applied perpendicular to the vertical element (Figure 8).

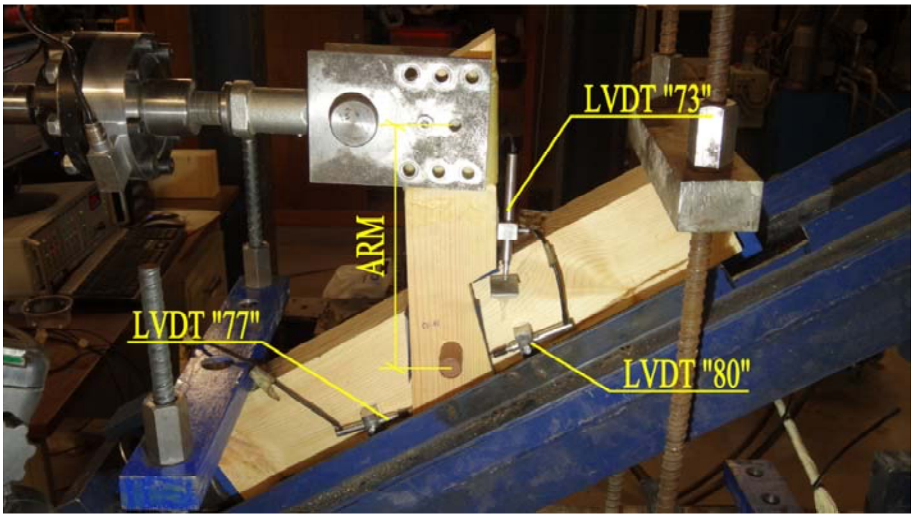

Figure 8: Positive bending moment test arrangement

Table 3 shows values obtained and evaluated from negative monotonic bending moment tests. Values mentioned in Table 3 were used for calculation of the ductility according to equation (1).

Table 3: Values obtained from negative monotonic bending moment test

F_{max}	4,542 kN	M_{max}	0,98 kNm
F_y	4 kN	V_y	1 mm
F_u	4,587 kN	V_u	2,538 mm

In Table 3 F_{max} is evaluated according EN 408, the force F_y and the slip V_y are values defined proportional limit. For evaluation of the yield values of the force F_u and the split V_u , EN 12512 [13] was used.

$$D_{CV-21} = \frac{V_u}{V_y} = \frac{2.538}{1} = 2.538 \qquad (1)$$

Limit values for displacement controlled cycling bending moment tests were established by monotonic moment tests mentioned above. The velocity of all tests was 0.2 mm/s, what is limit velocity according to EN 12512 for those type of tests. The same arrangement of the tests as was shown for monotonic moment tests was used also for cycling moment tests. From three tested specimens (1xCV + 2xPB) two models of the dovetail joint have failed.

Force – displacement relationship for cycling bending test of specimen CV-22 in comparison with behaviour during positive monotonic moment test ("+CV_18") and negative monotonic moment test ("-CV_21") is shown at Figure 9.

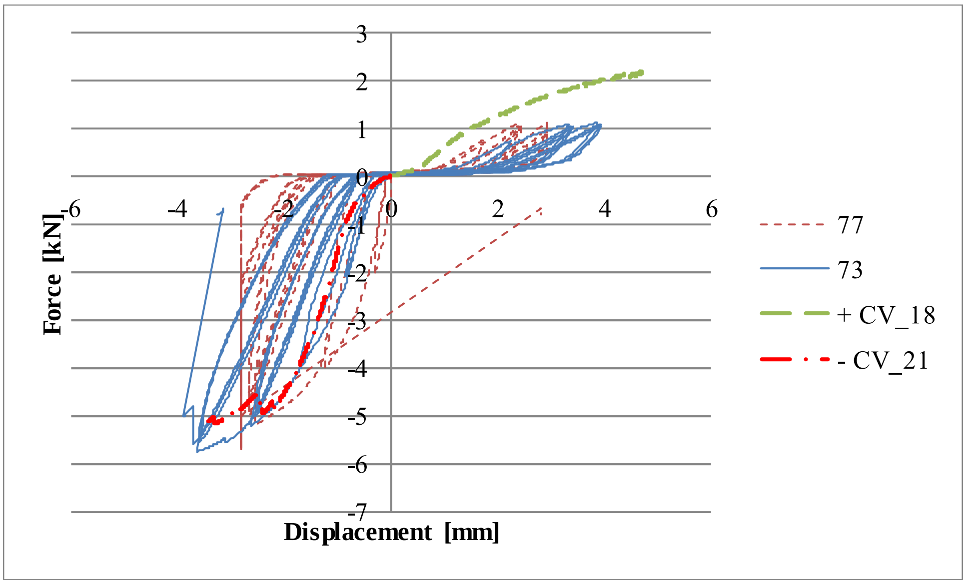

Figure 9: Force – displacement relationship for cycling moment test of specimen CV-22

For all cycling moment tests values of impairment of strength ΔF for all set of three cycles both for tension and compression were evaluated according EN 12512, evaluated values is shown in Table 4.

Table 4: Impairment of strength for CV-22

Compression [kN]		Tension [kN]	
ΔF_{c1}	0.293	ΔF_{t1}	-0.064
ΔF_{c2}	0.33	ΔF_{t2}	-0.058

Values of the dissipation energy E_d and the potential energy E_p corresponding with EN 12512 were evaluated for each set of three cycles. On the basis of dissipation and potential energies and equation (2), the damping factor was calculated, shown in Table 5. The damping factor is determines only for the third cycle from the set.

$$\nu_{eq} = \frac{E_d}{2\pi E_p} \qquad (2)$$

Table 5: Maximal forces and damping factor for CV-22

Cycle no.	Compression			Tension		
	F_{max} [kN]	M_{max} [kNm]	ν_{eq}	F_{max} [kN]	M_{max} [kNm]	ν_{eq}
1	2.071	0.46		-0.219	-0.05	
2	4.005	0.88		-0.742	-0.16	
3	5.206	1.15	2.9	-1.089	-0.24	3.8
4	5.75	1.27	3.4	-1.116	-0.25	3.8

The same evaluation, which was mentioned above for specimen CV-22, was used for specimens PB-11 and PB-111, results of the evaluation will be shown in the thesis [14].

4 Scaled models experiments conclusion

Few conclusions can be noticed from scaled models experimental campaign. The first and the most important is the confirmation of the theoretical behaviour assumed for the dovetail joint [11]. Sizes of some compressive areas, especially the one used for force transferring during influence of negative (clockwise) bending moment, have to be adjusted in theoretical model on the basis of made tests. This required adjustment will also influences the length of the arms assumed in the calculations as well.

Even through significant imbalance of properties in positive bending moment and negative bending moment, joint can transfer increasing loading with no sudden failure immediately after run off of elastic bearing capacity. This behaviour is proved at Figure 9 where joint can transfer higher loading in each cycle. This type of behaviour is suitable especially in terms of safety of the construction.

5 Acknowledgement

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References

[1] J. Branco, P. Cruz, M. Piazza, and H. Varum, "Experimental analysis of original and strengthened traditional timber connections," Portland, OR, 2006, pp. 1314-1321.

[2] P. Fajman. (2013, Force distribution in a splice skew joint due to the bending moment. Civil Engineering Journal 2013(04).

[3] P. Fajman, "A scarf joint for reconstructions of historical structures," Advanced Material Research of Trans Tech Publications, 2014.

[4] J. Jasienko, L. J. Engel, and P. Rapp, "Study of strains and stresses in historical carpentry joints," in Structural Analysis of Historical Constructions, New Delhi, 2006, pp. 375-384.

[5] M. A. Parisi and C. Cordié, "Mechanical behavior of double-step timber joints," Construction and Building Materials, vol. 24, pp. 1364-1371, 2010.

[6] M. A. Parisi and M. Piazza, "Mechanics of Plain and Retrofitted Traditional Timber Connections," Journal of Structural Engineering, vol. 126, p. 1395, 2000.

[7] R. H. Sangree and B. W. Schafer, "Experimental and numerical analysis of a stop-splayed traditional timber scarf joint with key," Construction and Building Materials, vol. 23, pp. 376-385, 2009.

[8] R. H. Sangree and B. W. Schafer, "Experimental and numerical analysis of a halved and tabled traditional timber scarf joint," Construction and Building Materials, vol. 23, pp. 615-624, 2009.

[9] J. R. Villar, M. Guaita, P. Vidal, and F. Arriaga, "Analysis of the Stress State at the Cogging Joint in Timber Structures," Biosystems Engineering, vol. 96, pp. 79-90, 2007.

[10] European Standard EN 408:2010 - Timber structures – Structural timber and glued laminated timber – Determination of some physical and mechanical properties, 2010.

[11] K. Šobra, J. Branco, and P. Fajman, "Force analyses of dovetail joint," presented at the 5th Conference Nano & Macro Mechanics, Prague, 2014.

[12] Standard in the Czech version of the European Standard EN 1995-1-1:2004 including its Corrigendum EN 1995-1-1:2004/AC:2006-06.

[13] European Standard EN 12512:2001 (E) - Timber structures - Test methods - Cyclic testing of joints made with mechanical fasteners, 2001.

[14] K. Šobra, "Analysis of selected historical carpentry joints," Doctoral, Department of Mechanics, Czech Technical University in Prague, Prague, 2015.

Repair of cracks and delaminations in glued laminated timber

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Summary

The repair of cracks and delaminations in glued laminated timber is experiencing an increasing demand. Cracks in the wood and partial delaminations of glue lines can significantly reduce the load bearing capacity and thus must be repaired when danger of failure arises. Currently, there are several two-component adhesives available on the European market that are suitable for such restoration activities. The present article provides an overview of suitable adhesive products, of the current status in European and national normative regulations and finally proposes step-by-step instructions for secure and sustainable repair actions in glued laminated timber elements.

Key words: Repair, Crack, Delamination, Two-component adhesive, Guideline

1 General - Need to repair cracks

Laminated timber beams have to be repaired if cracks in the wood or in the glue lines give rise to the danger that the load bearing capacity of the component could be significantly reduced. The purpose of such a repair campaign must, in the first instance, achieve sustainable restoration of the shear and/or transverse tensile strength of the component, and also the fastener embedment strength. Visual appearance may be another reason for carrying out such repairs.

The repair of cracks by injection/filling with adhesives is feasible where the crack openings are no greater than 10 mm. and the cracked surface exhibits only low to medium fibrosity/splintering.

2 Fundamental principles and technology

Europe: Available adhesives, and regulations in force

The repair of delaminated timber components is usually carried out with two-component adhesive systems. At the European level, there are currently no standards or generally applicable specifications covering the field of timber repairs or advising on the kinds of two-component adhesive systems that might be suitable.

Eurocode 5, Section 3.6 (EN 1995-1-1:2009) [1], referring to the use of adhesives in load-bearing timber components in Europe, states that *"adhesives for structural purposes shall produce joints of such strength and durability that the integrity of the bond is maintained in the assigned service class throughout the expected life of the structure."* There are also various European standards that specify the requirements on adhesives in load-bearing timber structures together with compulsory tests and inspection processes designed to facilitate general quality assessment. However, these standards relate exclusively to 1C-Polyurethane (PUR), phenolic, aminoplastic or Emulsion Polymer Isocyanate (EPI) adhesives. Test and inspection processes described in these standards can be applied to two-component (2C) adhesive systems, but there is still no general regulation specifying a uniform procedure for test and inspection, and thus in the approval, of 2C adhesive systems in Europe.

The fact is, however, that there is significant demand in Europe for the performance of repairs on load-bearing timber elements, and particularly laminated timber components. In practice, this takes place on the basis of national regulations which – depending on the country – define variously stringent requirements and processes.

Switzerland

In Switzerland, reference is made to both Eurocode 5 (EN 1995-1-1:2009) [1] and, in particular, the Swiss timber construction standard SIA 256 [2] as the basis for the gluing of load-bearing structural timber elements. The formulations appearing in both standards are very general in nature, allowing extensive scope in the application of adhesives for repair purposes. Selection of the appropriate adhesive system is possible by estimation of the engineer responsible. The suitability of the repair process and also the gluability of the component need to be verified on the basis of procedural trials. These in turn have to be aligned as far as possible to the actual service conditions that will subsequently prevail (SIA 265:2012). In Switzerland, therefore, there is wide scope in the selection of the adhesives to be used and the most appropriate repair processes. Not least for this reason, the conditions for the development of innovative adhesive systems are very good in Switzerland.

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Germany

Companies that carry out repairs on laminated timber components in Germany must have a certificate of competence for the adhesive bonding of load-bearing structural timber components – the so-called "Leimgenehmigung" or "Glulam Permit" in accordance with DIN 1052:2012-10, Section 5 [3]. Additional approval for the repair of load-bearing timber components and of laminated timber through the use of adhesives must also be entered in the Glulam Permit.

Glulam Permits are issued by the Institute for Material Testing known as MPA Stuttgart ("Materialprüfanstalt Stuttgart – Otto-Graf-Institut") on behalf of the German Institute for Building Technology (DIBt).

The work must be carried out by appropriately qualified skilled technicians with proven experience with the manufacture of laminated timber. Throughout the repair campaign, at least one employee should be on site who has taken part in the repairs training course provided by MPA Stuttgart and the Engineered wood association "Studiengemeinschaft Holzleimbau e.V," or a comparable training course (Code of Practice: Studiengemeinschaft Holzleimbau e.V 2010) [4].

In Germany, the adhesives permitted for use in the repair of laminated timber are those that meet the requirements of DIN EN 301:2013 [5].and DIN 68141:1995 [6]., Section 3.1.3 and Appendix 3.6. Adhesives which carry general building authority approval issued by the German Institute for Building Technology (DIBt) may also be used.

Austria

In Austria, the regulations contained in Eurocode 5 [1] apply to the use of adhesives in timber construction.

A formulation to be incorporated within the national application standard ÖNORM EN 1995-1-1:2014, Chapter 10.3 is planned for the future, and this could read something like this:

"For the execution of adhesive bonding work on load-bearing components and glued connections, those persons performing the work will be required to have a corresponding certificate of competence and continuous trainings. During planning and execution of adhesive bonding work the provisions of the adhesive specification (e.g. glue line thickness, temperature) must be regarded."

Table 1: Selection of two-component adhesive systems that may be used in Europe and in Switzerland for repair applications (with the most important technical data according to the respective technical data sheets or test reports)

Adhesive System	Viscosity (mPa s at 20 °C)	Pot time / curing time/ final strength (at 20°C)	Max. glue line thicknesses	Origin
Astorid GSA resin (neue Holzbau AG, Lungern)	25,000 (at 25 °C)	80-100min/ 24 h/ no data (at 23°C)	4 mm	Switzerland
Jowat two-component construction adhesive 692.30	36,500	30min/ 2h/ >24h	3 mm	Germany
Rotafix Structural Adhesive 400cc	No data	6h/ 48h/ no data	12 mm	UK
Spattling compound PURBOND RE 3040	Pasty	10min/ after 30min sufficient strength for capping or injection with PURBOND RE 3064	4 mm Practical experience up to 4 cm	Switzerland
With filling compound PURBOND RE 3064	9,000	10min/ 2h/ >24 h	4 mm Practical experience up to 4 cm	Switzerland
WEVO EP20 with hardener B20	No data	80min/ 6h/ 16h	4 mm (8 mm with limitations)	Germany (with General building authority approval Z-9.1-750)

*Figure 1**:** Test specimens manufactured as part of the Project "Assessment and Reinforcement of Laminated Timber Components", prepared using three different two-component adhesive systems and various adhesive injection methods. Smaller defects usually restricted to the immediate environment of the injection nozzles. In tests, all three systems produced similar strength values. (The project was financed by Swiss Federal Environmental Agency (BAFU))*

3 Application

Crack repair methodology

The most important criterion for a successful crack repair using adhesive injection is the achievement of an integral bond between the timber and the adhesive. In addition to ensuring the general suitability of the adhesive, suitable processing conditions must also be selected. In particular, it is important to ensure that the surfaces being bonded are of the required quality, and that the right technique is applied; defects, such as air pockets shall be avoided. The adhesive technology applied should be regarded as a single, integrated system, with planning and execution entrusted to experts or at least carried out under their supervision.

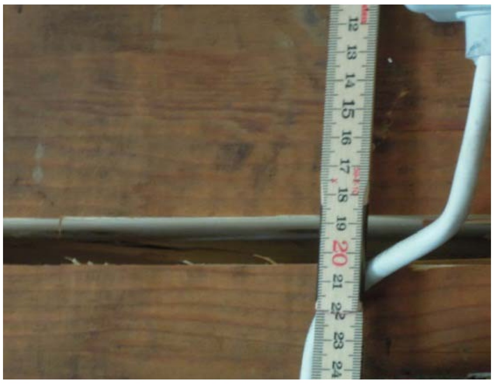
As in the case of other reinforcement methods, the barrier effect of the repaired (thick) bonded joints against the ingress of water and water vapor must also be considered during planning. Large moisture fluctuations, giving rise to significant changes in shrinkage and swelling behavior, may result in an increased risk of further delamination in the areas concerned.

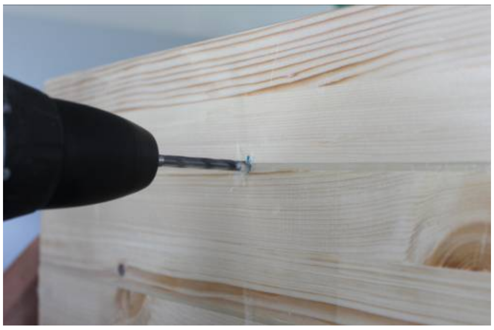
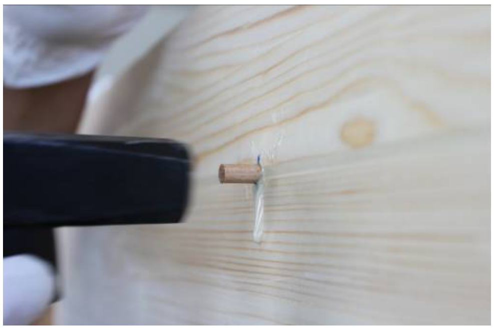
The repair of cracks in laminated timber beams requires a comprehensive, planned approach that is then executed with conscientious application. The adhesive bonds must be made with the greatest care. Possible gluing defects are very difficult to identify once the work has been completed and may have serious consequences.

Prior to commencement of the repair work, a thorough assessment of the damage needs to be made by an expert in the field who, in consultation with the repair engineer responsible, should then determine the feasibility of the repair approach and prepare an appropriate repair system concept. This concept should contain information relating to the following points (as per the Code of Practice "Studiengemeinschaft Holzleimbau e.V. 2010") [4]:

- Geometry of the building, stress patterns transmitted to and absorbed by the load-bearing timber components
- Extent of damage, with particular attention being paid to crack records
- Information relating to the areas requiring repair, with determination of the processes and materials to be employed
- As appropriate, information relating to foundation/soil loadability (ability to support accessibility aids, elevators etc., and stresses arising from adjustable supports installed in order to press the components back into their original shape).

Step-by-step instructions on the repair of cracks, as prescribed by, for example, the Code of Practice “Studiengemeinschaft Holzleimbau e.V. 2010,” Radovic et al 1992, MPA Stuttgart 2011 [4, 7, 8]:

1. Preparation • Assessment of the crack(s) • Measurement of wood moisture content • Decision whether the crack can be repaired in one step or in staged sections	
2. Machining of cracks to generate a clean bonding surface, if necessary • Use a manual circular saw or manual router • Cutting down to the bottom of the crack if possible • Slot width to at least the maximum crack width	
3. Cleaning of the cracks/joints • Blowing out the cracks/joint (note: make sure to work only with de-oiled and dried compressed air!)	
4. Underside support/bracing/clamping of the element to be reconstructed • Achievement of the greatest possible closure of the crack. Then slight release (approx. 2 mm)	
5. Sealing, capping or filling of the cracks/joints with... *Adhesive tape* • Use transparent adhesive tape in order to better observe the material flow • Additionally secure the adhesive tape at the sides by high-strength fixing tape *Spattling compound (e.g. PURBOND RE 3040)* • Spattling down to a depth approx. 5mm inside the crack • Additional adhesive/masking tape along the edges of the crack reduces the need for re-work	

6. Drilling of filling- and ventilation-holes • I Intervals of 10-30 cm, dependent on the crack width • Approx. 2-3 cm deep: diameter corresponding to the size of the injection nozzle • Alternative: drilling diagonally from above	
7. Injection with suitable adhesive • Preparation of a reference sample of the adhesive for filing/retention • Carry out the injection work in sections from one drilled hole to the next one, step by step • In the case of cracks that do not run horizontally, always work from bottom to top • Wait in each case for adhesive to emerge through the next drilled hole • Example of an injection system using angled drilled holes (sealing with adhesive strips and reinforced with strips of plywood)	 
8. Closing off the drilled holes • Taping of boreholes with suitable closure devices, e.g. with smooth wooden dowels	
9. Checking the quality of the adhesive bond • After full curing of the adhesive, extraction of drill-cores at suitable locations and block shear-testing according to EN 392 [9]	

10. Cleaning and surface rework where necessary	
11. Keeping records • Noting of the type of damage and accurate description the cracking encountered (number of cracks, dimensions, position, surface condition of the cracks – such as whether smooth, fibrous, clean, dirty) • Noting of the interior climate (temperature, rel. humidity) • Noting of the temperature and moisture content of the timber components in the immediate vicinity of the crack • Noting of the mixing process • Noting of the adhesive quantities and mixing ratios • Noting of the start of adhesive injection and the end of the process • Noting of the quantities used for each individual crack or crack group • Noting of any deviations from the planned procedure and other significant occurrences • Preparation reference adhesive samples for retention • Retaining records for 20 years.	

References

[1] EN 1995-1-1:2009: Eurocode 5: Bemessung und Konstruktion von Holzbauten – Teil 1-1: Allgemeines — Allgemeine Regeln und Regeln für den Hochbau

[2] SIA 265:2012: Holzbau

[3] DIN 1052-10:2012: Herstellung und Ausführung von Holzbauwerken – Teil 10: Ergänzende Bestimmungen

[4] nach Merkblatt „Sanierung von BS-Holzbauteilen" der Studiengemeinschaft Holzleimbau e.V., April 2010

[5] DIN EN 301:2006-09, Klebstoffe für tragende Holzbauteile, Phenoplaste und Aminoplaste – Klassifizierung und Leistungsanforderungen

[6] DIN 68141:1995-08: Holzklebstoffe – Prüfung der Gebrauchseigenschaften von Klebstoffen für tragende Holzbauteile

[7] Radović, B.; Goth, H. 1992: Entwicklung und Stand eines Verfahrens zur Sanierung von Fugen in Brettschichtholz. In: Bauen mit Holz, Heft 9/1992, Bruderverlag, Karlsruhe

[8] Unterlagen zum Sanierungslehrgang der MPA Stuttgart, März 2011

[9] DIN EN 392:1996-04, Brettschichtholz – Scherprüfung der Leimfugen

X-Ray technology for the assessment of timber structures

Steffen Franke[1], Bettina Franke[2]

Summary

For the assessment of timber structures, which is a permanent task to evaluate the normal function, non-destructive testing methods are preferred but the value of the information is limited due to the performance of the applied method. X-Ray is a non-destructive technology which allows a view into the structural member, where especially internal damages like cracks, holes or plastic deformations of mechanical fasteners can be detected. The mobile X-Ray technology is a non-destructive testing method with a good accuracy for detailed information. The digital analysis of the radiograms allow in-situ measurements. The method and its possibilities for non-destructive testing of timber structures are presented. The presentation based on practical examples gives an overview of the ability and the limits of this method. It shows that the mobile X-Ray technology offers a high potential for an effective assessment.

Key words: Timber, X-ray, Non-destructive testing, Connection

1 Introduction

The structural assessment of timber structures is caused by different reasons, such as regular inspections, structural modifications, changes in serviceability or historic preservation. The assessment of timber structures should always begin with a visual inspection of the complete building for the analyses of the supporting structure. The following assessment of the single members, connections or specific details will take place only after this step. An advantage in assessing timber structures is that abnormalities are normally relatively easy to detect due to discoloration, cracks or plastic deformations. Especially in combination with the measurement of the moisture content, first specification can already be done. Depending on the abnormalities found, specific testing methods are available and can be used. The test methods can generally be classified into nondestructive, semi-destructive and destructive test methods. For the general survey of the building and its detailed assessment, an overview of common methods is given in Table 1. Further explanation can be found in e.g. Aicher [1], Görlacher [2], Kasal & Tannert [3], Köhler et al. [4], Rinn [5], Steiger [6] and Vogel et al. [7].

Non-destructive testing methods are preferred, but the value of the information is limited due to the performance of the applied assessment method. Especially the occurrence of internal damages like cracks, holes, fitting inaccuracy or plastic deformations of mechanical fasteners cannot be detected reliably with these common methods. However the X-ray technology allows a view to the inside of a structural member or a connection. The application of the X-ray technology on wooden structures was investigated and the results and limitations are presented in this paper.

Table 1: Common assessment methods for timber structures

Nondestructive testing methods	Semi-destructive testing methods	Destructive testing methods
Visual inspection	Resistance drilling	Test of glue line quality
Survey	Penetration resistance tests	Mechanical testing for strength prediction
Moisture content	Withdrawal resistance test	
Crack detection and mapping	Drill core specimens	
Ultrasonic wave or echo method	Endoscopy	
Chemical investigations		
X-Ray		

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2 X-Ray Technology

2.1 Method

Since the discovery of X-rays in the late 19th century, this technology has been used for medical imaging which is its most famous application. Later, other applications were added for this technology e.g. in materials science and technology. Since mobile X-ray systems are available, this technology has also been used for the in-situ assessment of structures, as shown in Vogel [8], Brashaw et al. [9], Wei et al. [10], Pease et al. [11], Kruglowa [12], Lechner [13]. The adoption of this technology provides the possibility to look inside the member with a high resolution according to the measuring area of 30 by 40 cm for the film used. The X-ray technology is a non-destructive testing method and works quasi contactless. The use of a mobile X-ray technology in combination with the specific digital scanner allows in-situ assessment of existing structures.

The safety requirements for the use of the mobile X-ray system do not restrict the practical use on existing timber structures. The mobile X-ray system used works with hard X-ray pulse generator but with a very low dose as against stationary X-ray systems known. Furthermore the exposure transmitter is only active, meaning X-rays are only generated, while "taking" the picture. This process takes only a few seconds and before and after no X-ray exposure happens. In practical use, the safety zone is specified as follows: 3 meters around the transmitter, 30 meters in measuring direction and 11 meters perpendicular to it. The users carry a personal dosimeter to register any irradiation.

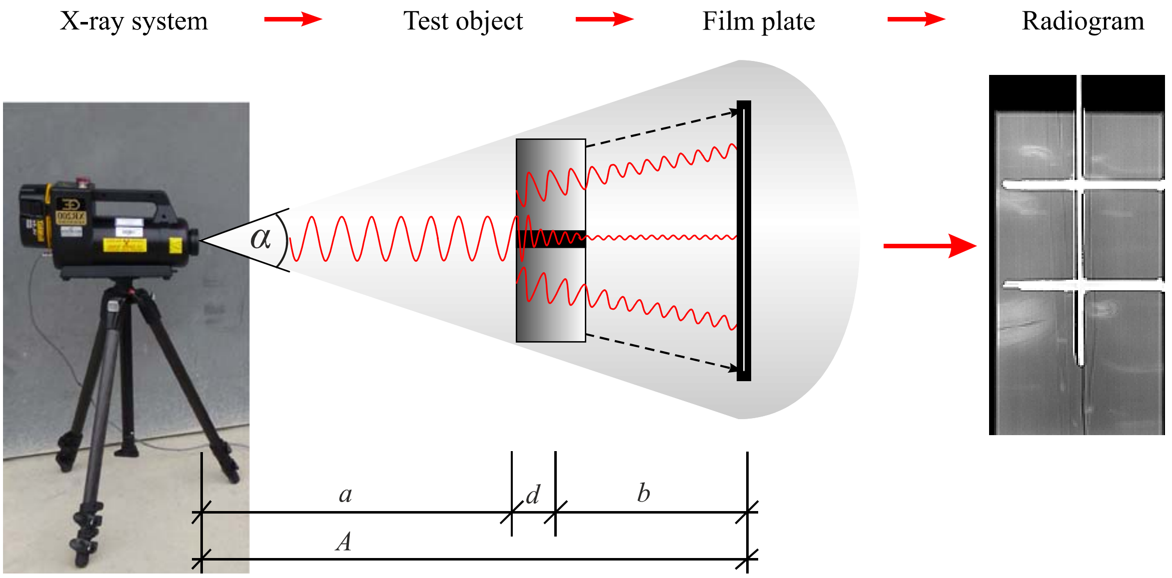


Figure 1: Principle process of X-ray technology and investigations

2.2 Theory and calculation

X-rays are short-wavelength electromagnetic radiation, compare Figure 2, generated by high-voltage electron processes. The X-rays are absorbed depending on the material respectively their density. The X-ray absorption parameter is defined by the Beer-Lambert law as follows:

$$I = I_0 \cdot e^{-\mu d} \tag{1}$$

Where I is the intensity after radiography in [W/m^2], I_0 the intensity before radiography in [W/m^2], d the thickness in [m] of the material and μ the X-ray absorption coefficient in [m^{-1}]. For wood, the X-ray absorption coefficient is defined as follows:

$$\mu = \mu' \cdot \rho \tag{2}$$

With μ' as the mass absorption coefficient in [m^2/kg] and ρ the density of the material in [kg/m^3]. The absorption capacity depends on the density of the material, the atomic mass, atomic number and the depth of the material, Purschke [15].

The X-ray radiography depends on the pulse intensity of the X-rays, the distance of the test object to the transmitter as well as to the film plate and also the thickness of the material. The principle of the process is shown in Figure 1, where the test object is located between the X-ray transmitter and the film plate. The X-rays transmitted travel through the test object and will be absorbed with different intensities before they hit the film plate. The material specific absorption of the X-rays leads to the so called radiogram which will finally be transferred into a grayscale picture. The volume of the three dimensional test object will be reproduced as a two dimensional picture.

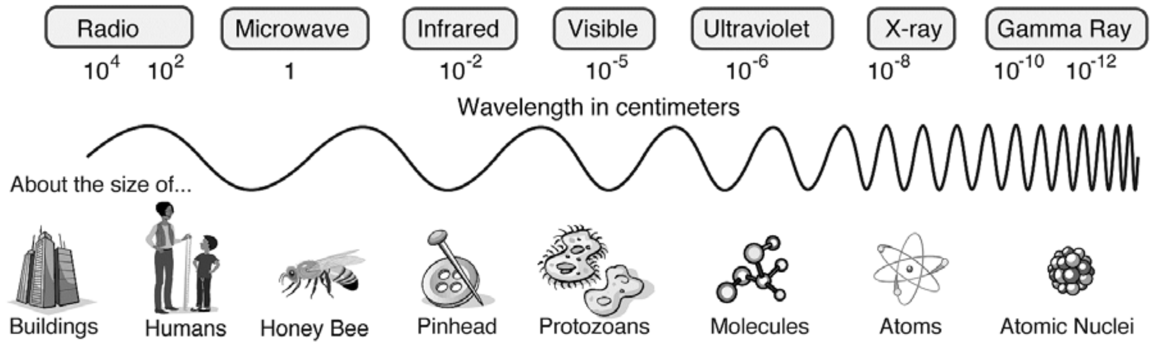


Figure 2: Electromagnetic spectrum, [14]

3 Applications and limits of the mobile X-ray technology

3.1 Laboratory analyses of the system

The principle of the X-ray process is similar to taking a picture with a photo camera. While the quality of a photo depends on the depth of field, sharpness of movement and focus, these parameters are not comparable for X-ray systems. For X-ray photography, the pulse intensity, the distances of the test specimen between the transmitter and the film plate and the thickness respectively density of the object influence or restrict the results, resolution and accuracy of the method. Some of these parameters were investigated and are described following. The X-ray unit XR 200 with a maximum photon energy of 150 KVP and X-ray dose per pulse of 0.026 - 0.040 mSv was used for the investigations.

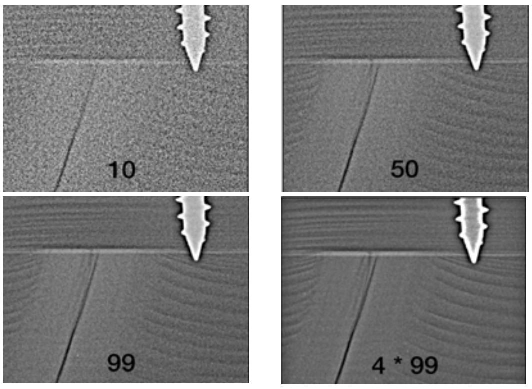


Figure 3: Radiograms with different number of pulses

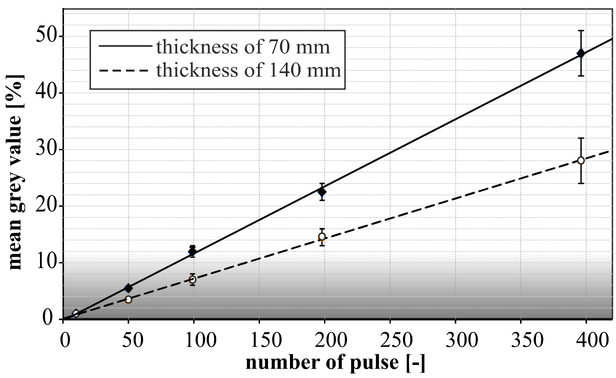


Figure 4: Effect of number of pulses, the shadowed area marks the not useful configurations for the used wood

According to Eq. (1), the intensity I on the film plate increases linearly with the intensity of the transmitter I_0. The gray value of one pixel is proportional to the intensity and will increase respectively. Figure 3 shows radiograms taken with different numbers of pulses from a steel screw tip inserted in a wood block as test object. The test object made of European Spruce with a density of 480 kg/m^3 and about 15 % moisture content had a constant thickness d of 70 mm. For a larger thickness of an identical material the absorption of the X-rays increases, so that for radiograms with comparable quality, the number of pulses has to be increased as well. The effect of the number of pulses was analyzed for two different thicknesses of the wooden block and is shown in Figure 4.

For the optimization of the test set-up, the influence of the distance a between the transmitter and the test object and the distance b between the test object and the film plate (Figure 1) was verified regarding accuracy and sharpness of the radiograms. The same test object with a steel screw inserted in the wood block, as mentioned before, with a thickness of 70 mm was used. An increase of the distance b (with constant A) results in the projection of a smaller area where the object is represented enlarged but with less sharpness and more noise, as shown in Figure 5. On the other hand, the reduction of distance a leads to a clear “burned” spot and unusable radiograms. A minimum distance a of about 1 meter was necessary for the test configuration with a film plate of 30 by 40 cm. The relation between the two distances a and b is summarized in Figure 6.

Finally the thickness of the test object was verified from 70 mm up to 350 mm. Hereby the number of pulses with 2*99, distances A with 2.0 m and b with 0.0 m were kept constant. The radiograms of the test object with the metal screw inserted in the wood block with different thicknesses are shown in Figure 7. The contrast of the radiogram decreases with the increase of the thickness of the specimen. Typical timber structural elements with

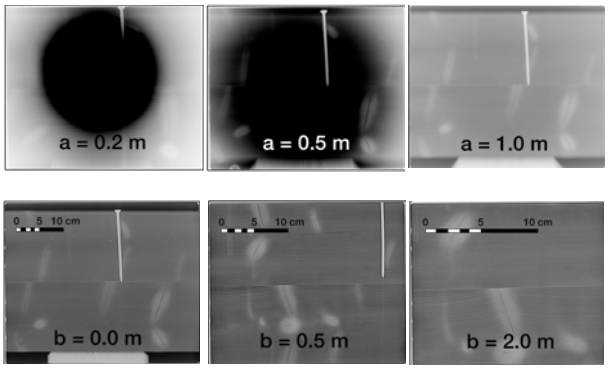


Figure 5: Radiograms with different distances, top row distance a and bottom row distance b

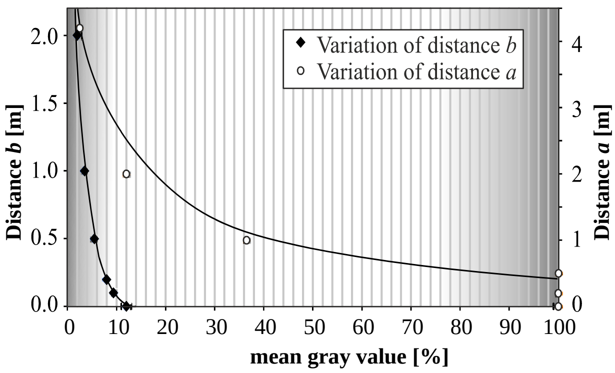


Figure 6: Effect of distances a and b, the shadowed area marks the not useful configurations

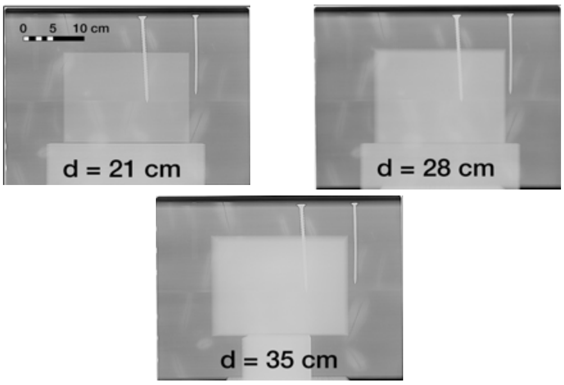


Figure 7: Radiograms with different thicknesses of the test object

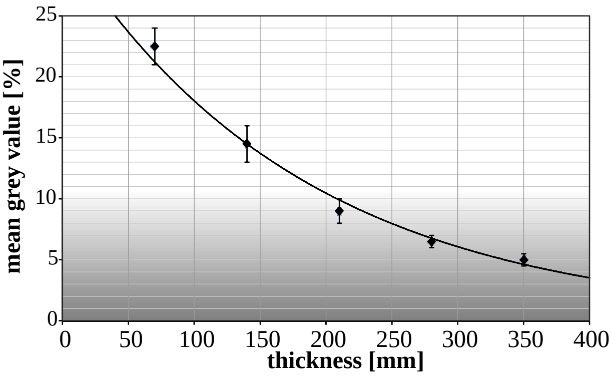


Figure 8: Radiograms with different thicknesses of the test object

thicknesses up to 200 mm can be assessed with the used system. For greater thicknesses the contrast vanishes and only objects with distinctly different densities, e.g. parts of steel embedded in wood are visible in the radiogram. The relation between the mean gray value and the thickness is shown in Figure 8 as summary.

3.2 Assessment of wood and connections

The first investigations were done in the laboratory with samples of historical wood to wood connections as well as with mechanical connections. Figure 9 shows a wood to wood connection with an internal hardwood dowel. Not only the two wood species, European spruce and beech, can be clearly distinguished but also characteristic features within one material like knots and annual grow rings are clearly visible. Furthermore the fitting accuracy of such a connection can be checked. In this case gaps are clearly detectable As a practical application, a historical timber construction in a castle was investigated. Wooden nails could be detected during the assessment of a multi layered beam construction, as shown in Figure 10.

The assessment of timber connections with mechanical fasteners is shown in Figure 11 for a dowelled connection with inner steel plate. The test specimen shown was loaded/unlodaded in certain steps at the laboratory and X-rayed after each load step. For every case, the visual inspection of the outside area (heads of the fasteners) did not indicate any irregularities. But the radiograms show that inside the connection plastic deformations according to the Johansen theory, Johansen [16], already occurred, as shown in Figure 11c)-d). The visible plastic deformations of the fasteners indicate an overloading and a failure of the connection which could not have been observed from outside. Furthermore, load bearing reinforcements have been investigated and checked for the presence of deformations of the self tapping screws,as shown in Figure 12. The self tapping screws are clearly visible. Not all are straight and perpendicular to the surface, but no further deformation (buckling) can be seen. However at this bearing area holes can be seen which shows, that screws have been inserted and withdrawn at other position they appear now.

3.3 Assessment of repaired glue lines and fungal/insect decay

Glulam is a common used engineered wood product for large span timber constructions. The assessment of these structures is a permanent task in order to ensure the integrity and performance. In some cases, glue lines or cracks have to be repaired or supports and high stressed areas have to be reinf orced. The assessment of restructured glue lines was therefore investigated with the mobile X-ray system within a research project. A glulam member with two repaired glue lines was X-rayed in different directions to check the quality of the restoration. In the first radiogram, taken in a direction perpendicular to the glue-line plane and shown in Figure 13b), a clear void at the outside of the beam can be seen. But the allocation to one of the glue-lines or even the evaluation if there are more failures in the same direction is not possible. Figure 13c) shows the radiogram of the same section but taken under an angle of about 45° to the glue-line plane. Here, the two repaired glue-lines can be distinguished from each other and the failure spots and injection holes are clearly visible for each glue-line. In this case, both glue-lines show the voids at the same position. Furthermore, the assessment of this member also shows further small voids and bubbles along the glue-line plane. Such images can help the engineer to determine the quality of a repaired glue line and on base of this to assess its load bearing capacity. In another practical application, also voids and bubbles within a glued-in rod connection could be detected.

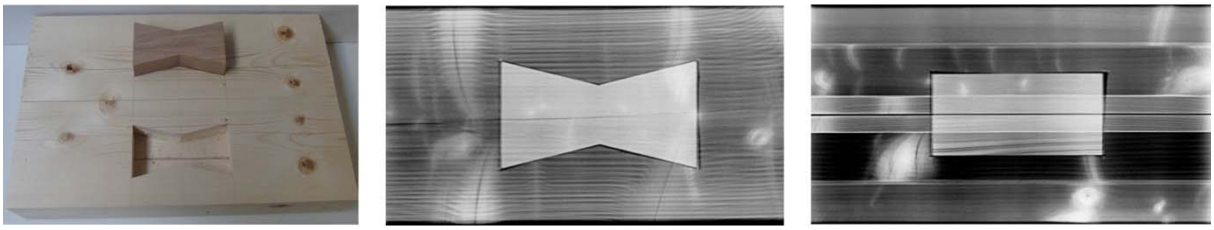


Figure 9: Wood to wood connection with a hardwood (beech) dowel embedded in softwood (spruce) members

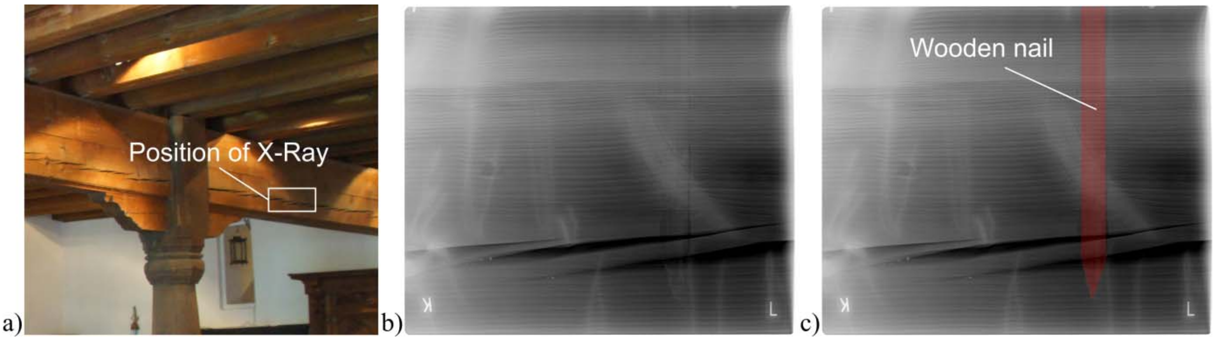


Figure 10: Wood nail in a historic multi layered wooden member, a) position of the measurement, b) original radiogram, c) wooden nail marked in radiogram

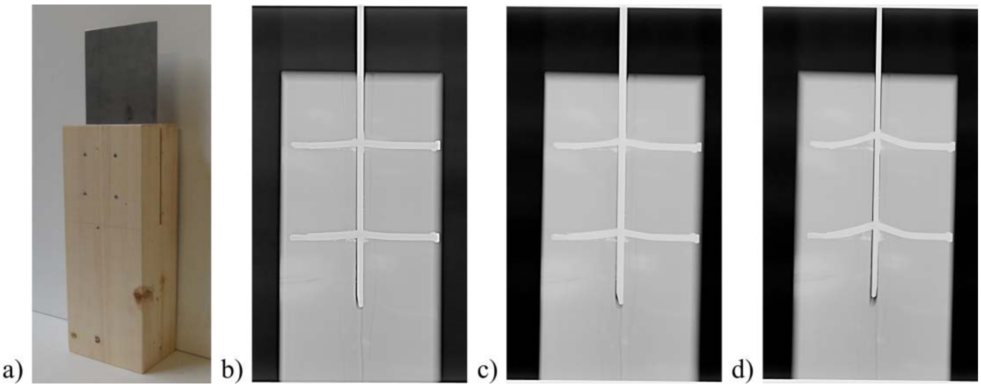


Figure 11: Connection with mechanical fasteners, a) Test specimen, b) unloaded connection, c) and d) connection with plastic deformations

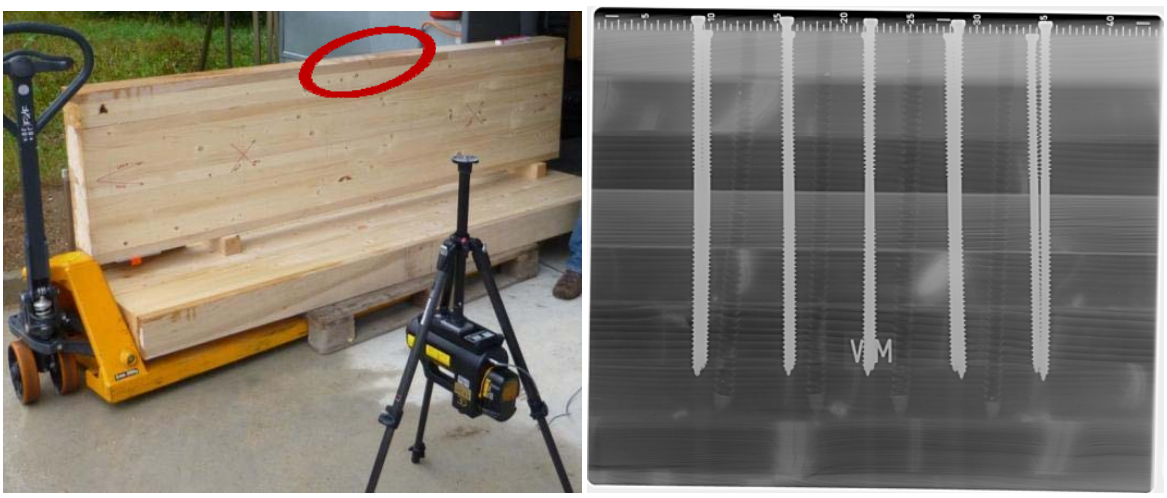
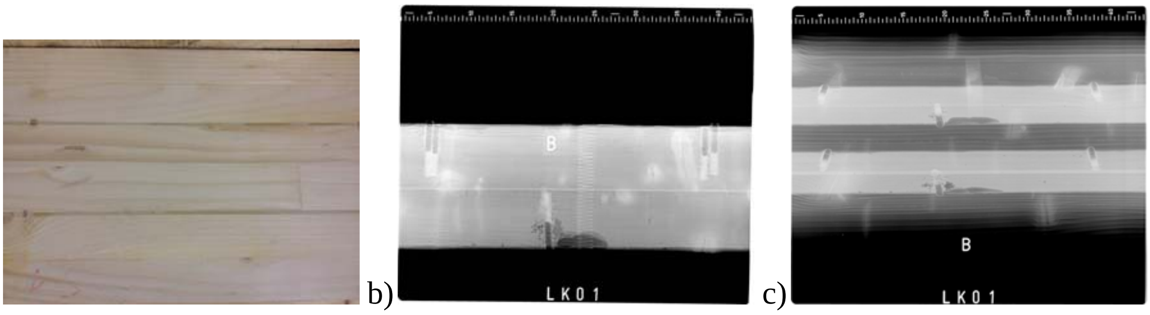
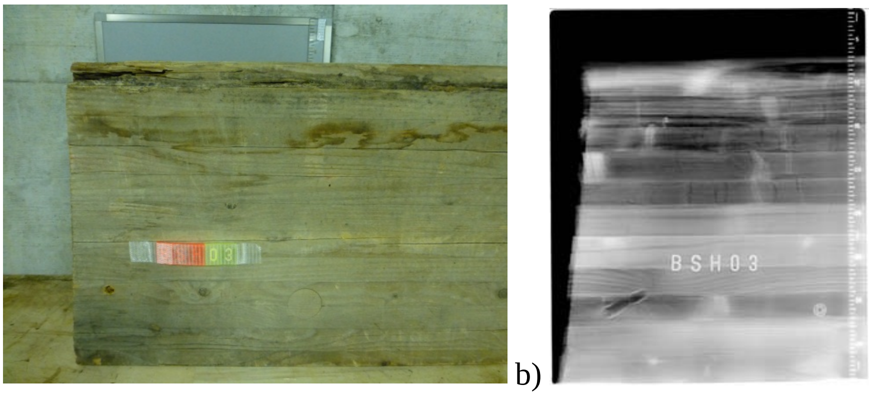


Figure 12: Investigations of the reinforcement at load bearing areas, left: beam and X-ray unit, right: radiogram with the self tapping screws and holes



a) b) c)

Figure 13: Gulam member with two repaired- cracks, a) test specimen, b) X-ray direction perpendicular to the glue-line (top view), c) X-ray direction inclined to glue-line



a) b)

Figure 14: Fungal decay in X-ray, a) test specimen, b) radiogram

In general fungal or insect decay can be observed within the visual inspection. But in some cases structural elements are covered or only viewable from one side, so that the mobile X-ray system can be used for detailed analyses or specification of assumptions. Figure 14 shows as example a glulam member with fungal decay in the top lamellae. The typical cubic failure structure is visible in the radiogram observed and allows to estimate the dimension of the decay.

4 Summary and conclusion

The X-ray system has been used in laboratory tests and practical situations at existing structures. It led to excellent results which allowed detailed analyses going further as common non-destructive assessment methods. It was shown that the mobile X-ray technology offers a high potential for an effective assessment of existing structures including connections and structural timber members. Deformations of mechanical fasteners like the formation of plastic hinges due to overloading are visible as well as the macroscopic structure of wood, knots or different wood species. Also glued connections like finger joints or restructured glue-lines were checked for quality and/or damages. Voids or bubbles but also cracks due to overloading could clearly be detected.

The practical examples presented, give an overview of the ability and the limits of this method and show that the mobile X-ray system is a novel successful non-destructive testing method of timber structures. With increase of the differences of the density of the investigated materials, the contrast is getting more and more intensive. However, reliable analyses of the resulting radiograms should be done by people who have experiences with the system and are professionals in timber structures in order to be able to identify irregularities from inaccuracies even in less contrast radiograms.

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References

[1] Aicher, S. (2008) Verfahren und Aussagemöglichkeiten bei der Begutachtung von Holzkonstruktionen in Fachtagung Bauwerksdiagnose. Berlin, Germany.

[2] Görlacher, R. (1996) Hölzerne Tragwerke: Untersuchen und Beurteilen in: Sonderforschungsbereich 315: Erhalten historische bedeutsamer Bauwerke, Verlag Ernst & Sohn, Berlin, Germany.

[3] Kasal, B., Tannert, T. (2010) In situ assessment of structural timber, State of the art report of the Rilem technical committee 215-AST, Springer Verlag, Dordrecht, Heidelberg, London, New York.

[4] Köhler J., Fink, G., Toratti, T. (2011) Assessment of failures and malfunctions, Publication of COST Action E55-Modelling of the performance of timber structures, Shaker Verlag GmbH, Aachen.

[5] Rinn, F. (1992) Chancen und Grenzen bei der Untersuchung von Konstruktionshölzern mit der Bohrwiderstandsmethode, Bauen mit Holz, 94/9.

[6] Steiger, R. (2009) Zustandserfassung und Zustandsbeurteilung, in EMPA/HSR-Conference, Rapperswil, Switzerland.

[7] Vogel, M., Tannert, T., Hansen, H., Kehl, D., Kraus, S. (2009) Überprüfungsmethoden geschädigter Holzkonstruktionen, Research report, Bern University of Applied Sciences, Biel, Switzerland.

[8] Vogel, M., Scharmacher, F. (2012) Qualitätskontrolle von Holzkonstruktionen – Mobiles Röntgen. Holzforschung Schweiz 2012/1, pp. 9-11.

[9] Brashaw, B.K., Bucur, V., Divos, F., Goncales, R., Lu, J., Meder, R., Pellerin, R.F., Potter, S., Ross, R.J., Wang, X., Yin, Y. (2009) Nondestrucitve testing and evaluation of wood: A wordlwide research update, Forest Products Journal 2009/59, pp.7-14.

[10] Wei, Q., Leblon, B., La Rocque, A. (2011) On the use of X-ray computed tomography for determining wood properties: a review, Can. Journal for Res. 2011/41, pp. 2120-2140.

[11] Pease, B.J., Scheffler, G.A., Janssen, H. (2012) Monitoring moisture movements in building materials using X-ray attenuation: Influence of beam-hardening of polychromatic X-ray photon beams, Construction and Building Materials 36 pp. 419-429.

[12] Kruglowa, T. (2012) In-situ assessment of density and material proberties in timber structures by non-destructive and semi-destructive testing, Thesis, Chalmers University of Technology , Gothenburg, Sweden.

[13] Lechner, T. (2013) Assessment of density in timber using X-ray equipment, International Journal of Architectural Heritage, 2013/7, pp. 416-433.

[14] Electromagnetic spectrum: Capture from http://imagine.gsfc.nasa.gov/docs/teachers/gammaraybursts/starchild/page2.html, July 2014

[15] Purschke, M. (1989) Verbesserung der Detailerkennnbarkeit in Röntgendurchleuchtungsbildern durch digitale Bildrestauration. Doctoral thesis, Technical University of Berlin, Germany.

[16] Johansen, K.W. (1949) Theory of timber connections. Int. Association for Bridge and Structural Engineering (IABSE) Publications, 9, pp. 249-262.

Approved Concept for Inspection of Modern and Historic Timber Structures: Theory and Practical Experiences

Frank Rinn[1]

Summary

Based on the development of resistance drilling in 1986 and combined with other methods, such as visual inspection, wood moisture measurements, and stress-wave timing, a comprehensive concept for inspection of timber structures and documentation of the results was developed in conjunction with experts from several other professions. Since 1987, several thousand historic and modern timber structures have been inspected: for example, buildings (castles, churches, family houses, sport/swimming halls), bridges, poles, harbors, and playground equipment. The major goal of the specific type of color-coded inventory sketches was to comprehensively show all relevant results of the inspection and at the same time revealing these findings in a way that can be understood by architects, engineers, carpenters, and heritage administrations in a quick and easy way without having to read text reports. The biggest difference from ordinary concepts is the step from damage documentation to condition inventory. As a consequence, costs of restorations and maintenance typically dropped by about 50% because of significantly higher planning safety (achieved by significant, reliable, and clear results).

Key words: Timber inspection, Resistance drilling, Color coded condition inventory

1 Introduction

More than 2.5 Mill historic half-timbered buildings and more than 5 Million buildings with wooden ceiling beams have to be preserved in Germany as good as possible due to regulations on historical monuments as the cultural heritage. More than 200 Billion Euro are spend for buildings in Germany every year. Approximately 60% of the building budget is spent for restoration and repair of existing buildings. However, the education of architects and engineers still mainly focusses on design of new buildings.

Between 1 and 3 Billion Euro are spent each year for preservation of historical monuments, mostly at least partially financed by taxes or lottery funds. Approximately 5% of new houses are built with structural timber (+10% p.a.). The inspections of ‘new’ timber buildings (built > 1950) is growing, due to poor quality of design, wrong use and missing maintenance.

As a consequence, the need for non-destructive timber inspection increases. However, specific boundary conditions have to be regarded:

- Architects and engineers are still mostly paid a percentage of the total costs (thus are not primarily interested in saving costs, especially if public money is involved).
- If a building is 100, 200 or even more years old and does not show significant deformation, the timber structure is supposed to be strong enough and then only decayed parts of the structure have to be replaced. That’s all! There is no need for structural analysis and calculation of load carrying capacity.
- If there are significant obvious deformations or the future use of the building will bring in more load (less than 5% of all cases) then stiffness and strength of beams have to be determined.

Based on the market, the client’s needs and the given boundary conditions, we developed a concept how to inspect timber structures in a fast, efficient and reliable way.

2 Major objection and tasks of our inspections

Non-destructively creating an easy to understand and clearly visible status report on timber structure condition without harming other (historic) fabric:

- create/modify sketch of construction covering all relevant timber beams
- determine dimensions of timber cross-sections and connections
- reliably identify decayed and intact parts of beams and connections
- (sometimes: determine MOE and estimate MOR)
- visualize results in color inventories (rather than writing long text reports)

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The inspection concept was developed in cooperation with architects, engineers, carpenters and administrations. Meanwhile, several thousand buildings have been successfully inspected.

Major steps of the inspection are

1. Create new or modify existing sketches of the construction.
2. Visually inspect all accessible parts (condition, external defects, dimensions of beams, previous repairs) - often major part of total work to be done.
3. Technical measurements:
   3.1. moisture content measurement
   3.2. resistance drilling (with calibratable machines only, such as Resistograph®)
   3.3. stress wave timing (for example using impuls hammer or Arbotom®).
4. Documentation of all results and all relevant information about timber condition into a few graphical sketches by avoiding long text reports.

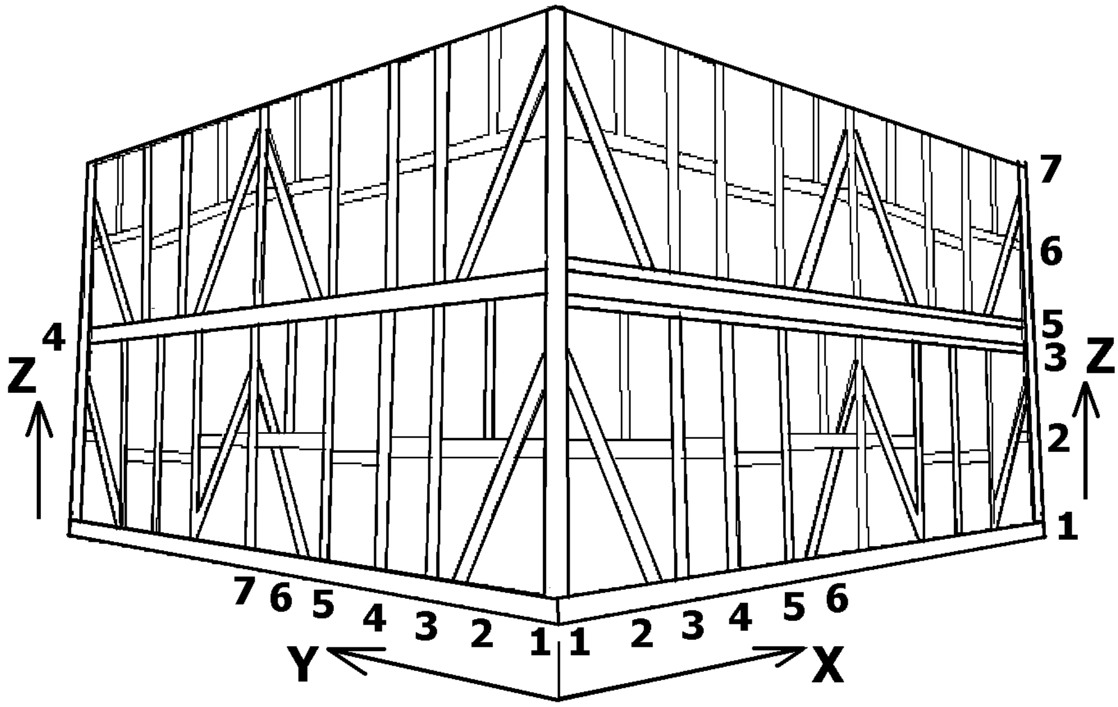


Figure 1: Example for coordinate system numbers to clearly identify timber axis and joints.

3 Basic sketch for colored inventory

All relevant beams have to be shown in at least one of the sketches. A coordinate system reliably identifies each beam and connection.

The demands made for an inventory regarding the timber construction can be easily formulated:

1. All pieces of timber relevant for the statics of the construction must be drawn in at least one plan.
2. When possible, no piece of timber should be drawn on top of any other.
3. The relative position of the beams to each other must be correct.
4. Great deformations relevant to the static construction must be included.

Unfortunately, it can be determined that existing inventories often do not fulfill these conditions. The common ground plans at approx. 1 meter height over the ceiling beams are useless for this purpose. It has proven better to draw up a new schematic sketch of the construction according to the above-mentioned conditions instead of trying to correct existing plans.

In some cases it makes sense to examine the hidden timbers in the ceiling by means of thermography. Up to now, this technique can predominantly be used in winter, because great differences in temperature are required.

4 Visual (ordinary) inspection

This first part of the inspection is often the major part of the total working time and usually consists of the following steps:

1. Look for external decay (by fungi or insects).
2. Knock on all beams and connections with ordinary hammer
3. Use handcraft tools to check in all suspicious holes/connections
4. Take samples from insect or fungal decayed parts and determine decay species.
5. Check quality of previous repairs and replacements.
6. Look for and measure major deformations.
7. Try to find reasons for decay and other defects.
8. Document all results in colored sketches.
9. Determine points where technical inspection is required.

At the end of the visual ('conventional') inspection, the colored sketch already contains a lot of information but has many white spaces where condition of the corresponding beams is still unknown.

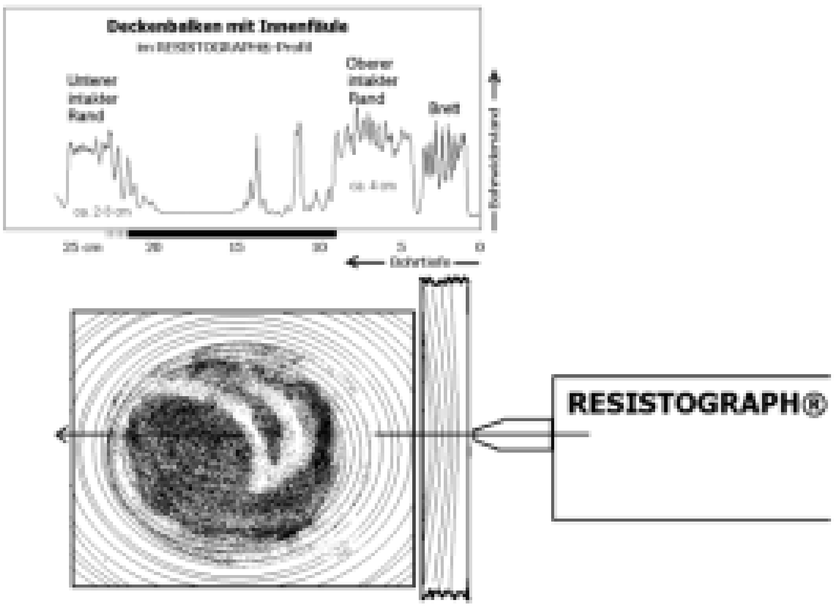


Figure 2: Resistance drilling using calibratable machines allows the experienced user to reliably identify decay even in hidden timber, such as ceiling beams below floor.

5 Technical inspection

When visual inspection was not able to clear all questions or of hidden beams have to be evaluated, technical methods are used in order to answer the remaining open questions:

1. Relative moisture content.
2. Drill resistance measurements
   a. find hidden beams behind stucco or below flooring
   b. assess depth of obvious outside decay or cracks (e.g. in glue-lam)
   c. check internal condition (of visible and hidden beams and connections)
   d. determine gross density (after calibrating drilling machine)
3. Stress wave timing: speed of sound, detect hidden cracks or connections
4. Combine (not only numeric) results, e.g. density * speed ² = MOE
5. Document all measurement points and all results in colored inventories.

Having inspected a timber structure visually and technically may lead to great results but does not help preserving historic fabric or making repair efficient if the experts planning and executing the repair work do not understand the results in an easy and clear manner.

Based on the success of the application of resistance drilling for inspecting timber starting 1986, we then developed a concept how to document inspection results that provides more precision and reliability but is, at the same time, more easy to understand for both engineers and carpenters.

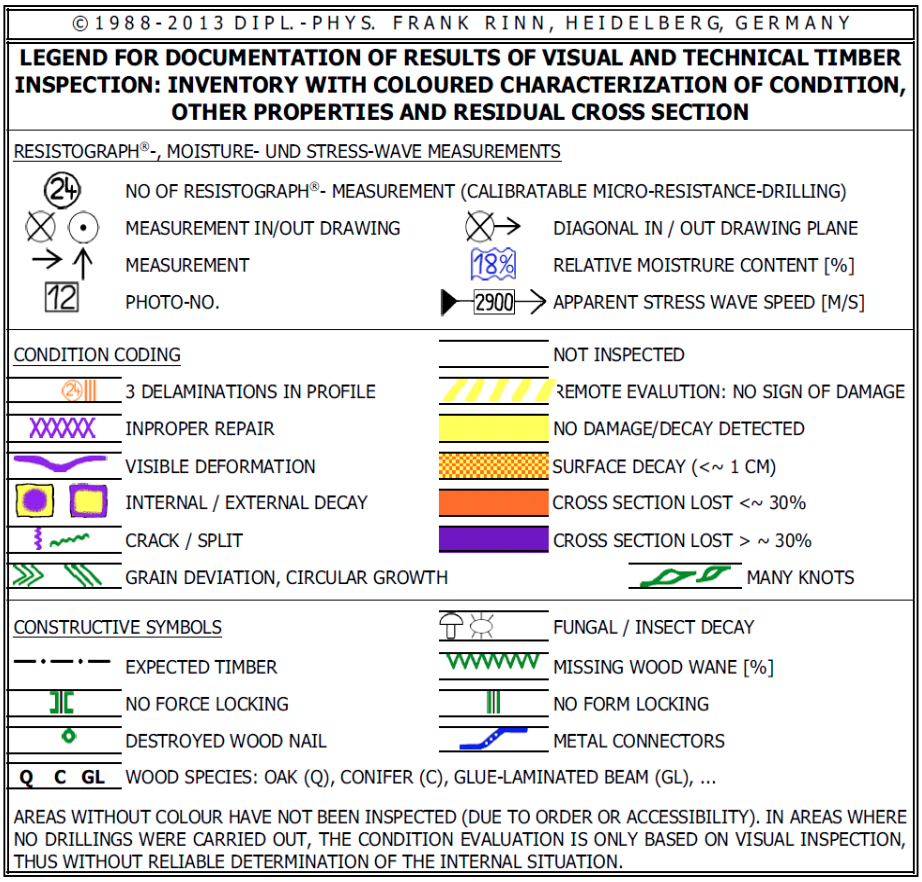


Figure 3: Legend of colored inventory sketches showing the condition of timber in three major colors and describing additional signs for specific symptoms identified at a beam or structure.

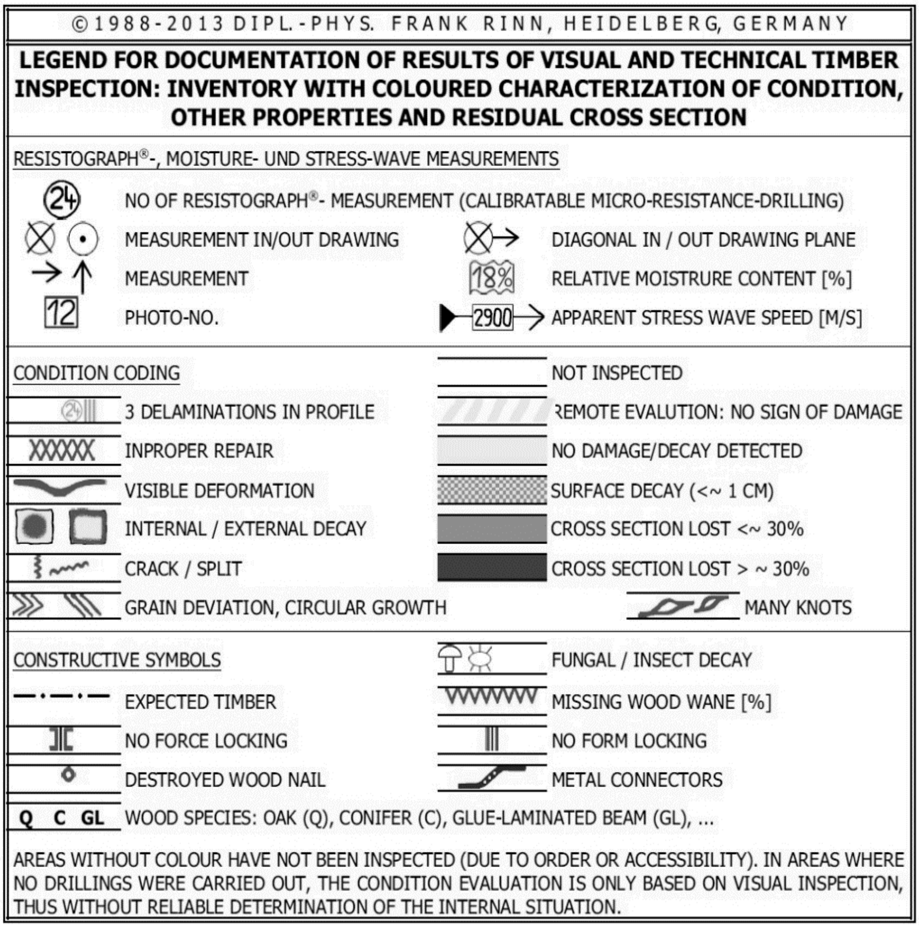


Figure 4: Black and white copy of the condition legend still providing three major condition markers reliably differentiated by different grey scales.

6 Documentation concept

The first step forward coming from black and white sketches of timber structures with shadings for marking decay was to use colors. But, in order to make the drawings as easy as possible to read, the number of main colors had to be as small as possible, at most three or four.

At the time we developed our concept (late 1980ies / early 1990ies), color copies were still quite expensive, especially if printing in larger than standard letter sizes. The colors thus had to be selected in a way that allows black and white copies still providing the major information about decay and condition (Fig. 3). Consequently, we selected red (extensively decayed), orange (mean decay), and yellow (intact) as the major colors – because they can be differentiated easily on the first view and because black and white copies still show the three colors reliably in differentiated types of grey (Fig. 4).

The traffic-light color scheme, green for intact), yellow for partially decayed, and red for strongly decayed parts, was no option because of several reasons: in a black and white copy, green was commonly darker than red, leading to a wrong impression about the condition of the corresponding parts. In addition, structural engineers in Germany commonly used green for marking structurally relevant, local aspects and symptoms, such as cracks.

The biggest step forward was introducing a color for marking parts of timber that were inspected (either visually, by tapping and/or resistance drilling) and where found to be intact and sound. This means, if a beam was tested in whatever kind and no sign of decay was found, this beam is marked with a certain color.

For the first time, this way it was possible to distinguish between the sections of a timber structure that were not inspected (no color) and the parts that were inspected without finding damages (yellow). This may sound as a tiny little aspect but changed a lot because from then on later planning and working steps did know what parts of the structure they can rely on without doubting whether these parts had been checked or not (because there was no decay marked).

Another big step forward was combining as many parts of the usually many individual sketches of a structure as possible into one single overview drawing: this reduced the total number of sketches representing the condition of a structure often from 10 to 1 or 2 – making it much easier for engineers and architects as well as for carpenters getting an overall impression about the condition of the bridge or structure as a whole. In addition, the overview given by a single sketch with a color coded condition inventory allows the identification of connections between sources and reasons of different spots or areas of decay. That means, these overview inventories provide a base for a much deeper understanding of the structure as a whole instead of only working locally on repair of individual parts.

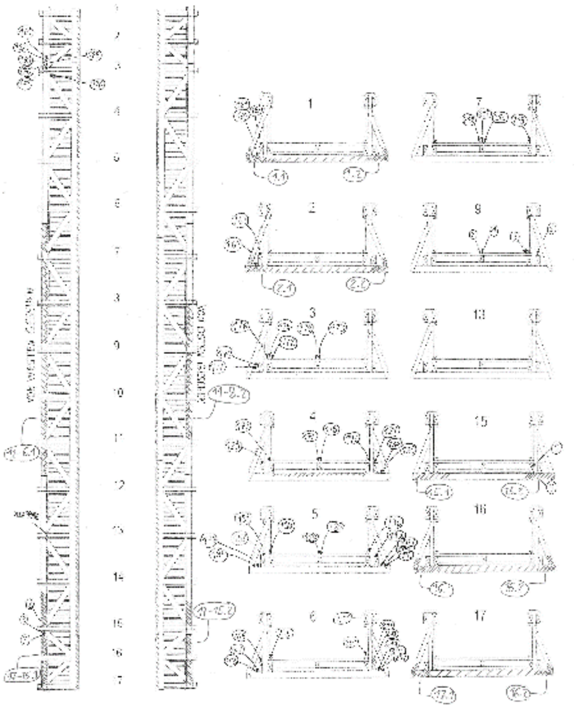
7 Practical working steps

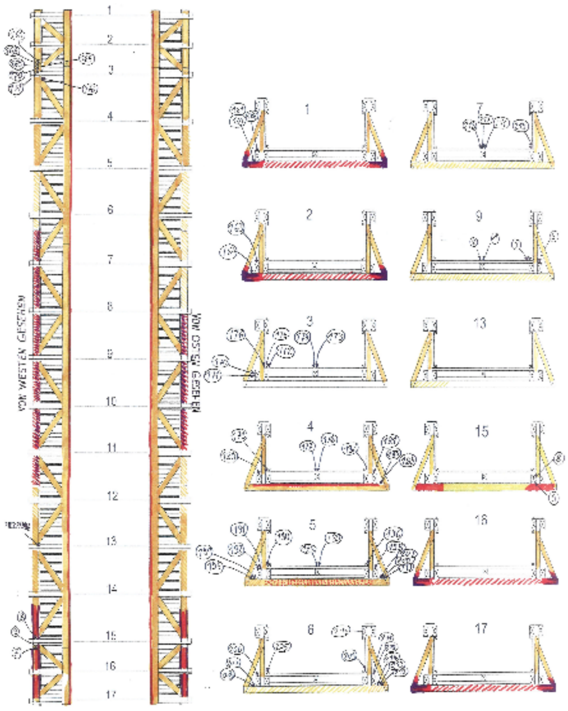
Commonly we prepare the basic drawings of structures before the technical inspection starts. Such structural sketches have to show all relevant timber parts that belong to at least one plane of the structure or are connected with this plane. While doing that, we try to avoid showing different beams in one sketch that in reality overlay each other and represent different planes – because it is impossible to show correct colors if these beams have different conditions and thus would have to be characterized by different colors overlaying each other.

Usually, the sketches are prepared in a larger size and scale for enabling the inspector on site to put in all relevant information while inspecting - as one of our major goals was to avoid text notes but reveal all relevant aspects in the sketch. And, all evaluations should be done on the spot without having to go back to office and again work on profile analysis and come to a conclusion that, for example, additional assessments are required. This is time consuming and inefficient. Our goal was to always come to a final conclusion about the condition of timber on site while inspecting because only on site at the structure you can just tap or drill another time at another spot in order to confirm unclear results or suspicious symptoms. The highest (cost and time) efficiency we always achieved when the inspection came to a final conclusion on the site and when all relevant results were documented in the color coded inventory map on site. This drawing has then only to be reproduced in the office and surrounded by a short text note.

The reproduction of the colored on-site drawing is usually done by a reduction factor of 4. These squeezed sketches then represent the most significant part of the report. In addition, the report usually contains some illustrating pictures and a short text summary with recommendations. Even the recommendations for repair work can be partially included in the color coded sketch because lines may be implemented indicating where and how damaged beams should be cut and/or replaced.

All this fits to the traditional German saying: “A good drawing is the language of a good engineer”.





Conventional black and white damage map of a timber bridge. Originally it was common to mark decayed parts with a certain kind of shading and a label that refers to the text list position of the corresponding description of the found damage.
Such a drawing consisted usually of 18 individual sketches of each axis and was accompanied by many pages of text within the report.

Colored version of an inventory map showing wood condition in different colors. The colors do not only reveal where decay was found but furthermore show what parts of the structure were found and proven to be intact. Because colors allow the reader to much easier identify damaged areas, such a combined sketch replaces many conventional drawings.

Figure 5: Typical timber bridge to be inspected because of decay (although made by tropical hardwoods).

Figure 6: Example for a simple, combined plan presentation of a half-timbered building. On top of the facade the complete rafter plane of this side is drawn. Such a combination allows for one, the recognition of possible reasons for damage, if, for example, the leaky roof has caused damages to the rafter foot points as well as to the purlin and the post head underneath it in the facade. In individual inventory plans with individual rafter axis and facade drawings, such correlations are often overlooked.

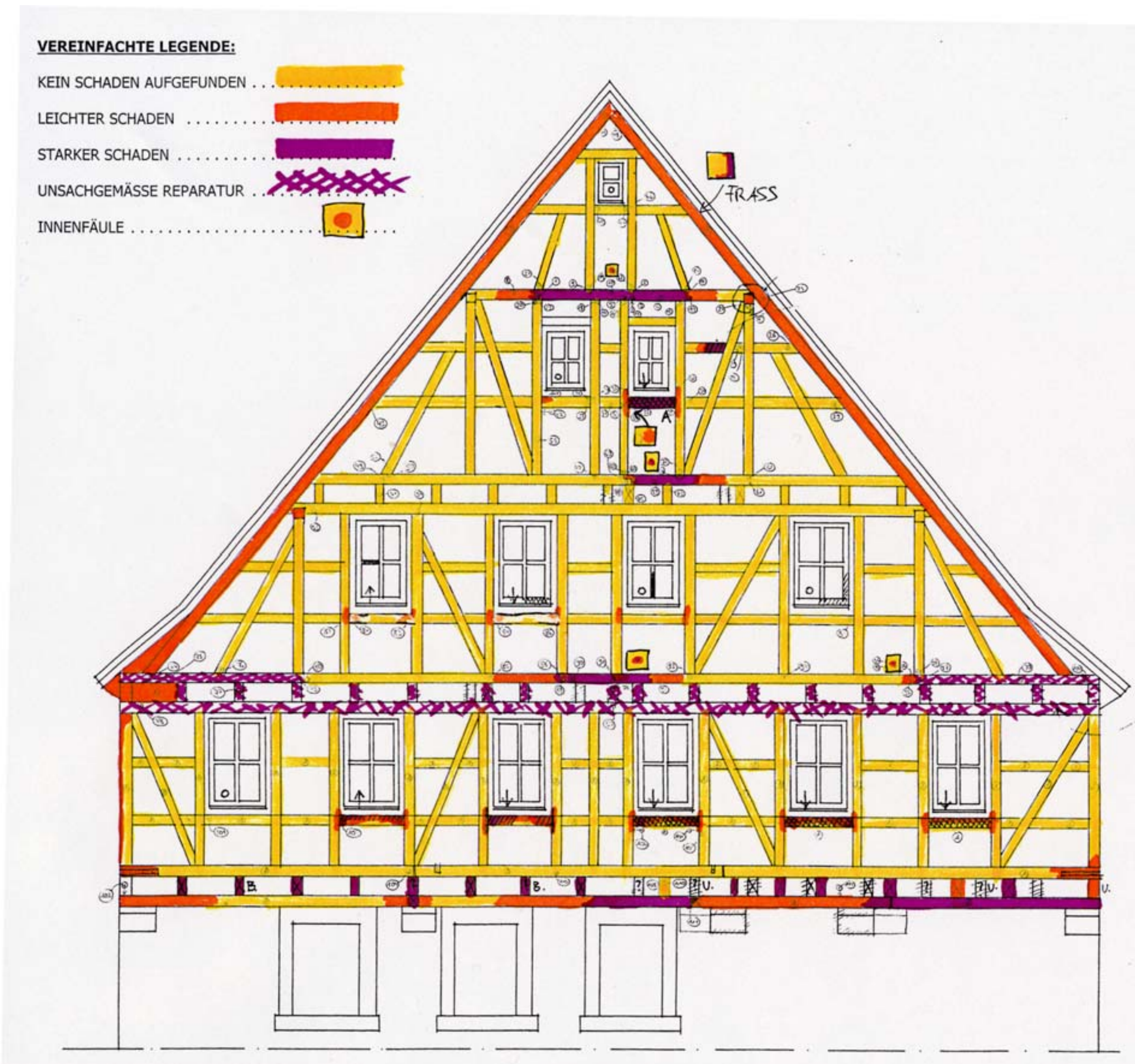


Figure 7: Overview sketch with a simplified color coded condition inventory. This inspection was carried out by one person on one day including the drawing of the inventory what is usually done on site.

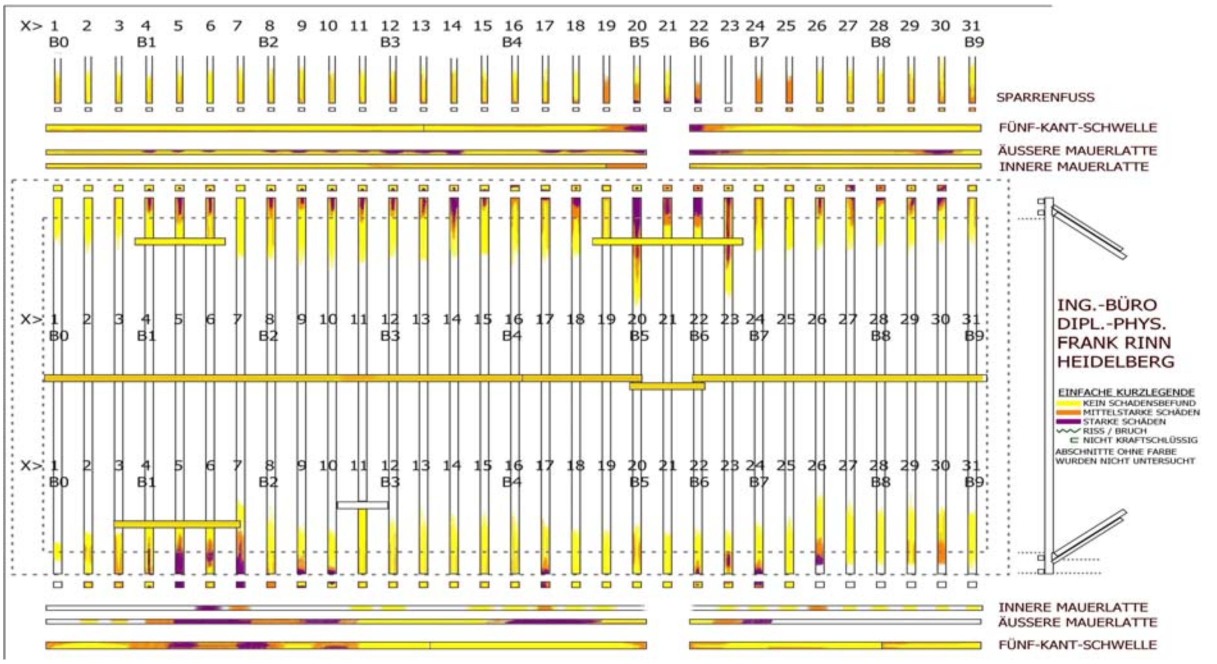


Figure 8: Ceiling beams of a historic church plus underneath wall beams and the foot parts of looming rafters. This one sketch replaced several dozen sketches of all axes of conventional documentations.

8 Consequences

Practical application of this concept in several hundred real market projects of very different size scales proved its suitability and led to a significant increase of planning safety and furthermore to dramatically reduced total costs.

References and further reading

[1] Dackermann, U., Crews, K., Kasal, B., Li, J., Riggio, M., Rinn, F., Tannert, T. (2013) In situ assessment of structural timber using stress-wave measurements. Materials and Structures. June 2013. DOI 10.1617/s11527-013-0095-4.

[2] Fischer, H.-B., Rinn, F. (1996) Bestandsplan mit farbiger Zustandskartierung der Holzkonstruktion. Bauen mit Holz 11 (1996): 852-858.

[3] Rinn, F. (1988) A new method for measuring tree-ring density parameters. Physics diploma thesis, Institute for Environmental Physics, Heidelberg University, 85.

[4] Rinn, F. (1990) Device for material testing, especially wood inspection by drill resistance measurements. German Patent 4122494.

[5] Rinn, F. (1993) Gucken, Klopfen, Bohren. Zerstörungsfreie Bohrwiderstandsmessung als Teil der ingenieurtechnischen Holzuntersuchung. Bausubstanz, 5 (1993), 49 - 52.

[6] Rinn, F. (1993) Catalogue of relative density profiles of trees, poles and timber derived from RESISTOGRAPH micro-drillings. Proc. 9th int. meeting non-destructive testing, Madison.

[7] Rinn, F. (1994) Resistographic visualization of tree ring density varia-tions. International Conference Tree Rings and Environment. Tucson, AZ, 1994. Printed in: Radiocarbon 1996, 871-878.

[8] Rinn, F. (1994) One minute pole inspection with RESISTOGRAPH micro drillings. Proc. Int. Conf. on wood poles and piles. Ft. Collins, Colora-do, USA, March 1994.

[9] Rinn, F. (1994) Resistographic inspection of building timber. Proc. Pacific Timber Engineering Conference. Gold Coast, Australia, July 1994.

[10] Rinn, F. (2006) Konzept für Zustandsanalysen von Holzkonstruk-tionen. bauen mit holz 10/2006, 26-33.

[11] Rinn, F. (2012) Basics of micro-resistance drilling for timber inspection. Holztechnologie 53(2012)3, 24 - 29

[12] Tannert, T., Anthony, R. W., Kasal, B., Kloiber, M., Piazza, M., Riggio, M., Rinn, F., Widmann, R., Yamaguchi, N. (2013) In situ assessment of structural timber using semi-destructive techniques. DOI 10.1617/s11527-013-0095-4.

Monitoring building climate and timber moisture gradient in large-span timber structures

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Summary

The evaluation of damages in large-span timber structures indicates that the predominantly observed damage pattern is pronounced cracking in the lamellas of glued-laminated timber elements. A significant proportion of these cracks is attributed to the seasonal and use-related variations of the internal climate within large buildings and the associated inhomogeneous shrinkage and swelling processes in the timber elements. To evaluate the significance of these phenomena, long-term measurements of climatic conditions and timber moisture content were taken within large-span timber structures in buildings of typical construction type and use. These measurements were then used to draw conclusions on the magnitude and time necessary for adjustment of the moisture distribution to changing climatic conditions. A comparison of the results for different types of building use confirms the expected large range of possible climatic conditions in buildings with timber structures. Ranges of equilibrium moisture content representative of the type and use of building were obtained. These ranges can be used in design to condition the timber to the right value of moisture content, in this way reducing the crack formation due to moisture variations. The results of this research also support the development of suitable monitoring systems which could be applied in form of early warning systems on the basis of climate measurements. Based on the results obtained, proposals for the practical implementation of the results are given.

Key words: indoor climate; temperature; relative humidity; monitoring; wood; glued laminated timber; timber moisture content; moisture gradients; shrinkage cracks

1 Introduction

The evaluation of damages in large-span timber structures ([1] - [4]) shows that a prevalent type of damage is pronounced cracking in the glue lines and lamellas of glulam timber elements. According to [4], in almost half of the cases, damage can be attributed to low or high moisture content (MC) or significant variations of this quantity over time. The resulting moisture gradient and the associated shrinkage or swelling will lead to internal stresses in the cross-section. If these stresses locally exceed the very low tension perpendicular to grain strength of wood, the result will be a stress relief in form of cracks.

Low or high moisture contents or severe changes of this quantity over time could sometimes be attributed to local conditions (e.g. roof leakage) but in the majority of cases, they could be explained by the climatic conditions, depending on the construction type and use of the building, and seasonal variations of the building climate. Figure 1 contains timber MC and climatic conditions for all structures for which such information was obtained during the assessment of the building. All evaluated measurements represent snap-shots of the situation on the day of assessment. They do neither give indication on the timber MC at the opening of the building (beginning of operation) nor on seasonal variations of the timber moisture content. The measured timber moisture contents for buildings in Service Class 1 (SC 1) [5] (see Chapter 2.1 for description) show pronounced variations around a mean value of u = 10.7 %. The corresponding measurements of temperature and relative humidity feature a pronounced variation as well. Structural elements in Service Class 2 (SC 2) show smaller variations (u_{mean} = 14.9 %). Structural elements in Service Class 3 (SC 3) unsurprisingly feature large variations of timber MC (u_{mean} = 22.4 %) and building climate. The mean values of timber MC in dependence of the Service Class correspond well with the values listed in [1].

The large variations in timber moisture content, temperature and relative humidity for buildings in SC 1 can partly be traced back to the diversity of types of use of these buildings. A differentiation of timber MC depending on the building use is given in Figure 2. This comparison only contains types of use for which at least three buildings could be evaluated. The timber moisture contents in closed and heated buildings are often noticeably

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low. If structural elements, featuring high timber MCs due to deficient roof structures were excluded, the mean values of timber MC in closed and heated buildings would all fall below u = 10. The mean values determined for riding rinks (u_{mean} = 18.2 %) and ice-skating rinks (u_{mean} = 21.6 %) support their categorization in SC 2 and SC 3, respectively [6].

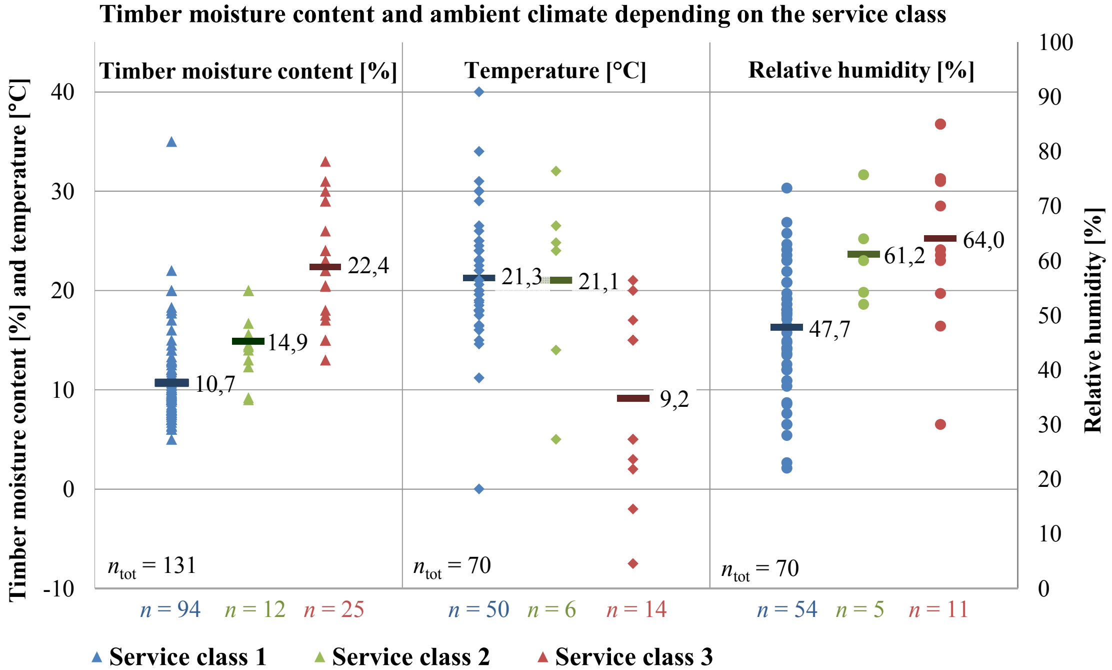

Figure 1: Timber moisture content and ambient climate depending on the Service Class, from the evaluation of 245 assessments of large-span timber structures [4]

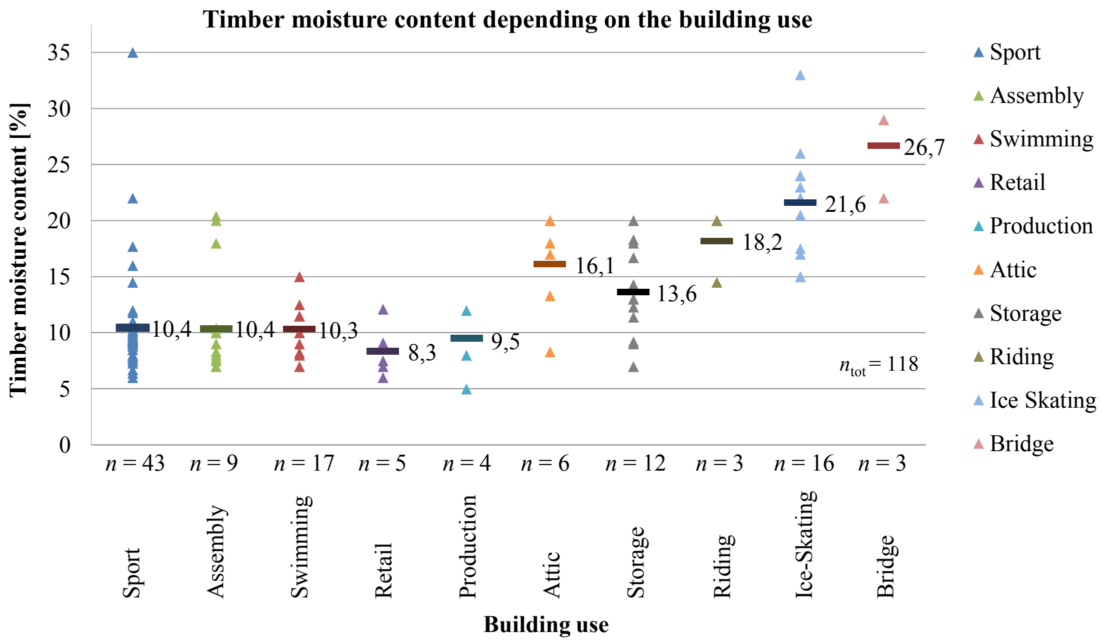

Figure 2: Timber moisture content depending on the type of building use, from an evaluation of 245 assessments of large-span timber structures [4]

Information on the sequence and magnitude of seasonal variations can only be obtained through long-term measurements of climate data (temperature, relative humidity) and timber moisture content. In the case of (large-span) timber structures, the measurement of moisture at different depths of the cross-section is of particular interest to draw conclusions on the magnitude and rate of adjustment of the moisture distribution to changing climatic conditions. Although past research projects covered the long-term measurement of timber MC and/or temperature and relative humidity [7] – [14], none of them was carried out under the objective to enable a comparison between timber structures in large buildings of different types of use. The same is valid for the long-term measurement of moisture content at different depths on structural timber elements in-situ (phase "operation" in Figure 3). Both objectives have been covered within the research project presented herein. Data received through such measurements can additionally be used to validate computational models, see e.g. [15], [16].

2 Description of the research project

2.1 Introduction

The reaction of wood to moisture forms an integral part of any task in connection with this natural and renewable building material. This also applies to the planning, execution and maintenance of buildings built with wood or wood-based products. From logging the tree to its anticipated use, e.g. as a structural element, wood will go through various phases of processing and shape during which it is subjected to varying environmental conditions. Their influence on the wood moisture content (MC) can be illustrated by the "moisture chain" (development of wood MC), sketched in Figure 3. The values given therein are indicative, more information can be found in [17] (growth), [18] (production) and [19] (operation).

Changes in wood MC lead to changes of virtually all physical and mechanical properties (e.g. strength and stiffness properties) of wood. In EN 1995-1-1 [5], this is accounted for by classifying the timber elements into one of three possible Service Classes according to the climatic conditions during the design service life. According to [5], "Service Class (SC) 1 is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 65 % for a few weeks per year. In Service Class 1 the average moisture content in most softwoods will not exceed 12 %. Service Class 2 is characterised by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 85 % for a few weeks per year. In Service Class 2 the average moisture content in most softwoods will not exceed 20 %. Service Class 3 is characterised by climatic conditions leading to higher moisture contents than in Service Class 2" [5].

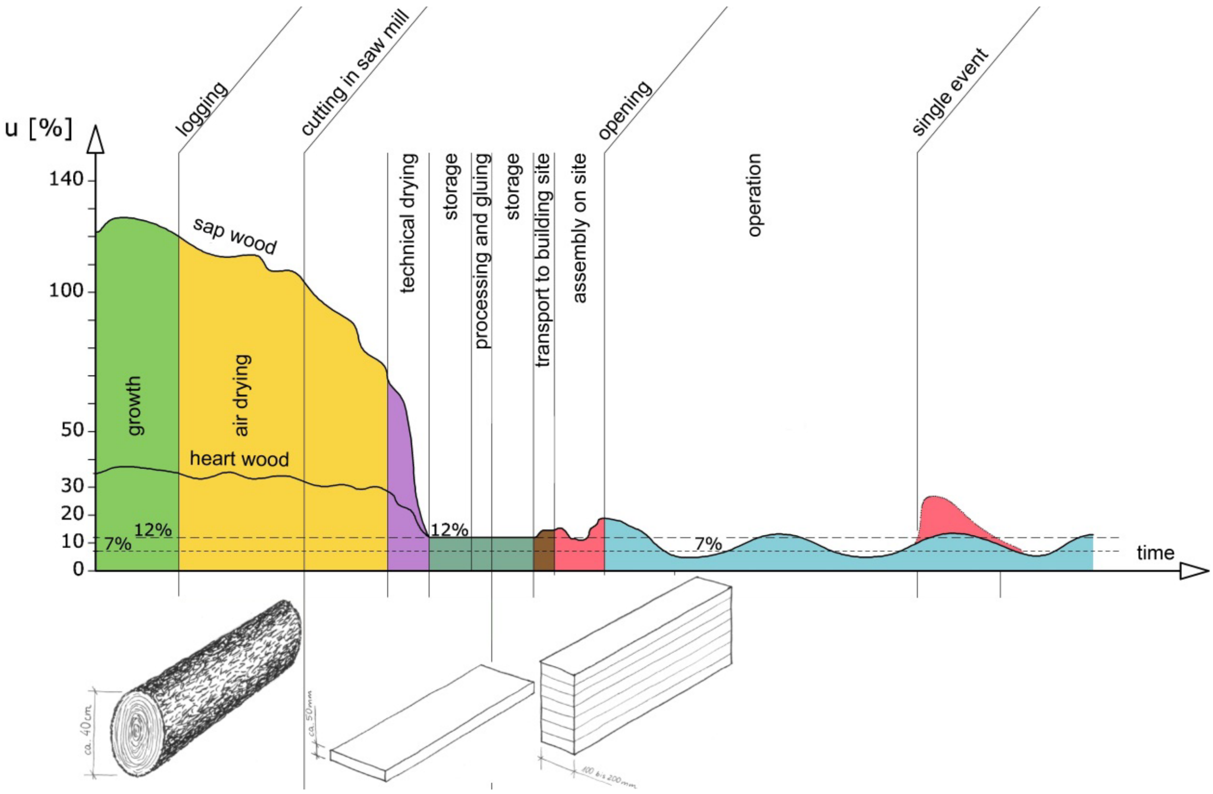

Figure 3: Sketch of a possible „moisture chain", i.e. development of wood moisture content from the tree to glued-laminated timber elements in the building (the values given in this schematic are indicative)

An additional effect of wood MC variations is the associated shrinkage or swelling of the material. Since the outermost fibres of the wood cross-sections adapt faster to the climatic conditions, the resulting moisture gradient and the associated shrinkage or swelling will lead to internal stresses in the cross-section. If these stresses locally exceed the very low tension perpendicular to grain strength of wood, the result will be a stress relief in the form of cracks which can reduce the load-carrying capacity of structural timber elements in e.g. shear or tension perpendicular to the grain.

2.2 Chosen types of use and types of buildings

Within this research project, long-term measurements of timber moisture content (MC), temperature and relative humidity in a total of 21 buildings (halls with large-span timber roof structures) with seven different types of use (destinations) were carried out (see Table 1). All buildings are located within 120 km around the City of Munich. While all buildings used as "indoor swimming pool", "gymnasium (sports facility)" and "production and sales" were heated and featured closed building envelopes, all buildings used as “riding rink”, “agriculture” and warehouse” were unheated and featured partly open building envelopes. In the case of ice-skating rinks, only closed buildings (climatized and non-climatized) were chosen since results for open or partly open ice-rinks are already available [11], [14]. When selecting the buildings, attention was given to cover the typical types of structural systems for large-span timber roofs. Only structures featuring softwood glulam with at least 140 mm width were investigated. In each building, the data was collected at two different locations of measurement in order to capture possible varying climatic conditions, e.g. due to solar radiation or due to the influence of heating systems. All necessary information for each building (e.g. building envelope, environmental conditions, climatization, structural system, element dimensions, surface treatment and locations of instrumentation) was prepared in separate building information sheets, including plan view, sectional view and photo documentation.

Table 1: Chosen types of use and number of buildings in each use

	Use	Number		Use	Number
A	Indoor swimming pool	3	E	Production and Sales	2
B	Ice rink	4	F	Agriculture (livestock)	3
C	Riding rink	3	G	Warehouse	3
D	Gymnasium (sports facility)	3		Total	21

2.3 Method of measurement and verification of measured data

The electrical resistance measurement method was chosen for the measurements of the timber moisture content. This method is reliable and widely applied, and allows for the non-destructive measurement of moisture gradients across the cross-section at one specific location (see e.g. [20]). For MC 6 % $\leq$ u $\leq$ 30 %, commercially available moisture meters feature variance tolerances of $\pm$ 1.0 % or lower. For an in-depth explanation of the electrical resistance measurement method and an overview and comparison of alternative methods for a continuous measurement of timber MC, the reader can refer to [21].

The measurement system had to be able to cover MCs in the low range which implies the measurement of high electrical resistances (e.g. 6 % MC in Norway spruce $\approx 10^{11}$ Ω). To validate the chosen measurement system, this system, was installed on test specimens of glued-laminated timber from Norway spruce (Picea abies) and exposed to very dry, very humid and varying climate in the climate chambers of the materials testing laboratory at the Technische Universität München. The MCs were continuously measured with the adopted measurement equipment and compared to the results obtained using a calibrated reference moisture meter (GANN Hydromette RTU 600). There was neither a significant difference in the results of the two measurement systems, nor when using different types of electrodes, demonstrating the accuracy of the chosen electrical resistance measurement method. For further verification, two independent series of 4 x 6 test specimens from Norway spruce were produced and stored under four different controlled climatic conditions [22] which were realized by saturated saline solutions. After the specimens had reached constant weight, their MC was measured with the chosen moisture meter (Scanntronik Gigamodule) and two reference meters (GANN Hydromette RTU 600 and Greisinger GMH 3850). By subsequent kiln-drying, the actual MC was determined. Within the range of timber MC measured during this research project (u_{max} = 19 %), good agreement was obtained for MCs between 12 % and 18 %. Maximum absolute deviations in MC of 1.3 % were measured for the dry specimen, whereby the chosen moisture meter as well as the reference moisture meters tend to underestimate the actual MC at low ranges.

2.4 Installation of measuring equipment, readout and processing of data

At each location of measurement, four pairs of teflon-isolated electrodes (GANN) with varying length were installed to enable the measurement of moisture content (MC) at clearly defined depths of the cross-section. To prevent erroneous measurements in the case of surface condensation, the heads of the electrodes were also partly teflon-isolated, see Figure 4. For exact positioning of the electrodes in one lamella, ideally perpendicular to the grain, a drill guide featuring two diameters for each depth was used in connection with a drilling template. The ram-in electrodes were connected to the moisture meter by custom-built, shielded coaxial cables. The moisture meter developed in cooperation with the project partner enables the determination of MC at up to eight channels. The measurements were taken every hour. Each channel was actuated separately to prevent mutual interference. Subsequently, the measurements were transmitted to a data logger. The climate data was recorded via a second data logger in combination with a sensor unit for relative humidity and air temperature. In addition, the surface temperatures at the two points of measurement were recorded to allow for the temperature compensation of the MC, see Figure 4.

After installation of the measuring equipment at two locations of the roof structure in each of the 21 buildings, the data stored in the data loggers was read out three times over the measurement period. A manual readout was preferred to remote transmission since it could be combined with a reference measurement taken with another moisture meter, a function control as well as a control of the point of measurement itself. During these controls and the subsequent data analysis, a few noteable issues were observed. In the indoor swimming pools, the chlorous climate resulted in accelerated corrosion and temporary malfunction of the climate sensors. This could be eliminated by exchanging them for digital, dew resistant sensors, see also [21].

In ice-skating rink "B2", a power line, although attached to the opposite side of the beam, led to an occasional shifting of the measurements for the duration of a few hours. Condensation around the point of measurement in buildings "C3" and "G1" caused a short-circuit between the non-isolated plug-connections of the electrodes, resulting in a temporary deviation of the measurements for the duration of a maximum of three days. In all cases, the corresponding data was ignored and linear interpolation was applied between the last and first set of correct measurements.

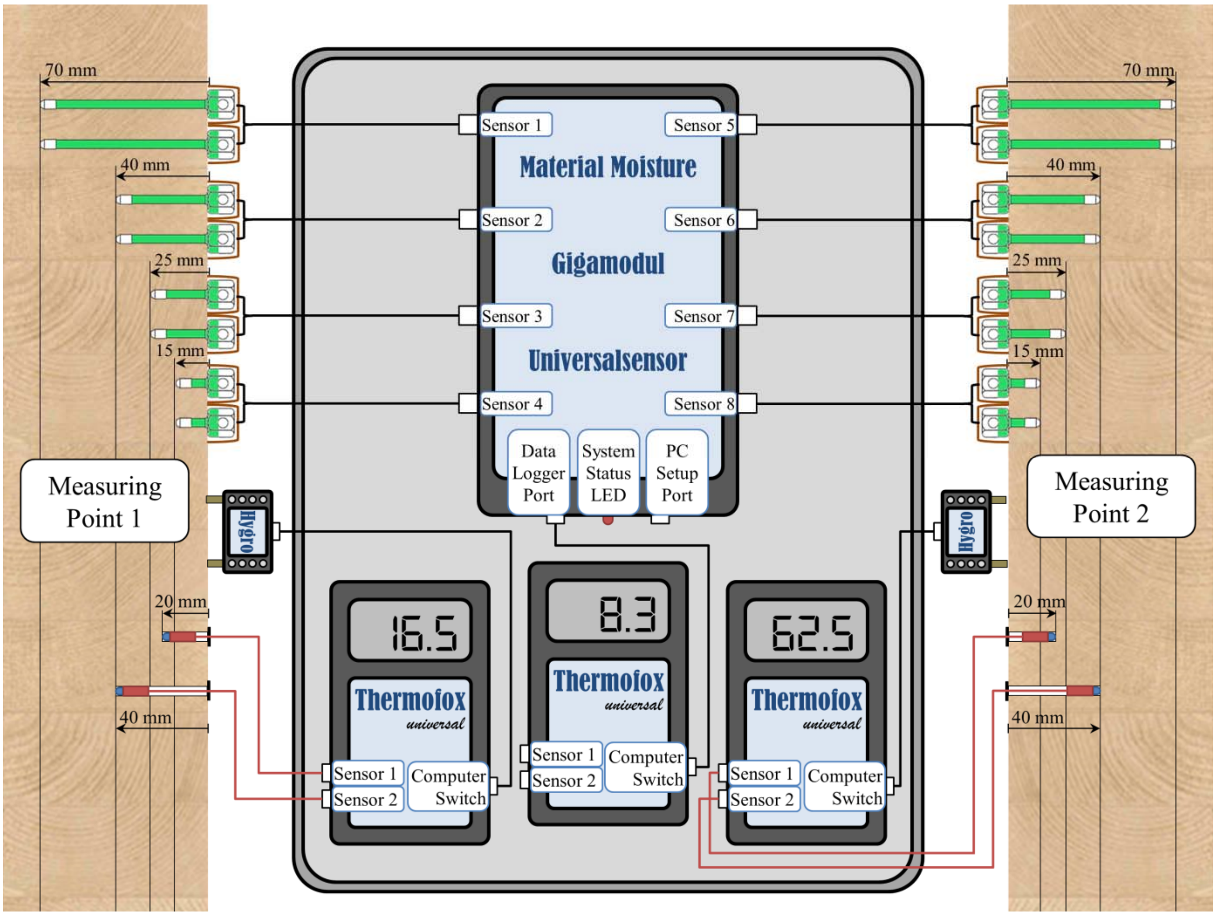

Figure 4: Schematic of the measuring equipment

To analyse the data, an Excel program was developed which made it possible to read the large amounts of data at the end of the planned duration of measurement and to further process and graphically illustrate the data in different charts. When converting the raw data, i.e. measurements of electrical resistance into timber MCs, a compensation of the effect of temperature was considered. To this aim, the actual material temperatures at the different depths were calculated from the measured surface temperatures, using the explicit Euler method [23] in combination with values for thermal conductivity given in e.g. [24] (see also [25], [26]). A modification of the measured timber MCs with respect to the differences to the values determined by kiln-drying observed during the laboratory tests (max. $\Delta u = 1.3$ %), was not undertaken since all measured timber MCs were in the range of the accepted variance tolerance ($u \leq 20$ %).

For comparative reasons, the measurements of relative humidity and temperature were used to determine the equilibrium MC prevailing in the cross-section near the surface as a moving average over ten days. This was done by applying the theoretical model of Hailwood & Horrobin [27] in combination with the coefficients determined by [28] (see also [25]). The influence of surface treatments which were present on the timber roof structure of ice-skating rinks "B1" and "B4" was not considered since the type of treatment could not be determined unambiguously.

3 Results

3.1 Processing and representation of results

Within the evaluation period from 1 October 2010 to 30 September 2011, a total of over 2.2 million records were collected and analysed by means of a specially developed Excel program. The data read from the data loggers was prepared as curves (time series) of relative and absolute humidity and temperature at the location of measurement over time, see Figure 5. The same type of representation was chosen for the measurements of timber moisture content (MC) at the four depths of the cross-section, see Figure 6. This figure also contains the calculated equilibrium MC. In addition, graphical representations over the cross-section were derived for the timber MC. This type of representation allows to create envelope curves of minimum and maximum timber MCs, see Figure 7, as well as envelope curves of minimum and maximum timber moisture gradient grad(u) = du / dx, see Figure 8. The graphical representations confirm the damped and delayed trends of timber MC with increasing depth.

Indoor climate at measuring point 1

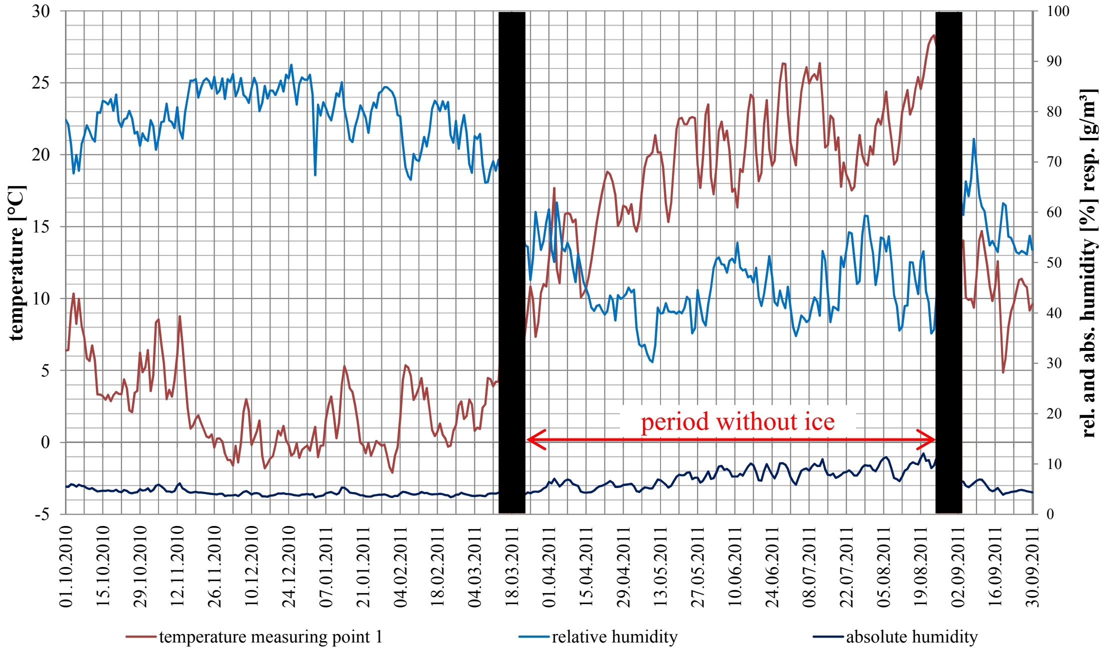

Figure 5: Variation of the relative and absolute humidity and the reference temperature over the measurement period, for the ice rink B2

Indoor climate at measuring point 1

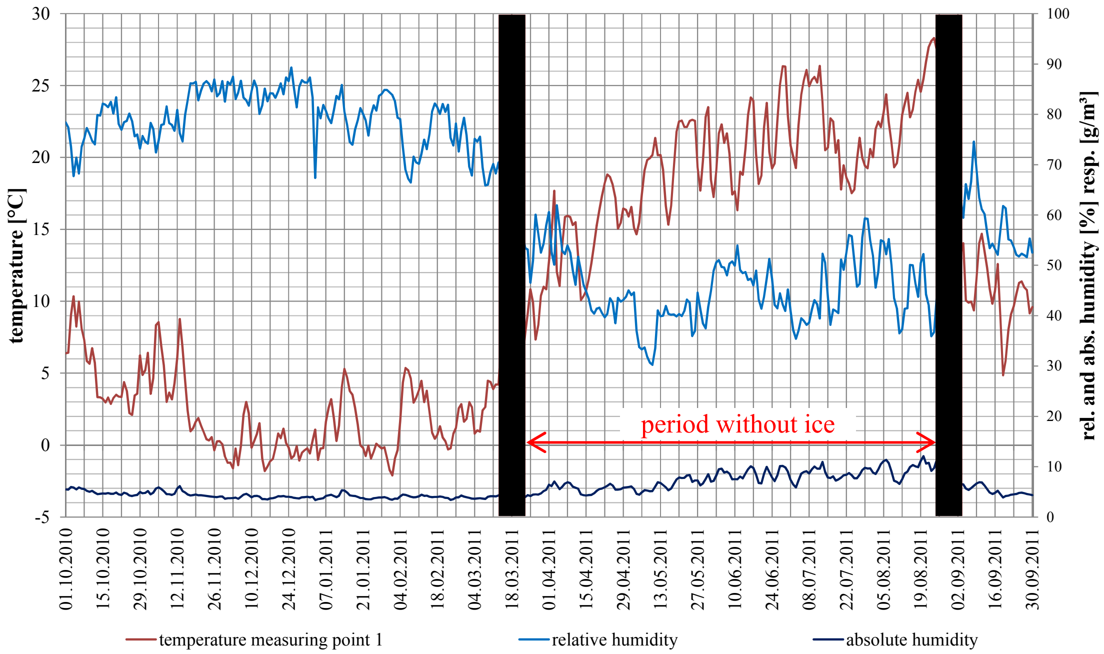

Figure 6: Variation of timber moisture content at different depths of the cross-section over the measurement period, for the ice rink B2

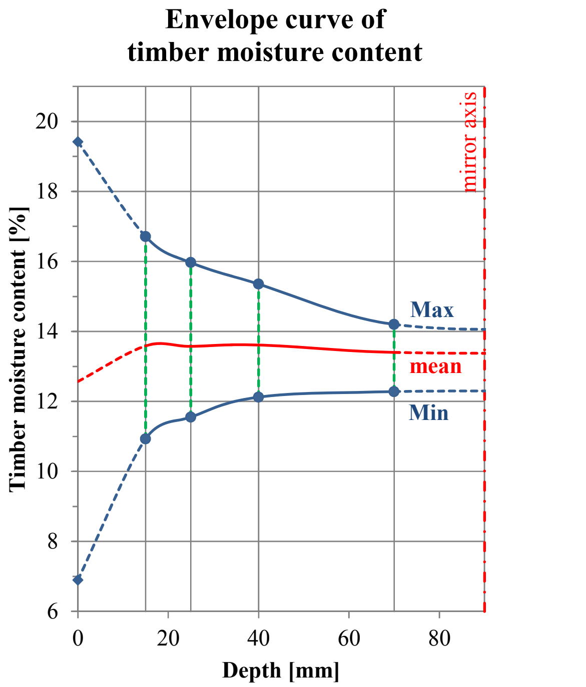

Figure 7: Envelope curve of the timber moisture content at different depths of the cross-section, for the ice rink B2

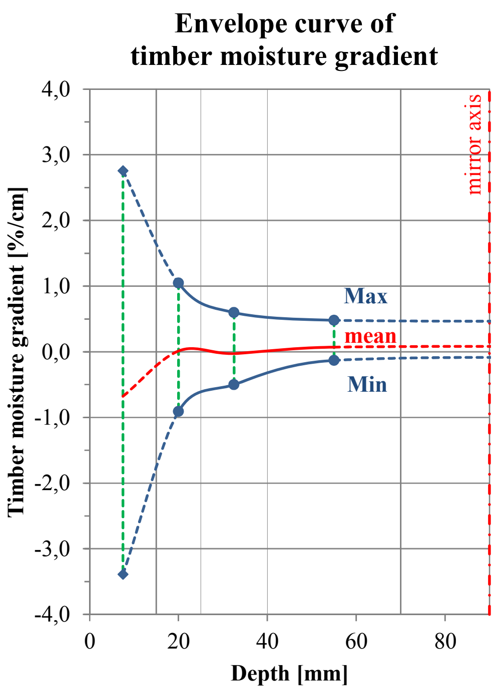

Figure 8: Envelope curve of the timber moisture gradient at different depths of the cross-section, for the ice rink B2

3.2 Results and remarks with regard to the different types of use

In the following, a summary of the results of all buildings is provided in tabular format, see Table 2. This type of representation was chosen since a graphical representation does not allow for a quick and concise overview of the results of all buildings. For the graphical representations, the interested reader is kindly referred to the final report [19] of the research project. The table contains the mean values of relative humidity and temperature (both based on daily mean values) as well as the mean value of timber moisture content (MC), averaged across all depths. In addition, the maximum amplitude, i.e. the difference between maximum and minimum value measured during the evaluation period, is given for all parameters. For the timber MC, the maximum gradient between two depths as well as the maximum difference in timber MC between the outermost (15 mm) and the innermost point of measurement (70 mm) is given. Figure 9 contains a graphical explanation of all data given in Table 2.

A comparison of the results of the individual types of building use confirms the expected large range of possible climatic conditions in buildings with timber structures. Evaluated for all types of use, the average timber MCs ranged between 4.4 % and 17.1 %, whereby the lower threshold should be regarded as a special case. As expected, the moisture gradients are lower in insulated and heated buildings than in non-insulated, partly open buildings with stronger influence of the naturally varying outdoor climate. If not explicitly stated, the numerical values of timber MC (u), temperature (T) and relative humidity (RH) given in the following, represent mean values.

Very constant climatic conditions (T ≈ 30°C, 50 % RH) were found for indoor swimming pools (buildings “A1” and “A3”) during standard operation. The timber moisture content (MC) featured small variations and small gradients. Transition zones to the outside air (building “A2”) represent an exception due to the lowering of the temperature which results in higher and more fluctuating relative humidity and timber MC.

In gymnasiums (sports facilities, buildings “D”), constant climate was observed as well. The relative humidity ranged between 40 % and 50 % and since all buildings were heated, the temperatures mostly remained constant at about 20° C. This resulted in constant timber MCs between 8 % and 10 % and very small moisture gradients. Building “D1” represents an exception since the roof structure is situated in a shed roof with skylights. This resulted in high temperatures and low relative humidities (RH = 28 %). The corresponding structural elements were very dry (MC of 4 % - 6 %). It should be noted that the measuring equipment tends to slightly underestimate the MCs at the low range (max. Δu = 1.3 %), see section 2.2.

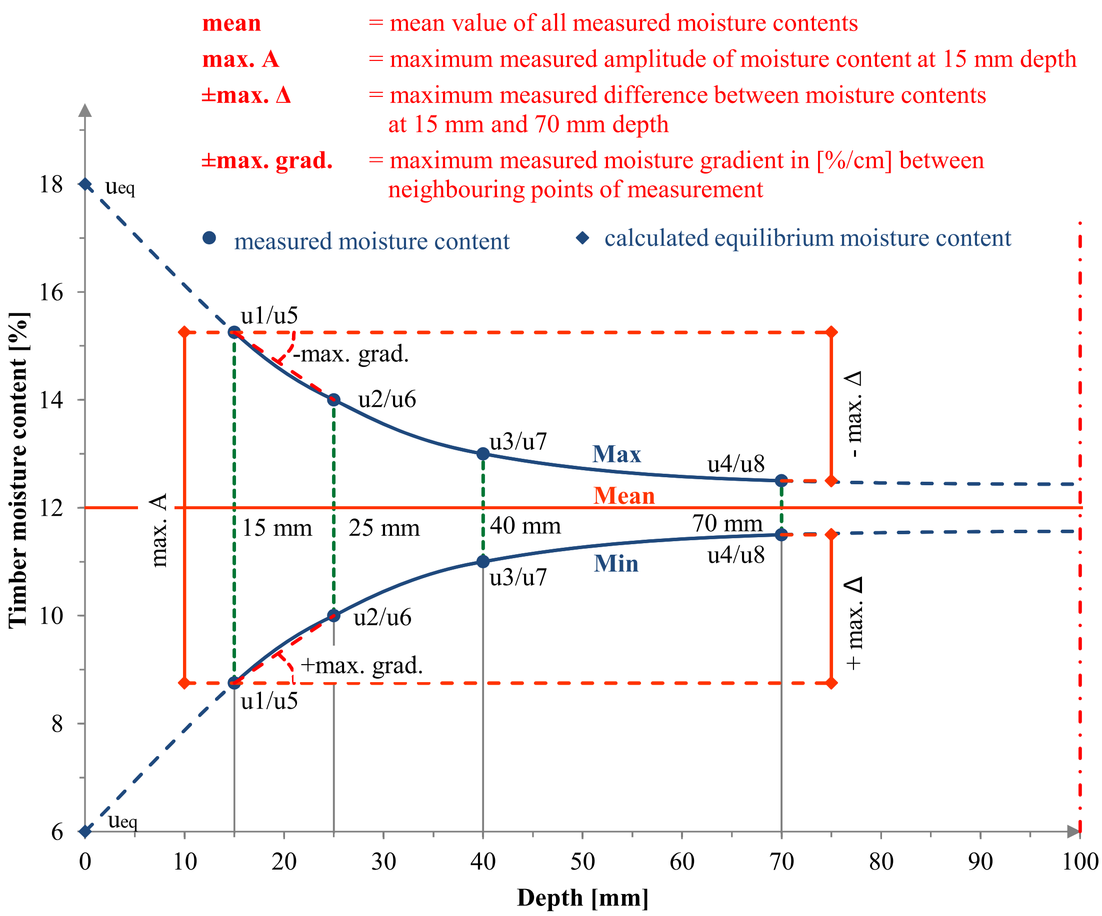

Figure 9: Schematic of envelope curve of moisture contents in the timber cross-section including notation of analysed parameters

The climate in both buildings “E - production and sales facilities” is only partially comparable due to their different type of use. Both halls are non-insulated and partly open but due to the heating system, the influence of the outside climate on temperature and relative humidity are damped. Therefore the timber moisture gradient was relatively constant. In building “E2”, the metal processing and ironwork resulted in high temperatures below the roof (temporarily above 30°), combined with very low relative humidities (temporarily below 20 %). The resulting timber MC was about 5 %.

The ambient climate in closed, non-air conditioned ice rinks (buildings “B1” and “B2”) was marked by a distinct change between winter (T = 4° C; 75 % RH) and summer months (i.e. ice-free period with T = 15° C; 60 % RH). The timber MC in ice-skating rinks was high and varied noticeably. In air-conditioned buildings (buildings “B3” and “B4”), this effect was significantly dampened. In buildings “B1” and “B4”, the film-forming surface treatment showed a damping effect on the moisture gradient. During operation (ice season), the timber MC in structural timber elements above the ice was on average 1.5 % higher than in elements above other areas. It should be noted that the measurements were taken at the side faces of the beams and not at the bottom side facing the ice. Surfaces facing the ice cool down due to radiation exchange. This can lead to condensation, partly resulting in the formation of an ice-layer, and in the case of timber elements to increased MC, see e.g. [11].

The climate in riding rinks (buildings “C”) was marked by seasonal variations leading to high amplitudes of temperature and relative humidity, the latter at high level (RH = 78 %). During the winter months, the combination of cold air in the non-insulated and unheated buildings and the humidity introduced by the sprinklers (to capture the dust), frequently resulted in condensation. Like in other types of buildings which are influenced by the outside climate, the timber MCs were higher (MC ≈ 16 %) and featured stronger seasonal variability. Due to the seasonal nature of the variations, these result in noticeable but not in exceptionally high timber moisture gradients.

Similarly strong seasonal variations of climatic condition were found for agricultural buildings with livestock (buildings “F”), the relative humidity being slightly lower (RH = 70 %). In the winter months, the interaction of the cold outside air and increased humidity in the non-insulated, unheated and partly open buildings resulted in high timber MCs and partly in condensation.

Table 2: Numerical summary of measurement results

Building	**Moisture content at measuring point 1**						**Moisture content at measuring point 2**						**Temperature**		**rel. Humidity**	
	mean	max. A	±max. Δ		±max. Grad.		mean	max. A	±max. Δ		±max. Grad.		mean	max. A	mean	max. A
	[% MC]		[% MC]		[%/cm]		[% MC]		[% MC]		[%/cm]		[°C]		[% rh]	
A	Indoor swimming pool															
A1	8,7	1,4	1,0	0,0	0,1	-0,2	9,3	1,2	0,4	-0,2	0,5	0,0	29,7	6,7	48,3*	6,8*
A2	16,1	1,8	0,6	-0,5	0,6	-0,4	15,0	2,6	1,6	-0,6	1,3	-0,3	28,7	6,0	88,6*	19,4*
A3	8,7	1,6	4,8	2,3	1,4	0,7	7,7	1,8	1,7	0,2	1,0	0,3	30,5	19,5	45,6*	29,0*
*In these buildings, a temporary malfunctioning of the climate sensors was encountered. The values given represent the periods of regular measurement.																
B	Ice rink															
B1	15,5	3,3	1,7	-1,0	0,9	-0,5	14,2	2,5	0,4	-1,9	0,7	-0,3	9,4	26,2	69,0	44,0
B2	13,5	5,8	1,9	-2,8	0,9	-1,0	15,2	6,6	1,9	-3,9	1,2	-0,8	9,9	29,9	62,2	59,1
B3	10,8	5,1	3,8	-1,6	1,5	-1,0	9,6	4,0	2,1	-1,7	1,3	-0,4	19,9	14,1	40,2	57,0
B4	13,3	1,9	0,9	-0,6	0,7	0,2	14,9	2,8	-0,3	-2,1	0,0	-0,7	9,2	18,8	68,3	44,7
C	Riding rink															
C1	17,1	3,3	1,3	-1,0	0,6	-0,5	16,4	3,4	0,0	-2,8	-0,2	-1,2	13,3	22,5	79,7	52,6
C2	15,5	5,1	0,1	-3,5	-0,1	-2,8	15,8	3,8	1,2	-1,4	0,8	-0,7	10,5	28,6	77,8	48,6
C3	14,4	4,9	2,7	-1,5	0,7	-1,1	15,5	4,5	1,8	-1,6	0,8	-0,5	9,8	30,5	77,9	52,3
D	Gymnasium (sports facility)															
D1	4,4	2,1	0,6	-0,3	0,3	-0,2	5,9	1,2	1,1	0,0	0,7	0,2	27,4	26,7	27,7	29,6
D2	8,0	2,0	0,7	-0,9	0,2	-0,3	8,1	2,1	1,1	-0,6	0,6	-0,2	20,6	16,7	42,8	42,0
D3	10,2	2,2	1,3	-0,5	0,8	-0,1	10,0	2,1	1,7	-0,2	0,7	-0,1	20,8	7,9	51,2	34,0
E	Production and Sales															
E1	7,7	1,8	0,6	-1,2	0,5	-0,1	7,8	1,6	0,3	-1,3	0,5	-0,1	18,4	17,5	40,9	38,6
E2	4,8	1,9	0,5	-0,7	0,7	-0,3	4,7	2,2	0,9	-1,1	0,5	-0,9	27,1	21,3	25,8	49,9
F	Agriculture (livestock)															
F1	16,4	3,7	-0,9	-3,7	-0,3	-1,2	15,6	3,0	-0,9	-2,7	-0,5	-1,9	11,6	21,6	74,7	45,6
F2	14,9	5,6	-0,1	-2,8	-0,7	-2,1	15,1	3,7	0,2	-2,1	-0,1	-1,4	14,2	22,4	68,4	48,1
F3	14,4	4,7	-1,3	-5,5	-0,9	-2,8	15,2	4,5	-1,2	-5,1	-0,7	-2,6	12,6	28,2	69,2	54,1
G	Warehouse															
G1	10,5	8,7	3,0	-5,2	1,2	-3,2	13,9	5,4	1,4	-2,6	0,7	-2,1	10,1	32,6	74,3	62,5
G2	13,3	6,1	1,2	-4,4	1,2	-1,4	12,7	3,6	0,7	-2,5	0,5	-1,0	9,7	32,5	67,1	54,0
G3	11,5	3,6	1,7	-1,4	1,1	-0,3	12,1	2,9	0,7	-1,7	0,7	-0,7	13,4	25,6	61,3	44,0

Since warehouses (buildings "G") are oftentimes realized as partly open buildings, the climate is highly influenced by the outside climate. The mean values of timber MCs ranged between 10 % and 14 %, their variation was amongst the highest of all evaluated types of use. Building "G1" is used to store plants during winter. The additional humidity introduced by the plants resulted in high relative humidity and occasionally in extensive condensation. The structural elements below skylights (i.e. exposed to direct sunlight) featured the highest amplitude and moisture gradient of all buildings evaluated.

In addition to the previously described, construction and use-dependent climatic conditions, the results of the research project highlight one more important aspect. Temporary interventions (such as renovations) or changes of use (temporary or permanent) can lead to major changes in climatic conditions, which are reflected by distinct changes in timber MC. Within this research project, strong drying of timber elements (renovation of indoor swimming pool "A3" and temporary conversion of ice-skating rink "B3") as well as strong moistening of dry timber elements (conversion of former metal-processing production facility "E2") was measured. Although the evaluation period could sometimes not cover the full effect of the intervention, a noticeable increase of the moisture gradient was observed. Accordingly, care should be taken during such interventions to realize a decelerated change of climatic conditions.

4 Conclusions

4.1 General

Historically the subject of moisture content in structural timber elements tended to be treated from the viewpoint of how to prevent high moisture contents to inhibit decay or growth of fungi. The evaluation of damages in large-span timber structures shows that cracking parallel to the grain due to low or severe changes of moisture content is amongst the prevalent types of damage in such structures. These cracks reduce the capacity of the cross-section to transfer tension perpendicular to grain or shear stresses. Shrinkage related cracking might be less pronounced in structural elements from solid timber if the correct sawing patterns are applied. Structural elements from glued-laminated timber with large cross-sections are more vulnerable in that aspect due to their reduced adaptability to changing ambient climate. Fast and/or significant changes of ambient climate can be due to the type of construction and use of the building. Locally, these changes can be intensified, e.g. around skylights or in the proximity of heating systems.

The conclusions and recommendations linked to this project can be grouped into conclusions that are directly derived from the results given above and guidance on best-practice that is supported by the results of this research project.

4.2 Conclusions from the research project

A comparison of the results for buildings of different type and use confirms the expected wide range of possible climatic conditions (temperature, relative humidity) in buildings with timber structures. Evaluated for all types of use, the average moisture contents range between 4.4 % and 17.1 %. The graphical representations confirm the damped and delayed adaptation of timber moisture content with increasing depth. The moisture gradients are lower in insulated and air-conditioned buildings compared to buildings with stronger influence of the naturally variyng outdoor climate. The closed, insulated and heated buildings (indoor swimming pools, gymnasiums, production and sales) featured rather constant but dry climate. Here, the most severe change of moisture content will mostly occur during the first winter of operation, after assembly and closure of the building. Accordingly, care should be taken to realize a decelerated change of ambient climate. The second group of buildings featured strong but periodic changes of moisture content (e.g. ice-skating rinks), partly caused by an increased influence of the outdoor climate on the indoor climate in unheated and non-insulated buildings (e.g. riding rinks, agriculture, warehouses).

In addition to the previously described, use-dependent climatic conditions and their influence on timber moisture content, the results of the research project identify another important aspect. Temporary interventions (such as renovations) or changes of use (temporary or permanent) can lead to major changes in climatic conditions, which are reflected in distinct changes in timber moisture content. This results in a major increase in potential for damage due to e.g. crack initiation in glued-laminated timber elements. Accordingly, care should be taken during such interventions to realize a decelerated change of ambient climate.

4.3 Recommendations for best-practice

Potential measures to avoid fast decrease or increase of timber moisture content include adjusting the heating system so as not to reduce the relative humidity too fast and too strong. An artificial air humidification, e.g. in the form of evaporation basins is another possibility to damp the speed of drying of the structural timber elements. An alternative is a surface treatment, e.g. in the form of products which damp the moisture absorption and release in the first years of operation of the building (to counteract fast drying of newly installed elements in constant but dry climates). Such interventions should be carried out by expert personnel. The timber moisture content during production, transport and installation should not deviate too much from the expected equilibrium moisture content ($u \leq 12$ %).

In buildings featuring a substantial influence of the outdoor climate on the indoor climate, the application of insulation on the roof could help to dampen the strong changes of indoor climate and correspondingly the timber moisture gradients. Timber structures in areas exposed to direct sunlight (e.g. below skylights) or in the proximity of exhausts of the heating system, should be given attention with respect to potential crack initiation due to rapid drying after a period of increased humidity. Protective, exchangeable covering in the form of panel materials seem to be one feasible measure. This possibility is being investigated and measured in a separate research project carried out by the authors in collaboration with the Studiengemeinschaft Holzleimbau e.V. In ice-rinks, the largest change in the building climate and timber moisture gradient resulted from the ice preparation after the summer break. By air-conditioning the buildings, this effect can be significantly dampened. In riding rinks, the combination of cold air and humidity introduced by the sprinklers, frequently results in condensation. In order to reduce this effect during the cold season the sprinklers should only be used when it is absolutely necessary for the equestrian sport.

The findings presented imply that designers should be given more information and guidance on how to treat the subject of timber moisture content during construction, use, temporary interventions and change of use of their specific building. A potential implementation of the conclusions presented would be to include such information in textbooks or commented versions of codes, highlighting the benefits of using timber elements which feature a moisture content mirroring the expected average moisture content. To increase the awareness towards specific climates it should be considered to include examples of classification of buildings of specific use into Service Classes (e.g. riding rinks, ice-skating halls) in textbooks or commented versions of codes. At the same time it should be stated that the expected average moisture content is to be determined individually for each building. Another important objective is to increase awareness towards dry climates. It would be worthwhile to consider including a note in the code stating that the average moisture content of softwoods in heated and insulated buildings (Service Class 1) will in most cases be below 10 %.

4.4 Outlook

Currently measurements are continued in 10 of the 21 buildings featuring seasonally varying climate. For this, the measurement equipment was upgraded in order to take additional measurements of the temperature within the cross-section. These measurements shall be used to verify the approach to calculate the material temperature in the different depths on the basis of the measured surface temperatures. The continued measurements shall also help to answer the question whether the outside climate during the first year of measurement is representative for the regular climate at the location of the buildings.

References

[1] Blaß, H.-J., Frese, M., Schadensanalyse von Hallentragwerken aus Holz, Band 16 der Reihe Karlsruher Berichte zum Ingenieurholzbau, KIT Scientific Publishing, Karlsruhe, 2010

[2] Frühwald, E., Serrano, E., Toratti, T., Emilsson, A., Thelandersson, S., Design of safe timber structures – How can we learn from structural failures in concrete, steel and timber?, Report TVBK-3053, Div. of Struct. Eng, Lund University, 2007

[3] Dietsch, P., Winter, S., Assessment of the structural reliability of all wide span timber structures under the responsibility of the city of Munich, Proceedings 33rd IABSE Symposium, Bangkok, Thailand, September 9-11 2009

[4] Dietsch, P., Einsatz und Berechnung von Schubverstärkungen für Brettschichtholzbauteile, Dissertation, Technische Universität München, 2012.

[5] EN 1995-1-1:2004, Eurocode 5: Design of Timber Structures – Part 1-1: General – Common rules and rules for buildings, European Committee for Standardization CEN, Brussels, Belgium, 2004.

[6] Blaß, H., J., Ehlbeck, J., Kreuzinger, H., Steck, G., Erläuterungen zu DIN 1052:2004-08, Bruderverlag, Karlsruhe, 2004

[7] Meierhofer, U., & Sell, J. Physikalische Vorgänge in wetterbeanspruchten Holzbauteilen – 2. und 3. Mitteilung, Holz als Roh- und Werkstoff, 37 (6+12), 1979, pp. 227-234 and 447-454

[8] Krabbe, E., Neuhaus, H., Über Konstruktion, Klima und Holzfeuchtigkeit eines Hallenbades, Bauen mit Holz, 91 (4), 1989, pp. 214-217

[9] Koponen, S., Puurakenteiden kosteudenhallinta rakentamisessa, TKK-TRT Report 1-0502, Helsinki, 2002

[10] Evans, F., Kleppe, O., Dyken, T., Monitoring of Timber Bridges in Norway - Results, Report Norsk Treteknisk Institutt, Oslo, 2007

[11] Feldmeier, F., Ergebnisse und Schlussfolgerungen aus den Felduntersuchung einer Eissporthalle, Tagungsband Ingenieurholzbau - Karlsruher Tage, 2007, pp. 98-104

[12] Brischke, C., Rapp, A. O., Untersuchung des langfristigen Holzfeuchteverlaufs an ausgewählten Bauteilen der Fußgängerbrücke in Essing, Arbeitsbericht der Bundesforschungsanstalt für Forst- und Holzwirtschaft, Hamburg, 2007

[13] Marquardt, H., Mainka, G.-W., Tauwasserausfall in Eissporthallen, Bauphysik, 30 (2), 2008, pp. 91-101

[14] Niemz, P., Gereke, T., Auswirkungen kurz- und langzeitiger Luftfeuchteschwankungen auf die Holzfeuchte und die Eigenschaften von Holz, Bauphysik, 31 (6), 2009, pp. 380-385

[15] Fragiacomo, M., Fortino, S., Tononi, D., Usardi, I., Toratti, T., Moisture-induced stresses perpendicular to grain in cross-sections of timber members exposed to different climates, Engineering Structures, 33 (11), 2011, pp. 3071–3078

[16] Fortino, S., Genoese, A, Genoese, A., Nunes, L., Palma, P., Numerical modelling of the hygro-thermal response of timber bridges during their service life: A monitoring case-study, Construction and Building Materials, 47, 2013, pp. 1225-1234

[17] Klaiber, V., Dimensionsstabilität von Fichtenholz unter dem Einfluss verschiedener Varianten der Rundholzbereitstellung und Schnittholztrocknung – Untersucht an Beispiel eines Fichtenreinbestandes in Mittelgebirgslage, Dissertation, Albert-Ludwigs-Universität, Freiburg im Breisgau, 2003.

[18] EN 14080, Timber structures – Glued laminated timber and glued solid timber – Requirements, CEN, Brussels, 2013

[19] Gamper, A., Dietsch, P., Merk, M., Winter, S., Building Climate – Long-term measurements to determine the effect on the moisture gradient in timber structures, Final Report, Lehrstuhl für Holzbau und Baukonstruktion, Technische Universität München, 2012

[20] Ressel, J., B., Fundamentals of wood moisture content measurement, Course notes, COST E53 Training School "Methods for measuring of moisture content and assessment of timber quality", BFH, Hamburg, October 17 – 19 2006

[21] Dietsch, P., Franke, S., Franke, B., Gamper, A., Methods to determine wood moisture content and their applicability in monitoring concepts, Journal of Civil Structural Health Monitoring; DOI 10.1007/s13349-014-0082-7

[22] Dietsch, P., Franke, S., Franke, B., Gamper, A., Monitoring building climate and timber moisture gradient in large-span timber structures, Journal of Civil Structural Health Monitoring, DOI 10.1007/s13349-014-0083-6

[23] Euler, L., Institutiones Calculi differentialis, Berlin, 1755

[24] Kollmann, F., Coté, W., A., Principles of Wood Science and Technology I: Solid Wood, Springer, Berlin, 1968

[25] Fortuin, G., Anwendung mathematischer Modelle zur Beschreibung der technischen Konvektionstrocknung von Schnittholz, Dissertation, Universität Hamburg, 2003

[26] Keylwerth, R., Noack, D., Über den Einfluß höherer Temperaturen auf die elektrische Holzfeuchtigkeitsmessung nach dem Widerstandsprinzip, Holz als Roh- und Werkstoff, 14 (5), 1956, pp. 162- 172.

[27] Hailwood, A. J., Horrobin, S., Absorption of water by polymers: analysis in terms of a simple model, Transactions of the Faraday Society, Vol. 42b, 1946, pp. 84-92

[28] Simpson, W. T., (1973), Predicting equilibrium moisture content of wood by mathematical models, Wood and Fiber Science, 5 (1), 1973, pp. 41-48

Ultrasonic Echo Methods for Structural Timber

Martin Krause[1], Klaus Mayer[2], Ute Effner[3]

Summary

Ultrasonic echo technique with shear waves in the frequency range of 50 kHz is one of the promising methods for assessment of structural timber and is already frequently applied for this purpose. The article summarizes the state of the art of applying echo measurement via linear measurements as well as the development and first results for an imaging method for this purpose. The highly anisotropic properties of the ultrasonic velocity have to be considered. 3D ultrasonic imaging results are described for the example of a pedestrian bridge made from Siberian larch.

Key words: Ultrasonic echo, SAFT reconstruction calculation, Glued-laminated timber (glulam), Footbridge

1 Introduction

There are two approaches for applying ultrasonic echo techniques for timber. Both are based on techniques, which have already been successfully applied for concrete elements. Low frequency ultrasonic technology with transmitting/receiver probes is applied in the low frequency range around 50 kHz, working which dry coupling point contact transducers without need of coupling agent. With this technology two measuring principles are usually applied:

a) Measuring the back wall echo of the timber elements along lines and evaluating the intensity of the back wall echo. Areas giving back a stable distinct back wall echo are classified as sound, whereas areas with weak or vanishing back wall echo are probably damaged and need to be further investigated in detail.

b) Measuring and recording the ultrasonic data of equidistant measuring lines or 2D-measuring fields and evaluating them by means of reconstruction techniques (**S**ynthetic **A**perture **F**ocusing **T**echniques; SAFT). In analogy to similar techniques for concrete elements, internal objects (scatterers) and defects may be imaged and represented in 2D- or 3D representations, respectively.

The main difference applying these techniques for concrete and timber are the strong acoustic anisotropic properties of timber, which make it necessary to apply ultrasonic shear wave pulses having a polarisation axis parallel to the longitudinal axis (fibre orientation), when working with method a). This method is applied successfully since ca. 2005 for timber elements. Some examples are summarized in this contribution.

The imaging approach mentioned in point b) is still under research and will be further developed with the aim to investigate and classify localised defects and mounting parts. First applications will be briefly described referring to literature.

2 Equipment and presentation of results

Most ultrasonic measurements at timber are carried out with dry contact transducers. This has the advantage that the surface is not contaminated with grease ore other coupling agents. There are two types of commercially available measurement devices frequently used for this task:

- Transmitter/Receiver sensor head made from 24 shear wave point-contact transducers, working with a handheld interface [1]. In figure 1 the sensor head is shown mounted in an automated scanner [2]. Similar measuring equipment is recently available from other sources [3].
- Linear Array: For fast measuring of surface areas there is an innovative device, which has 12 transducer modules [4]. Each module consists of 4 parallel switched dry contact shear wave transducers (50 kHz as described above). The polarization axis is orientated orthogonally to the longitudinal axis. In the meantime similar techniques are also available from other sources [5].

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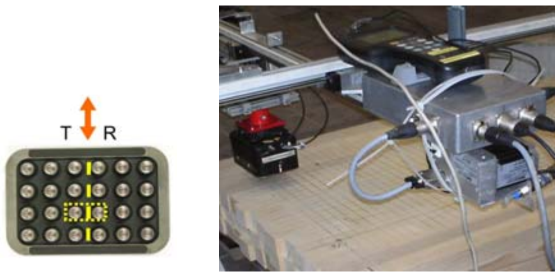

Figure 1: Automated scanner with shear wave point contact transducer (transducer driven in the 1 transmitter- 1 receiver mode).

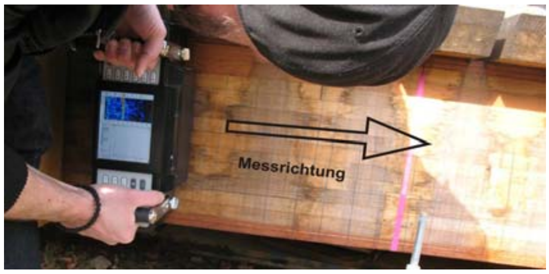

Figure 2: Example of application with linear array imaging at a timber footbridge(direction of movement ~ polarization ~ fiber axis L.

These devices can be used as handheld equipment with evaluation software supplied by the company. In combining many equidistant measurement points in lines and measuring areas, the results may be represented in different graphs. They are usually named as such:

- **A-scan**: Amplitude of the measured signal vs. time (output voltage proportional to the sonic pressure) vs. time. Three types of A-scans are used: HF-signal (not rectified), rectified and calculated envelope function. Figure 3 left shows the HF Signal of an echo measurement at a polyurethane specimen.
- **B-scan**: Amplitude of the time signal along the measuring (or a selected) axis showing the depth information of the reflection (alternatively named: longitudinal or cross section of ultrasonic amplitude). It consists of a line (x-axis) of several A-scans and corresponds to the radargram (radar) or sonogram (geophysical experiments). It is usually shown in a false colour (or grey scale) representation. The y-axis represents either the time of the receiver after pulse excitation or the depth after calibrating the sound velocity (figure 3b).
- **C-scan**: Amplitude of the measured time signal along a plane parallel to the surface at a specific depth (or time, respectively) showing the depth information. Alternatively named: depth section of ultrasonic amplitude. It corresponds to the “time slices” in radar results.

Since those transducers enable measurements of large building areas without coupling agent, there are numerous research and development activities in order to investigate large areas with automated equipment (building scanners). Originally developed for concrete elements, these techniques were applied for timber investigation since 2007 [6].

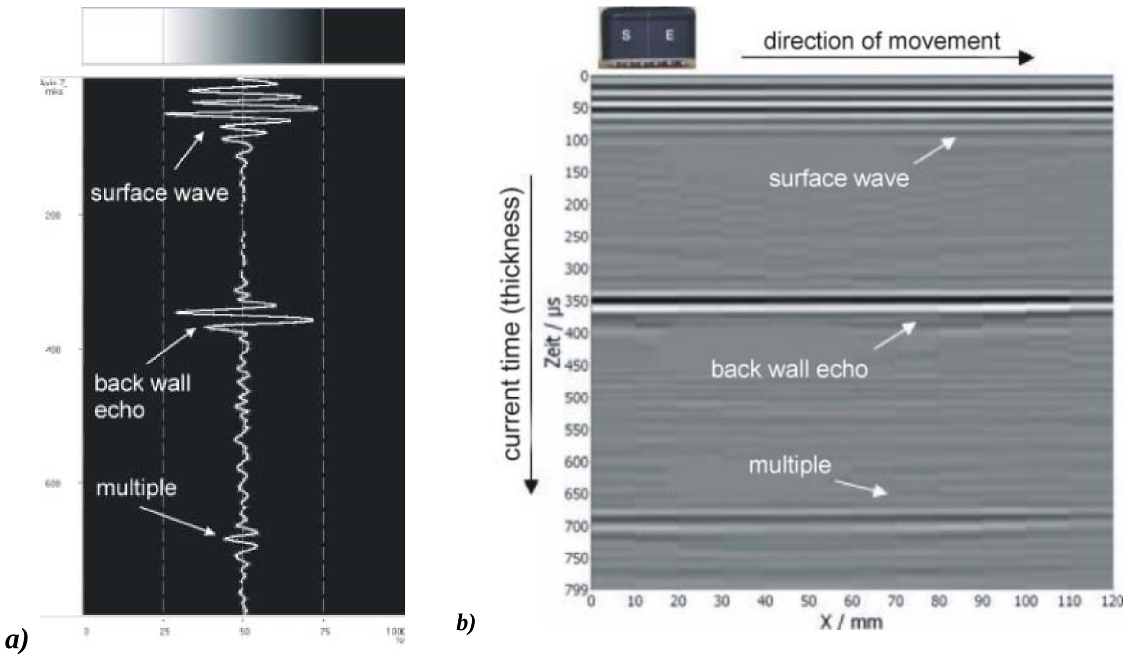

Figure 3: Principle for controlling the homogeneity of a timber beam (idealized images measured at a polyurethane specimen). A-scan with back wall echo and 1^{st} multiple (colour bar on top (black/ white) (left).
B-scan measured stepwise with T/R-Probe (50 kHz shear waves) with back wall echo and multiple of the calibration specimen (right) (from [6]).

Additionally to the simple visualisation of the data, there is an imaging technique, which allows representing the shape of the reflecting interfaces measured from different directions. In order to do so the data measured along lines and measuring surfaces are combined to one data set and evaluated with fast reconstruction calculation. The principle is named **S**ynthetic **A**perture **F**ocusing **T**echnique, SAFT). Simply described it is based on a time corrected superposition of the measured data under consideration of the location of the currently active transmitter and receiver location for each data point. Originating from geophysics and applications for NDT of steel elements, this technique is developed and applied in civil engineering since 1995 mainly for concrete elements [7]. Developments for (anisotropic) timber began in 2010 [8], [9].

3 Results and discussion

Shadowing of Back wall echo

Experimental work applying ultrasonic waves on timber show that shear waves, having the polarisation axis parallel to the fibres, are weakly attenuated. Beginning in 2002 this principle was developed to be used for non-destructive testing of timber beams [10], [11]. It is based on measuring the back wall echo by means of point contact transducers and representing it in a B-scan as showed in figure 3. Contiguous areas showing a weak or vanishing back wall echo are then classified as suspicious for damage (mainly fungi or insects attack).

There are numerous applications of this method in practice, including automated application [6]. Often the method is applied in combination with drilling resistance, because in this way suspicious areas can be checked effectively [12], [13], [14], [15].

Research projects on gluelam compounds show that missing adhesive may be directly measured via echo imaging or shadowing of back wall echo [16]. Otherwise it has to be stated that missing or insufficient adhesive may be detected by echo measurement or weak back wall echo. It has to be noted that an intense back wall echo will not assure good gluing conditions in any case [17].

Imaging approach

In order to improve reconstruction calculation for timber, it is indispensable to take into account the anisotropic properties of elastic wave propagation, thoroughly described by e.g. [18], [19]. Since wave propagation in timber is even more complicated because of the curvature induced by growing and indicated by the annual rings, the initial development and tests were done with artificial data sets calculated with EFIT modelling (Elastodynamic Finite Integration Technique) [20], [21], [22]. Experimental work and further developments of evaluation tools then show that this kind of ultrasonic imaging is feasible under quasi homogeneous anisotropic conditions. This means that the curvature of the annual rings should be negligible for the timber elements in focus. An example is shown in figure 4, specimens for investigation are glued from those elements (example see figure 5). They may be measured with automated equipment as demonstrated in figure 1. Extensive research work was done in developing measuring and imaging procedures for timber [23]. In the following some results are briefly summarized.

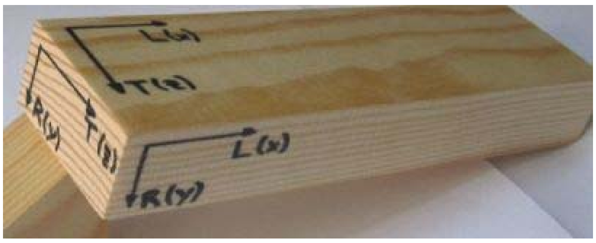

Figure 4: Small specimen as used for producing specimens of glue laminated timber with quasi homogeneous anisotropic conditions showing only negligible curvature of annual rings.

Figure 5: View of typical specimen with symmetry axis (pine)

Applying scanner tools as described in figure 1 combined with further development of reconstruction evaluation has led to a substantial progress in imaging of scatterers in timber during the last years. Main results in this field are [23][25]:

- Imaging of artificial reflectors inserted in the rear side of pine specimens. This is only feasible when additionally to body waves head waves are taken into account, which are produced by the interchange of body waves and surface waves. In addition the surface waves have to be numerically suppressed for the evaluation process

- Improving the accuracy of results in developing an interactive evaluation procedure in the reconstruction software. This means that the anisotropic velocity profile of the timber may be adapted to the current individual conditions of the investigated specimen interactively. Thus the software is named Inter-SAFT [24]
- Imaging of flat bottom reflectors in spruce specimen (thickness 90 mm).

There are ongoing research activities mainly focussed on the dependence of resolution on penetration depth of ultrasonic waves and how to select the adequate wave modes for achieving the best imaging results.

Application on site

The newly developed imaging approach was tested at a pedestrian bridge constructed from Siberian larch gluelam. The measurement was carried out in 2 configurations: a) a scanning system as depicted in figure 1 and b) applying the linear array system presented in figure 2.

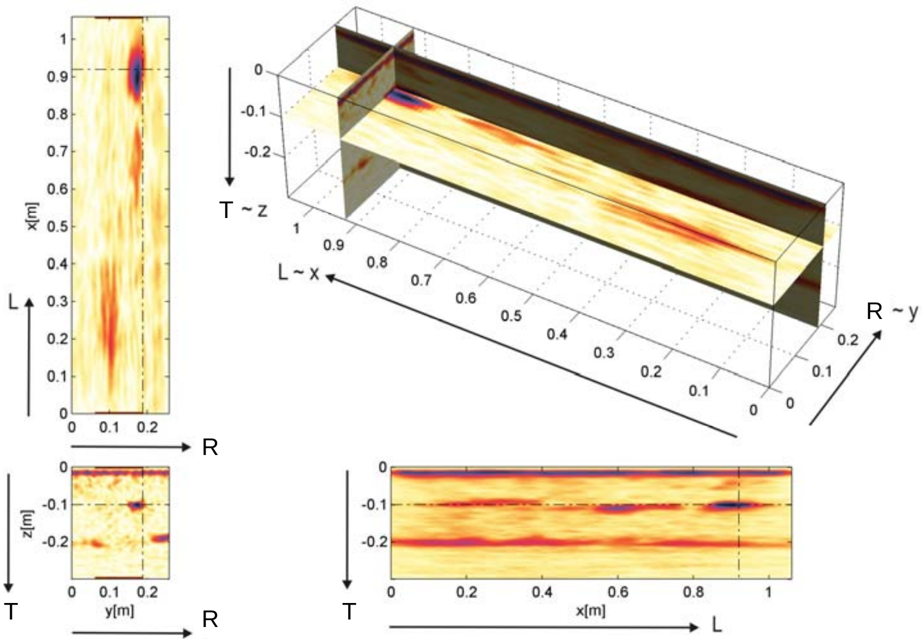

*Figure 6: Application of ultrasonic imaging at an foot bridge made from Siberian Larch gluelam: 3D representation of localized scatterers from shear wave echo measurement with automated scanner, evaluated by means of Inter-SAFT (Axes of symmetry: **L:** Longitudinal axis / fibre orientation; **R**: radial; **T**: tangential; compare figure 4)*

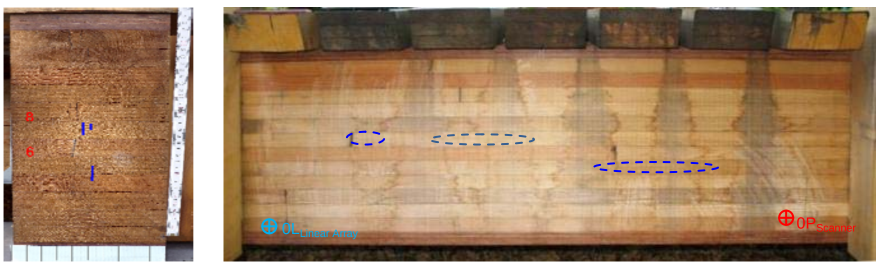

Figure 7: Segment of the footbridge measured by scanning ultrasonic echo. The locations of the internal reflectors depicted in figure 6 are marked on the view of the building surfaces.

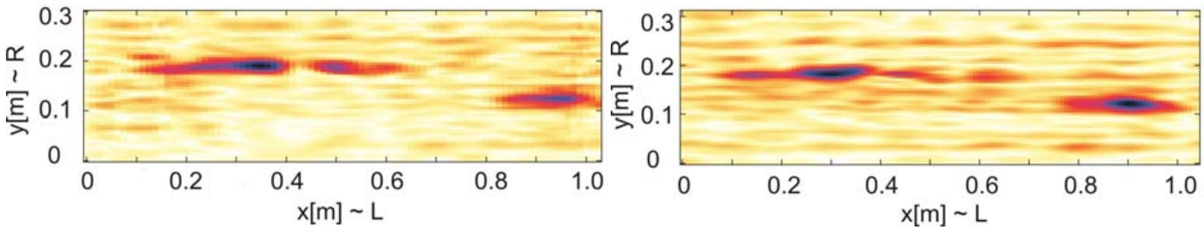

Figure 8: Result measured at the footbridge with Linear Array as depicted in figure 2, presentation of the internal reflectors as SAFT-B-scan from Inter-SAFT evaluation. Repeated measurements after interval of measurements of 7 months.

For the SAFT reconstruction the velocities were adapted in a so-called elliptical approach using the interactive features of the software system Inter-SAFT [24]. A three dimensional overview of the imaging results obtained by the scanning system are depicted in figure 6, more details can be found in [25].

For fast practical application the linear array is very useful as already presented in figure 2. Several sensor head positions may be combined to one dataset and are then evaluated by the Inter-SAFT procedure. Figure 8 shows 2 SAFT-C-scans (depth sections in a depth of z = 103 mm) of the measured area. They are measured in a time interval of 7 months showing that the imaged reflectors are relatively stable in time. These are probably vertical cracks in the lamellas, which are not relevant for static considerations in the present case. But this experiment demonstrates the capability of the measuring and evaluation system for clear ultrasonic imaging in glulam

4 Conclusion and future work

Assessment of structural timber applying ultrasonic echo measurement and analysing the shadowing of the back wall echo is a useful method for practical application. In order to go further in analysing the inner state of timber the encouraging result of a research work is that artificial reflectors in wood can directly be imaged by low frequency ultrasonic echo measurements. This is possible with interactive reconstruction software based on the principle of synthetic aperture focusing technique considering the anisotropic properties of the material. The method was verified by using synthetic data calculated from modelling of wave propagation. For practical applications it is not necessary to use exclusively anisotropic velocities parameters given in literature or from reference measurements with similar material, but they can be adapted interactively during evaluation.

In glue laminated timber made from spruce the best results for artificial reflectors were obtained with pressure waves. In a pedestrian bridge made from Siberian larch shear waves polarized in fibre direction enables imaging of inner reflectors, which were probably cracks.

Further research activities are necessary in order to develop a non-destructive imaging system for glue laminated timber based on ultrasonic methods. The first step could be to enhance the imaging depth for gluelam made from spruce. Afterwards the measuring technique and evaluation software have to be adapted for practical relevant testing tasks. Examples are: locally concentrated fungi attack, knot content and glued bolts.

For fast application on site the use of a linear ultrasonic array with subsequent 3D reconstruction calculation seems to be very promising. More experiments at real objects followed by verification may help to motivate practical application of this system.

5 Acknowledgement

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References

[1] Kozlov V. N., Samokrutov A. A., Shevaldykin V. G. (2006) Ultrasonic Equipment for Evaluation of Concrete Structures Based on Transducers with Dry Point Contact. In: Al-Quadi, I. and G. Washer (eds.); Proceedings of the NDE Conference on Civil Engineering, 14.-18. August 2006, St. Louis, MO, USA, 496-498 (Company ACSYS, Russia).

[2] Krause M., Effner U., Müller S., Nowak T., Mayer K., Chinta P. K. (2013) 3D-SAFT Imaging von Streuern in Brettschichtholz. In: Tagungsband der DGZfP Jahrestagung, ZfP in Forschung, Entwicklung und Anwendung. Dresden, 06.-08.05.2013, DGZfP-Berichtsband BB 141-CD, Poster 8.

[3] Company Proceq, Switzerland.

[4] Bishko A. V., Samakrutov A. A., Shevaldykin V. G. (2008) Ultrasonic echo-pulse tomography of concrete using shear waves low-frequency phased antenna arrays. In: Proceedings of the 17th World Conference on Nondestructive Testing, 25.-28.10.2008, Shanghai, China, 9 pages; (Company ACSYS, Russia).

[5] Schickert M., Hillger W. (2010) Automated Ultrasonic Scanning and Imaging System. ndt.net, 10 pages http://www.ndt.net/article/ecndt2010/reports/1_14_05.pdf.

[6] Maack S., Krause M. (2008) Diagnostic Investigations of Wooden Structures using Ultrasonic-Echo Technique. In: Proceedings of the 1st International RILEM Conference, on Site Assessment of Concrete, Masonry and Timber Structures (SACoMaTiS), September 1-2, 2008, Varenna, Italy (Proceedings - PRO 59), 1081-1090.

[7] Schickert M., Krause M., Müller W. (2003) Ultrasonic Imaging of Concrete Elements Using Reconstruction by Synthetic Aperture Focusing Technique. Journal of Materials in Civil Engineering (JMCE), ASCE Vol. 15 (2003) 3: 235-246.

[8] Mayer K., Chinta P. K., Langenberg K.-J., Krause M. (2012) Ultrasonic Imaging of Defects in Known Anisotropic and Inhomogeneous Structures with Fast Synthetic Aperture Methods. In: Proceedings of the 18th World Conference on Non-Destructive Testing, Durban, South Africa, 16.-20.04.2012, CD-ROM, 10 pages.

[9] Chinta P. K., Mayer K., Krause M. (2010) Ultraschallmodellierung und SAFT-Rekonstruktion von Fehlstellen in Holzbauteilen. In: Berichtsband der DGZfP-Jahrestagung, Erfurt, 10.-12. Mai 2010, BB 122-CD, Poster 59, 8 Seiten.

[10] Hasenstab A., Krause M., Hillemeier B., Rieck C. (2006) Ultraschallecho-Messungen an Holz. Holz als Roh- und Werkstoff 64, 475-481.

[11] Hasenstab A. (2006) Integritätsprüfung von Holz mit dem zerstörungsfreien Ultraschallechoverfahren. Bundesanstalt für Materialforschung und -prüfung (BAM) (Hrsg.); BAM-Dissertationsreihe, Band 16, Berlin (2006) 190 Seiten.

[12] Frühwald K., Peterson L., Hasenstab A. (2012) Prüfverfahren zur Begutachtung der Materialeigenschaften von Holztragwerken. Bauphysik-Kalender 2012, Berlin: Ernst und Sohn, Kap. B Materialtechnische Grundlagen, Abschn. B1, 105-155.

[13] Tannert T., Kasal B., Anthony R. (2010) RILEM TC 215 In-situ assessment of structural timber: Report on activities and application of assessment methods. In: Proceedings from the 2010 World Conference on Timber Engineering, Riva del Garda. Italy. June 2010.

[14] Hasenstab A., Frühwald K. (2013) Holzbauwerke mit Ultraschallecho und Bohrwiderstand geprüft, in: Technische Akademie Esslingen, editor, Erhaltung von Bauwerken, 3. Kolloquium 22. u.23. Januar 2013, CD-ROM, TAE, Ostfildern, Germany.

[15] Kasal B., Lear G., Tannert T. (2011) Stress Waves. Chapter in: RILEM State-of-the-Art Reports, “In-situ assessment of structural timber”, B. Kasal and T. Tannert (eds.) Springer, Vol. 7: 5-24.

[16] Aicher S., Dill-Langer G. (2008) Non-destructive detection of glue line defects in glued laminated timber. In: Proceedings of the 10th World Conference on Timber Engineering, 02.-05.06.2008, Miyazaki, Japan, 8 pages.

[17] MFPA Stuttgart (2010) Qualitätsprüfung von Klebefugen in Brettschichtholz mittels Ultraschall. Schlussbericht zum AIF-Forschungsprojekt, AIF-No. 15585 N.

[18] Musgrave M. (1970) Crystal Acoustics, Introduction to the study of elastic waves and vibrations in crystals. Holden-Day, San Francisco, USA.

[19] Bucur V. (1995) Acoustics of Wood. CRC Press, Boca Raton, Now York, Tokyo, 284 pages.

[20] Marklein R. (2002) The Finite Integration Technique as a general tool to compute acoustic, electromagnetic, elastodynamic and coupled wave fields. Stone W. (ed.); Review of Radio science 1999-2002, New York: IEEE Press.

[21] Chinta P. K., Mayer K., Krause M. (2010) Ultraschallmodellierung und SAFT-Rekonstruktion von Fehlstellen in Holzbauteilen. In: Berichtsband der DGZfP-Jahrestagung, Erfurt, 10.-12. Mai 2010, BB 122-CD, Poster 59, 8 Seiten.

[22] Chinta P. K. (2013) Ultrasonic Nondestructive Testing of Isotropic and Anisotropic Media: Modeling and Imaging. Dissertation, Universität Kassel, 172 pages.

[23] Krause M., Effner U., Milmann, B., Müller S., Nowak T., Borchardt K., Mayer K., Chinta P. K., Ballier G. (2013) Hochgenaue Strukturerkennung von Holzbauteilen mit 3D-Ultraschall. Forschungsinitiative Zukunft Bau, Band F 2849, Fraunhofer IRB Verlag, 105 Seiten.

[24] Mayer K. (2008) Softwarepaket Inter-SAFT zur bildgebenden Auswertung von Ultraschallechomessungen an Holzbauteilen, by order of BAM.

[25] Krause M., Mayer K., Chinta P. K., Effner U. (2013) Ultrasonic Imaging of Defects in Building Elements Made from Timber. Advanced Materials Research 778:312-320, Trans Tech Publications, Switzerland.

Structural behaviour of self-tapping screws - Theory

Robert Jockwer[1]

Summary

Self-tapping screws are modern fasteners offering high performance and flexibility for various applications. They exhibit high resistance when loaded in axial direction. The load-carrying capacity of screws depends on their geometrical and material properties which are optimized by the manufacturers for specific applications. This paper gives an overview about the relevant properties of screws and summarizes the relevant design equations for the application of self-tapping screws as connecting or reinforcing elements.

Key words: Self-tapping screw, Fastener, Connection, Reinforcement

1 Introduction

Self-tapping screws are modern fasteners that originated in lag screws, which were only rarely used in traditional timber structures. Lag screws require a high installation effort because their holes have to be predrilled in two stages and are only of relevant in situations when loaded in the axial direction. Other dowel type fasteners like e.g. nails can be installed in an automatized process and are therefore much more attractive for a fast and economic design of connections. The drawback of the high installation effort was eliminated with the development of self-tapping screws. Current screws do not require predrilling of the holes and their shape can be optimized for various applications. The thread of self-tapping screws is forged or rolled into the wire made of high strength steel instead of being cut from the wire, as in the case of lag screws. Due to their production process self-tapping screws can be manufactured with a much longer length.

Besides the application as connecting elements, self-tapping screws are often used as reinforcing elements. Due to their high potential for carrying load in axial direction they aree are adequate for reinforcing regions of tension and compression stresses perpendicular to the grain and, when installed at an angle to the grain, also for regions of high shear stresses. The availability of fully-threaded self-tapping screws of longer length makes it possible to reinforce the beam regions where the highest loads occur.

2 Geometrical parameters and materials

The most important characteristic properties and differences between lag screws and self-tapping screws are summarized in Figure 1. The geometry of the screw is defined by its outer thread diameter d, the inner core diameter of the threaded part d_1 or d_{core} and for screws with a partially unthreaded shank the shank diameter d_s. Screws with two threaded part have commonly different steepness in the two threads. The thread at the side of the screw head has often a lower steepness and a larger diameter than the thread at the side of the screw tip. The larger diameter of the thread at side of the screw head allows a good load transfer of the head-end thread and the different steepness leads to a tightening of the connected members during installation of the screw. The total length l of the screw has to be reduced for the design by the length of the screw head and the screw tip. The length of the threaded part l_g can be used for the transfer of axial forces between the screw thread and timber. Along the length of the unthreaded part of the screw l_s, no load transfer is possible. However, this part allows for the creation of an axial tightening force between the threaded part and the screw head, or between two threaded parts.

Beside these geometric parameters, the load carrying capacity of screws is influenced by the yield moment M_y, the embedment strength of the timber f_h and by the withdrawal strength f_{ax} of the thread. The properties of screws are specified in EN 14592 (2008) [9] where it is distinguished between the different production methods for lag screws and self-tapping screws. The geometrical parameters of lag screws are often produced according to DIN 7998 (1975) [5]. The ratio of between inner and outer thread diameter of lag screw is specified with $d_1 / d = 0.7$ and 0.75, respectively. Self-tapping screws offer a much wider diversity of geometries. EN 14592 (2008) [9] is valid for ratios $d_1 / d = 0.6 - 0.9$. Commonly the ratio is in the range of $d_1 / d = 0.7$ for self-tapping screws.

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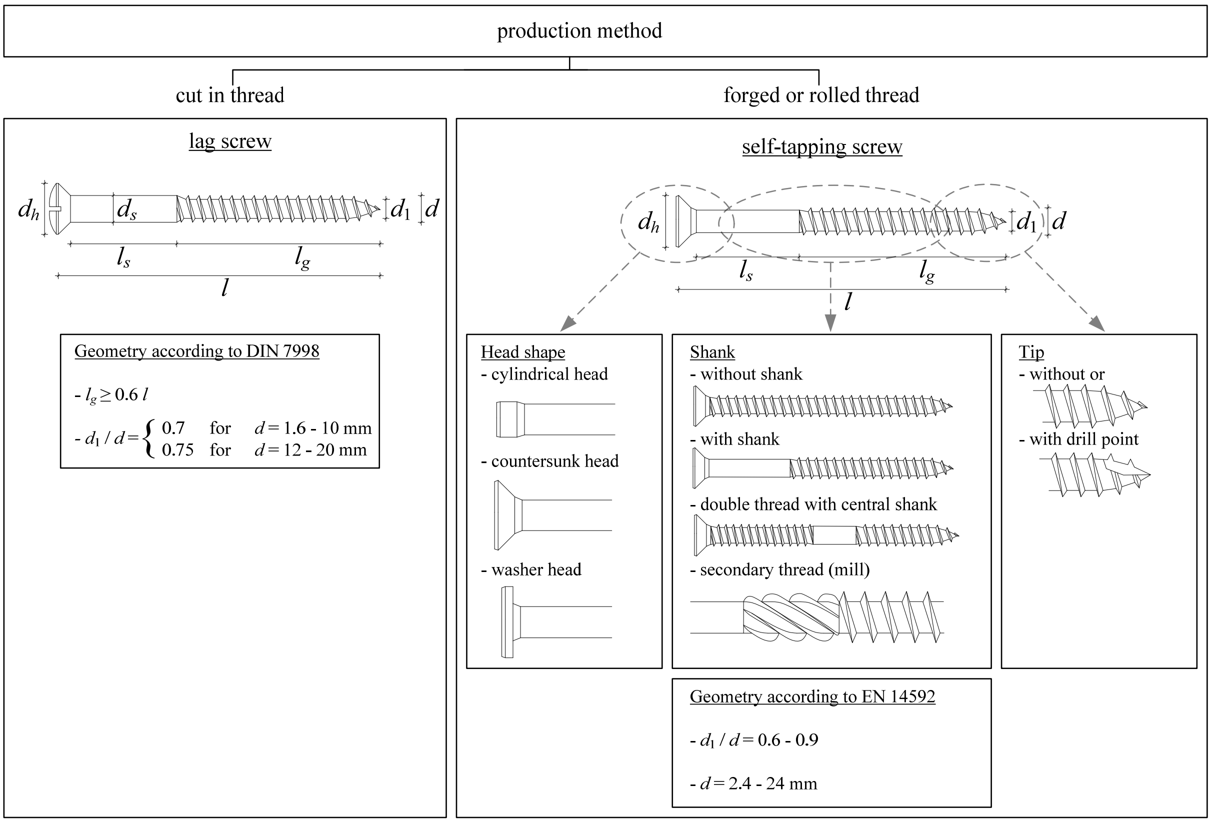

Figure 1: Geometric properties of screws

The steel quality of the screw is important with regard to it's bending and axial resistance, but also maximal length. This length is limited by the maximal torsional resistance of the screw, especially long fully-threaded screws, which are installed without predrilling of the holes, require high torsional strength. High-carbon steel with tensile strengths $f_{u,k} \geq 800$ N/mm^2 are often used for self-tapping screws. The application of a surface coating reduces the friction during insertion of the screw. Lag screws are often made of low-carbon steel, with tensile strength in the range of $f_{u,k} \approx 600$ N/mm^2. The use of high-carbon steel for self-tapping screws increase the risk of brittle instead of ductile failure.

The flexibility in the production of self-tapping screws makes it possible to optimize the different parts of the screw with regard to their specific application needs. A drill tip is often applied in order to avoid the need of predrilling the hole. However, depending on the diameter of the screw and the density of the timber, predrilling can still be necessary. Hence, naming all the screws as "self-tapping" can be misleading. The shape of the screw's head can be adapted to the requirements of the pull-though resistance, e.g. milled ribs at the head can facilitate the countersinking of the head. Screws with a partially unthreaded shank often have a secondary milling thread between the threaded and the unthreaded parts, to widen the diameter of the screw hole and to reduce the friction in the unthreaded part of the screw.

When describing the resistance of screws it is distinguished between axial and lateral load-carrying capacities. The structural behavior in the lateral direction is similar to other dowel-type fasteners, whereas the structural behavior in axial direction is specific for screws. In the following chapters the resistances in both directions are explained.

3 Axial resistances

The failure modes of a screw loaded in the axial direction are illustrated in Figure 2: Withdrawal failure of the threaded part of the screw, b) head pull-through failure of the screw, c) tensile failure of the cross-section of the screw and d) buckling failure of the screw loaded in compression. The first three failure modes a) – c) are relevant in connections and when the screws are used as reinforcement. The buckling failure mode is relevant for screws in applications such as reinforcement for local concentrated loads e.g. at supports. The design of screws with regard to these axial failure loads are discussed in the following sections.

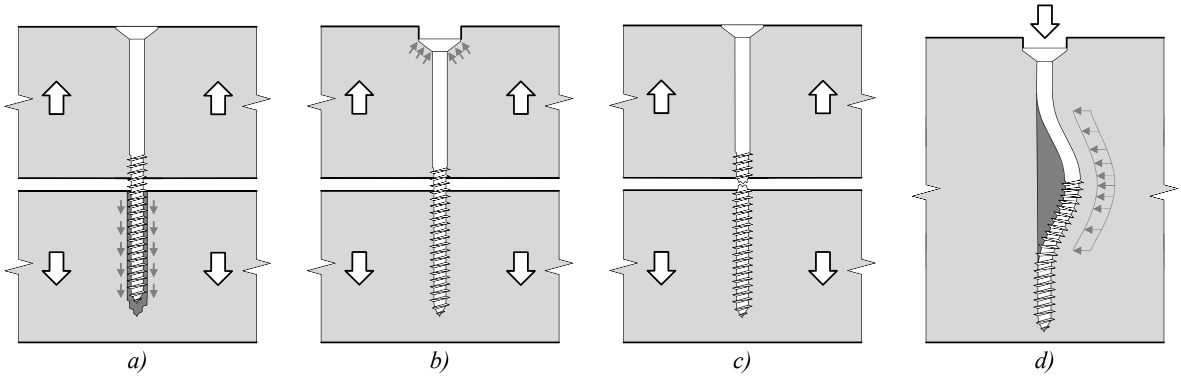

Figure 2: Failure modes of axially loaded self-tapping screws: a) withdrawal, b) head pull through, c) tensile and d) buckling failure

3.1 Withdrawal resistance

The resistance against withdrawal arises from the anchorage of the screw's thread in the timber. The withdrawal strength can be determined in experiments according to EN 1382 (1999) [6] from the maximum withdrawal force divided by the projection of the effective cross-section of the threaded part of the screw in the timber. The use of the projection of the cross-section is the reason why the withdrawal strength is more precisely named withdrawal parameter, which cannot be directly connected to the shear strength in the surface of the threaded part of the screw. The characteristic value of the withdrawal strength $f_{ax,k}$ given in EN 1995-1-1 (2008) [8] was determined by Blaß et al. (2006) [3] in experiments on various different types of screws:

$$f_{ax,k} = 0.52\, d^{-0.5}\; l_{ef}^{-0.1}\; \rho_k^{0.8} \tag{1}$$

Where d is the outer diameter of the screw in mm, l_{ef} is the effective length of the threaded part in the screw in the timber in mm and ρ_k is the characteristic value of the density of the timber in kg/m^3.

Additional influences on the withdrawal strength of screws like moisture content of the timber, temperature or the predrilling of the screw was studied by Pirnbacher et al. (2009) [13]. Small angles γ between screw axis and grain direction influence considerably the withdrawal strength. Different models for the specification of the withdrawal strength have been studied by Frese et al. (2010) [10].

The characteristic value of the withdrawal resistance $F_{ax,k,Rk}$ of a screws with diameters between 6 mm $\leq d \leq$ 12 mm and $0.6 \leq d_1/d \leq 0.75$ can be calculated according to Equation (2) using the withdrawal strength in Equation (1). Additional parameters account for the impact of the effective number of screws in a connection ($n_{ef} = n^{0.9}$), the diameter of the screw ($k_d = \min\{d/8\ ;\ 1\}$) and the angle between screw axis and grain direction α.

$$F_{ax,k,Rk} = \frac{n_{ef}\; f_{ax,k}\; d\; l_{ef}\, k_d}{1.2 cos^2\alpha + sin^2\alpha} \tag{2}$$

For screws with other geometric properties, the withdrawal strength has to be determined according to EN 1382 [6] and it is accounted for the ratio between characteristic density in the design ρ_k and the density used in the tests ρ_a as follows: $(\rho_k / \rho_a)^{0.8}$.

3.2 Head pull-through resistance

The head pull-through resistance of the screw depends on the diameter and shape of the screw's head. The influence of the shape of the screw head is accounted for by the pull-through parameter f_{head}. This parameter is experimentally determined according to EN 1383 (1999) [7] from the ultimate pull-through force divided by the square of the diameter of the screw head d_h. The characteristic value of the pull-through resistance can be calculated from the characteristic value of the pull-through parameter $f_{head,k}$ (associated with a density ρ_a) and the characteristic value of the density ρ_k :

$$F_{ax,\alpha,Rk} = n_{ef}\; f_{head,k}\; d_h^2\, \left(\frac{\rho_k}{\rho_a}\right)^{0.8} \tag{3}$$

The pull-through resistance increases for large and flat screw heads. It can be increased by using special washer heads instead of countersunk heads or even placing washers under the screw's head. In cases where screws should be sunken, e.g. due to aesthetic reasons or for fire protection, cylindrical screw heads can be chosen, which have negligible pull-through resistance and thus can be screwed into the timber.

3.3 Tensile resistance of the screw

The tensile resistance of the screw can be relevant for long and slender screws. Head tear-off or tensile failure of the screw can occur when the effective threaded length and thus the withdrawal resistance are high. The tensile capacity f_{tens} of a screw can be determined in experiments similar to the head pull-through tests according to

EN 1383 (1999) [7] by using a steel plate instead of a timber member underneath the screw head. The test length should be long enough and contain the transition between threaded and shank part of the screw. The characteristic value of the tensile resistance can be calculated using the characteristic value of the tensile capacity as follows:

$$F_{t,Rk} = n_{ef}\ f_{tens,k} \tag{4}$$

An indication of the tensile capacity of a screw can be calculated from the tensile strength of the steel of the screw and the inner cross-section of the thread (thread root) of the screw. In Figure 3, the axial resistance of a screw as the minimum of withdrawal capacity and tensile capacity is shown in relation to the effective length. A tensile strength of $f_{u,k}$ = 800 N/mm^2 is used. It can be seen that at relative effective lengths $l_{ef} > 20 \cdot d$, the tensile resistance is limiting the axial resistance. Longer effective lengths do not increase the axial resistance any further, in this example, and are not economic.

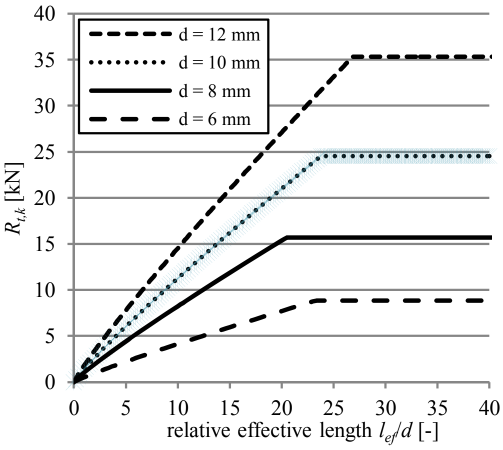

Figure 3:Maximum axial resistance of screws as the minimum of withdrawal and tensile strength in dependency of the diameter d and the relative effective length l_{ef}/d.

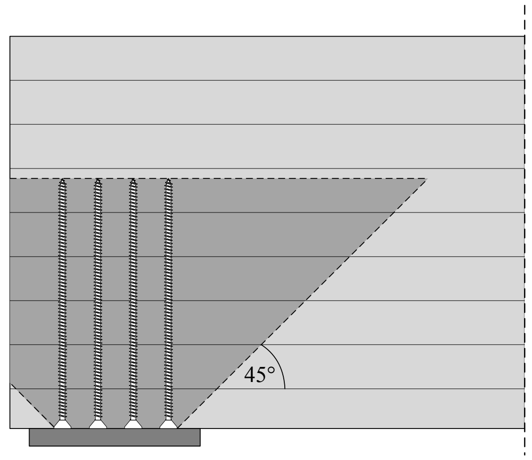

Figure 4: Assumed spreading of the compression perpendicular to grain stresses at the beam support

3.4 Buckling of the screw under compression load

The resistance of screws loaded in compression can be limited by withdrawal failure of the thread, head pull-through failure and in addition by instability leading to buckling. Slender and long screws loaded under compression are prone to fail in buckling. These screws are typically used for reinforcement against compression perpendicular to the grain e.g. at supports. A design procedure for the buckling failure mode of screws is not given in EN 1995-1-1 (2008) [8]. Bejtka and Blaß (2006) [2] studied the buckling with continuous elastic foundation more detailed with regard to the corresponding elastic foundation and support and the buckling modes. The proposed design procedure is similar to the buckling design of columns. The characteristic value axial, plastic resistance of the screw in compression is reduced by the instability factor k_c:

$$F_{ax,c,Rk} = k_c f_{y,k}\, \pi \frac{d_1^2}{4} \tag{5}$$

Where $f_{y,k}$ is the characteristic value of the yield strength of the screw, and d_1 is the thread root diameter.

The instability factors k_c and k can be calculated as follows:

$$k_c = \frac{1}{k + \sqrt{k^2 - \lambda_{rel}^2}} \leq 1 \tag{6}$$

and

$$k = 0.5[1 + 0.49(\lambda_{rel} - 0.2) + \lambda_{rel}^2] \tag{7}$$

The relative slenderness ratio λ_{rel} is calculated from the plastic resistance $N_{pl,k}$ and the buckling load of the screw $N_{ki,k}$ taking into account the elastic foundation perpendicular to the screw axis.

$$\lambda_{rel} = \sqrt{\frac{N_{pl.k}}{N_{ki.k}}} \tag{8}$$

Where:

$$N_{pl.k} = f_{y,k}\, \pi \frac{d_1^2}{4} \tag{9}$$

The buckling load of the screw depends on the elastic foundation and the buckling mode. The buckling mode itself depends on the length of the screw and on whether the screw head is a hinged or clamped support i.e. on the shape of the screw head. An approximation of the buckling load of the screw can be determined according to Bejtka and Blaß (2006) [2] from the buckling load of a beam on elastic foundation without support for hinged screw heads:

$$N_{ki.k} = \sqrt{c_h E_k \pi \frac{d_1^4}{64}} \tag{10}$$

and with support for fixed screw heads:

$$N_{ki.k} = 2\sqrt{c_h E_k \pi \frac{d_1^4}{64}} \tag{11}$$

Where E_k is the Young modulus of the screw (210‘000 N/mm^2) and ch is the foundation modulus. Bejtka and Blaß (2006) [2] found the following mean value for c_h in the tests reported in (Bejtka 2005) [1]:

$$c_h = \frac{(0.22+0.014\,d)\rho}{1.17\sin\alpha^2+\cos\alpha^2} \tag{12}$$

At the reinforced beam supports, the load can be carried both by the screws and by the timber. For the resistance of the reinforced support the resistance of the screw in compression can be added to the resistance perpendicular to the grain capacity of the timber. However, it has to be verified that the load distribution is sufficient to prevent compression failure in the timber above the reinforcement. The Bejtka and Blaß (2006) [2] suggested a load distribution at the reinforced support by 45° as shown in Figure 4.

4 Lateral load-carrying capacity

Dowel-type fasteners are traditionally used mainly in single or multiple shear plane connections. Compared to other fasteners, screws have the benefit of being able to carry loads in the axial direction, unlike dowels and smooth nails, which allows for a better load transfer in shear connections. The failure modes of shear connections are similar for all dowel type fasteners.

4.1 Failure modes

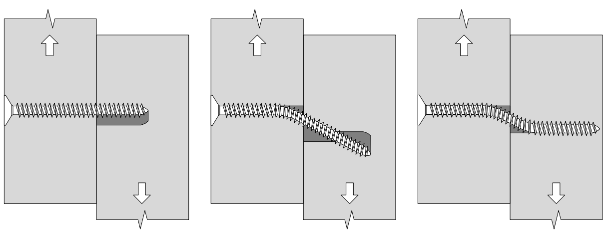

Figure 5: Failure modes of laterally loaded fasteners

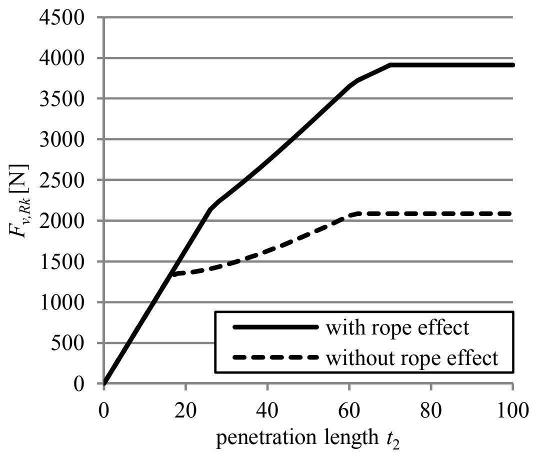

Figure 6: Lateral resistance of a screw connection (d = 8 mm) in dependency of the penetration depth t_2

The failure mode of a laterally loaded shear connection depends mainly on the thickness of the connected parts and the slenderness of the fastener. Johansen (1949) [11] distinguished different failure modes with and without plastic yielding of the fastener. The design model developed from these failure modes is often called European Yield model (EYM). The load-carrying capacity of a connection increases with increasing thickness of the connected members for a constant fastener diameter. The lateral resistance of a screwed connection is given in Figure 5 as a function of the penetration depth t_2 in member 2. It is distinguished between the resistance including the so called rope effect and without rope effect. With increasing penetration depth the failure modes change from embedment failure via partly ductile failure towards fully ductile failure with two plastic hinges in the screw. With increasing penetration depth also the resistance of the connection increases. It can be seen that for a sufficient thickness of the members, the maximal resistance with a ductile failure of the fastener is reached. It should always be aimed at reaching this thickness in order to achieve the maximal resistance and the ductile failure behaviour of the connection.

4.2 Plastic failure mode with rope effect

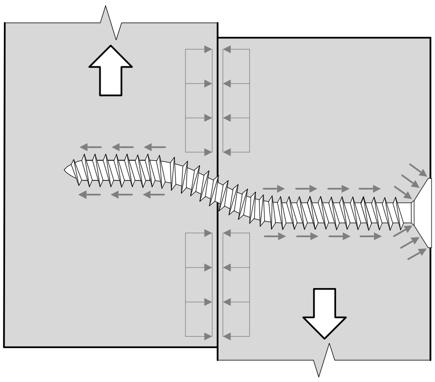

Figure 7: Ductile failure mode of the screw with plastic hinges and compression stresses from the rope effect.

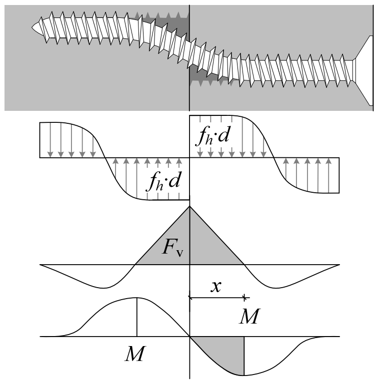

Figure 8: Lateral forces and moments in the screw

The lateral resistance of a connection with screws consists of portions from shearing of the fastener and from the rope effect leading to friction between the members.

$$R_{connection} = R_{Johansen} + R_{rope\ effect} \tag{13}$$

The according to the Johansen theory the resistance with a ductile failure mode is as follows:

$$R_{Johansen} = \sqrt{\frac{2\beta}{1+\beta}}\ \sqrt{2\ M_y f_{h,1} d_{ef}} \tag{14}$$

Where $\beta = f_{h,2,k} / f_{h,1,k}$ is the ratio between the embedment strength f_h of the members, M_y is the yield moment of the fastener and d_{ef} is the effective diameter of the screw. This effective diameter depends on the location of the yield moment. For fully threaded screws and in cases where the shank part of the screw is at a larger distance than $4d$ from the shear plane between the member the effective diameter is $d_{ef} = 1.1\ d_1$. In other cases the effective diameter is equal to the diameter of the shank d_s.

In EN 1995-1-1 (2008) [8], no specific embedment strength is specified for screws. The equations given for nails and dowels are to be used. Hence, it is distinguished between the diameter and the predrilling of the holes. The values are as follows.

For $d \leq 8$ mm without predrilled holes

$$f_{h,k} = 0.082\ \rho_k\ d^{-0.3}, \tag{15}$$

with predrilled holes

$$f_{h,k} = 0.082(1 - 0.01d)\ \rho_k \tag{16}$$

and for $d > 8$ mm in predrilled holes in softwood

$$f_{h,k} = \frac{0.082(1-0.01d)\ \rho_k}{(1.35+0.015\ d)\sin^2\epsilon + \cos^2\epsilon} \tag{17}$$

A much more detailed study on the embedment strength of self-tapping screws was performed by Bejtka (2005) [1]. The impact of the force to grain angle ε and the screw axis to grain angle α were studied. Only a minor impact of the force to grain angle ε was found in the tests. Additional parameters like e.g. the geometry of the screw thread were not considered in the determination of embedment strength.

$$f_{h,k} = \frac{0.019\ \rho_k^{1.24}\ d^{-0.3}}{2.5\sin^2\alpha + \cos^2\alpha} \tag{18}$$

The portion of the rope effect arises from the relative contraction of the screw when loaded perpendicular to its longitudinal axis, which leads to compression forces acting between the members. The friction coefficient between timber members is assumed to be $\mu = 0.25$ in EN 1995-1-1 (2008). However, the portion of the rope effect on the total lateral resistance should be smaller than the portion of the Johansen yield theory according to EN 1995-1-1 (2008) [8].

$$F_{v,Rk} = 1.15\sqrt{\frac{2\beta}{1+\beta}}\ \sqrt{2\ M_{y,Rk}\ f_{h,1,\mathrm{k}}\, d_{ef}} + \frac{F_{ax,Rk}}{4} \tag{19}$$

The factor 1.15 takes into account the increase of the design value of the resistance due to a reduction of the partial safety factor for the ductile failure mode.

4.3 Shear and compression

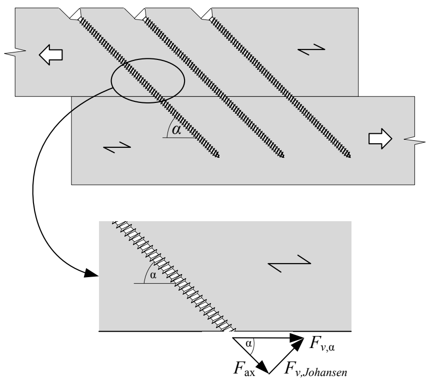

Figure 9: Application of self-tapping screws with an inclination a in shear connections in order to apply tensile forces in the screw.

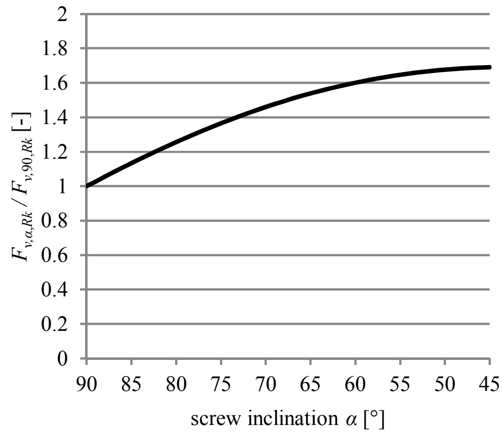

Figure 10: Increased resistance of self-tapping screws in dependency of the inclination angle α.

A much better structural behaviour can be achieved for connections when the screws are installed with an inclination, as shown in Figure 9, so that way the screws are also axially loaded. According to EN 1995-1-1 (2008) [8] the resistance of inclined screws with combined lateral and axial loading is supposed to be calculated by means of the quadratic criterion specified for threaded nails. Bejtka (2005) [1] propose a more detailed design approach for shear connections with inclined screws for different angles between screw axis and grain direction. The resistance of a connection with inclined screws $F_{v,\alpha,Rk}$ results from the portions of the axial $F_{ax,Rk}$ and the lateral resistance of the screw $F_{v,Johansen,Rk}$. Both these resistances have portions acting parallel and perpendicular to the interface of the members. The difference of the portions in direction perpendicular to the interface induces a friction force between the members. This friction force depends on the friction coefficient μ and increases the resistance of the shear connection. For the ductile failure mode 3 according to the Johansen model (EYM) with plastic hinges in the screw in both timber side members, the resistance of the shear connection can be calculated for a single screws as follows:

$$F_{v,\alpha,Rk} = F_{ax,Rk}(\mu \sin\alpha + \cos\alpha) + F_{v,Johansen,Rk}(\sin\alpha - \mu\cos\alpha) \tag{20}$$

5 Edge and end distances

It is well known, that connections with dowel type fasteners in a row parallel to the grain increase the risk of splitting of the timber leading to brittle failure of the connection. In order to avoid this brittle failure sufficient spacing between the fasteners is necessary. Also the edge and end distances of the fasteners should be satisfied. The specified minimal values in EN 1995-1-1(2008) [8] are for spacing $a_1 = 7d$ (parallel to grain) and $a_2 = 5d$ (perpendicular to grain) and for end and edge distances $a_{1,CG} = 10d$ (parallel to grain) and $a_{2,CG} = 4d$ (perpendicular to grain).

The same spacing and distances are suggested in EN 1995-1-1 (2008) [8] for self-tapping screws and for other dowel type fasteners. These distances are based on results from tests on doweled and nailed connections. It is assumed that self-tapping screws exhibit a similar structural behavior and, hence, similar distances are used. With the development of high performance screws, the spacing and distances should be reviewed as well.

Brittle failure of the timber around the whole connection can also occur. In shear connections this type of failure is known as block or plug shear failures. Also tensile connections can fail due to exceedance of the strength of the surrounding timber (net cross-section failures).

6 Reinforcement by means of fully-threaded self-tapping screws

Screws are perfectly suitable as reinforcing elements due to their good structural behaviour in the axial direction. The screws should be fully-threaded in order to allow for a good load transfer along the whole length between screw and timber, without stress concentrations. The diameter of screws for reinforcing purposes is often considerably larger than for laterally loaded connections. By choosing diameters in the range of $d \approx$ 10-20 mm, high resistances can be achieved and stresses in timber can be distributed in large regions. Threaded rods of variable length are often used for these large diameters. The structural behaviour of these threaded rods and self-tapping screws is assumed to be similar with regard to withdrawal, tension and lateral resistance. Further investigations are needed for the determination of the load distribution effect of these reinforcing elements.

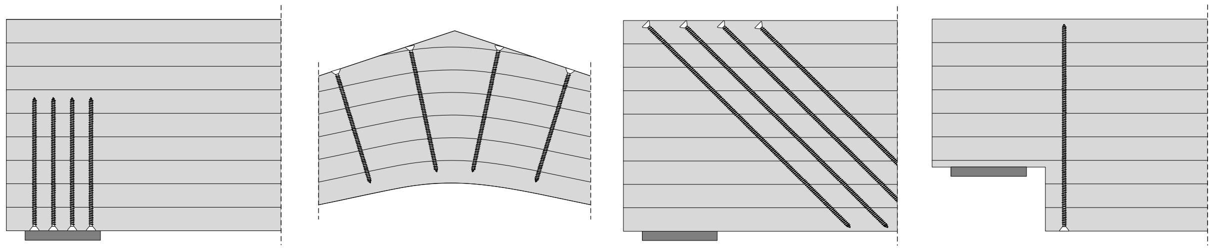

Figure 11: Different types of reinforcement by means of fully threaded self-tapping screws.

For reinforcing purposes not only the strength but also the stiffness is of major interested. Nevertheless only a small number of publications are available for the stiffness of self-tapping screws.

Design procedures for reinforcement are not given in EN 1995-1-1 (2008). Information about the reinforcement of supports can be found in Bejtka (2005) [1], about shear reinforcement in Dietsch (2012) [4] and about reinforcement of notched beams in Jockwer (2014) [12].

7 Conclusion and future work

Self-tapping screws are multifunctional fasteners with excellent resistance and stiffness in axial direction. They can be used in various types of connections and as reinforcing elements. The structural behaviour in axial and lateral direction of self-tapping screws in connections is summarized in this paper. The different screw characteristics and their impact on the resistance of screws are explained. The shear resistance of the connection can be increased considerably by installing screws with an inclination to the shear plane. Due to their axial capacity the best reinforcing effect can be achieved by adjusting the screw axis in direction of the acting forces. That way fully-threaded self-tapping screws can act as effective reinforcing measures.

Self-tapping screws have made a large development coming from lag screws towards optimized high strength fasteners. The design procedures for these high performance screws have not yet been completely updated in order to make use of all advantages of self-tapping screws. In addition further research is needed in order to give more precise recommendations with regard to the spacing and distances of self-tapping screws.

References

[1] Bejtka, I. (2005). Verstärkung von Bauteilen aus Holz mit Vollgewindeschrauben, Fakultät für Bauingenieur-, Geo- und Umweltwissenschaften, Universtity of Karlsruhe. PhD Thesis.

[2] Bejtka, I. und Blaß, H. J. (2006). Self-tapping screws as reinforcements in beam supports. Proceedings of the CIB-W18 Meeting 39, Florence, Italy, Conference. 39-37-32

[3] Blaß, H. J., Bejtka, I., et al. (2006). Tragfähigkeit von Verbindungen mit selbstbohrenden Holzschrauben mit Vollgewinde. Karlsruher Berichte zum Ingenieurholzbau. Karlsruhe, Lehrstuhl für Ingenieurholzbau und Baukonstruktionen, Universität Karlsruhe (TH). 4.

[4] Dietsch, P. (2012), Einsatz und Berechnung von Schubverstärkungen für Brettschichtholzbauteile. Fakultät für Bauingenieur- und Vermessungswesen, Technical University of Munich, PhD Thesis.

[5] DIN 7998 (1975): Gewinde und Schraubenenden für Holzschrauben, DIN Deutsches Institut für Normung e.V., Beuth Verlag GmbH, Berlin, Deutschland.

[6] EN 1382 (1999): Timber structures - Test methods - Withdrawal capacity of timber fasteners. CEN European committee for standardization.

[7] EN 1383 (1999): Timber structures - Test methods - Pull through resistance. CEN European committee for standardization.

[8] EN 1995-1-1 (2008): Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings:2004 +AC:2006 +A1:2008. Brussels, Belgium, CEN European committee for standardization.

[9] EN 14592 (2008): Timber structures - Dowel-type fasteners - Requirements. CEN European committee for standardization.

[10] Frese, M., Fellmoser, P., et al. (2010). Modelle für die Berechnung der Ausziehtragfähigkeit von selbstbohrenden Holzschrauben. European Journal of Wood and Wood Products 68(4). 373-384.

[11] Johansen, K. W. (1949). Theory of Timber Connections. IABSE publications 9. 249-262.

[12] Jockwer R., Steiger R., Frangi A. (2014): Fully threaded self-tapping screws subjected to combined axial and lateral loading with di erent load to grain angles. In: Aicher S., Reinhardt H.W., Garrecht H. (eds) Materials and Joints in Timber Structures, RILEM Bookseries, vol 9, Springer Netherlands, pp 265-272

[13] Pirnbacher, G., Brandner, R., et al. (2009). Base parameters of self-tapping screws. Proceedings of the CIB-W18/42-7-1, Duebendorf, Switzerland, Conference.

Selftapping screws - Possibilities and Application

Nils Horn[1]

Summary

Fully-threaded self-tapping screws have been continuously gaining ground in timber engineering, with the range of products available increasing steadily. This has resulted in ever-new application possibilities, whereby a clear trend is to develop application-specific fasteners as well. Static reinforcement of timber components in both new build and renovation work is one of the latest developments in applications for fully-threaded self-tapping fasteners. Solutions, particularly for strengthening work on existing support structures, are generally project-specific, with little or no systematic guidelines. Choosing the optimum reinforcements, whether that be modifications to the support structure, or using supporting fasteners, is really down to the knowledge and expertise of project personnel. There are tried-and-trusted systems for new build applications; albeit lacking in part in any Europe-wide regulatory control.

Key words: Self-tapping fully-threaded fasteners, Strengthening, New building, Refurbishment work

1 Introduction

These days in many European countries renovation accounts for up to 50% of building work. It seems reasonable to suppose that this trend will be maintained over the medium term. This is primarily governed by the number of buildings in existence in many European countries, which require renovation, increasing demands on energy efficiency and the urgent need for new living space in many cities. There is indeed massive dormant potential for redevelopment of older existing buildings; whether that be improving energy efficiency, change of use, extending or converting – to mention but a few alternatives.

It is very clear that there are extreme requirements placed on the building solutions applied and the materials used in renovation work. In every case, the most economical, energy-efficient and aesthetically pleasing solution has to be found.

In every renovation there are changed requirements on the existing bearing structure. It is thus often necessary to reinforce the existing structure to ensure that the required strength characteristics are met. Where a bearing structure does not meet its revised requirements, components in the structure can be replaced or enhanced, changes can be made to the static system or individual components repaired, replaced or reinforced in order to meet the new requirements.

A very recent development is the use of fully-threaded screws and non-self-tapping threaded rods for reinforcing timber structures – albeit pioneers of flight were working on optimising timber aircraft with regard to weight and support strength as long ago as the 1920's.

2 Material and method

Fasteners are used in a similar way to the steels rods in reinforced concrete structures. Timber as a natural product exhibits certain characteristics which are problematic for the use of a load bearing elemen; e.g. the very limited load bearing capacity under tensile or compressive load perpendicular to the grain. Shear loads are also problematic for timber. Such weaknesses can be compensated via strengthening with fully-threaded fasteners or by making the system more rigid overall. Critique from non-specialists centres on the question of whether the necessity of using steel strengtheners maeks timber as such unsuitable for building support applications: oblivious to the fact that steel reinforcements in concrete have been in use since the middle of the nineteenth century to address the so-called weakness in concrete. For, like timber, concrete has little resistance to tensile stress. Another advantage in using internal steel strengthening elements in timber is that they have minimal optical impact; with just the heads visible. Indeed even the heads can also be countersunk and covered. This also offers an advantage in terms of fire-resistance.

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There are numerous tried-and-trusted solutions to choose from when it comes to reinforcing a timber construction or wooden cross section. The following is a list of the best known systems:

- Closing cracks, glued joints and de-laminations
- Strengthening with fully-threaded fasteners and threaded rods
- Reinforcements with glued-in plates made of steel and carbon-fibre polymers
- Timber or reinforced plastic cladding
- Adding additional timber or reinforced plastic layers
- Transverse pre-stressing with fasteners or bolts
- Timber-concrete constructions
- Modifications to the static system
- etc.

In other words, strengthening procedures can be either internal or surface-mounted. Whether for new build or restoration, fully-threaded fasteners offer economical and long-lasting solutions as internal reinforcement components. There is a clear trend in new building towards thinner and thinner sections, which obviously means reduced load application and support surface. In addition to that the drive to an optimized enclosed space makes notches in big cross sections necessary. That might be in order to ensure that the maximum building height is not exceeded. Certainly there are many more complex individual solutions when it comes to renovation work.

For existing support structures, the choice of the right strengthening system depends a lot on the specific local conditions. It is essential to match the strengthening measures to the specific case to find the optimum solution in every case. The following guidelines are critical in the choice of the right system and materials:

- The condition of the existing cross sections (damage from fungal attack, insects, cracking, penetrating damp, mechanical damage…)
- Cross-section values of the existing timbers
- Causes of damage to existing structures
- Accessibility to the system and its components
- Climatic conditions
- Building regulations
- Requirements resulting from the intended future use

Various other points may need to be considered according to circumstances:

- Thermal insulation and sound-proofing
- Fire-resistance requirements
- Inspection procedures
- Maintenance
- Aesthetics

Any strengthening measures also have to be economically viable. It is therefore necessary that the costs are balanced against the projected life of the building.

The following paragraphs discuss various popular strengthening systems using self-tapping, fully-threaded fasteners currently available on the market for use in both new building and renovation work.

Beam strengthening

Beam strengthening is one of the commonest reinforcements in renovation work with secondary beams fixed above, below and on the side of existing beams. This procedure may be necessary where a change of use dictates that the existing support strength is insufficient, or cannot be guaranteed. Another reason for this approach can be a situation where the beam-end is no longer safe: perhaps bricked in the outside wall as was common in older brick-built construction and subject to attack by mould and fungi or susceptible for other reasons.

The fasteners are designed to create a tight fit between the existing timbers and the new components. The stiffness of the new structure is critical because the new increased section has to take up the load from t = 0. Otherwise there is a risk of failure before the load is transferred across the entire section.

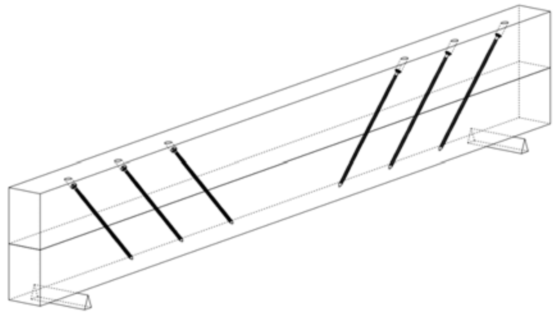

Figure 1: A schematic drawing illustrating the strengthening of a beam via doubling up

Reinforcement perpendicular to the grain for suspended loads

It may be necessary to fit new timber components to the existing structure without the possibility to select statically optimised joints. Side loading and secondary beams give rise to transverse loads. The reinforcements are based on the assumption of a tear zone in the timber. The forces have to be carried by the additional fasteners inserted perpendicular to the grain. Screws are inserted at right angles to the fibre.

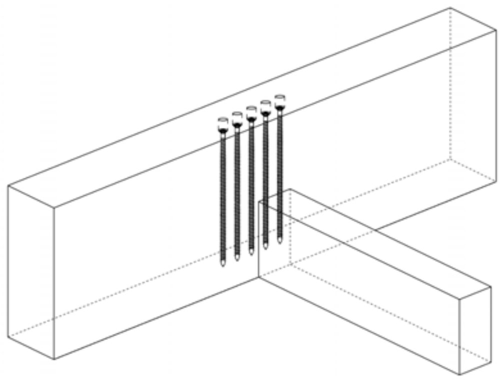

Figure 2: Transverse strengthening procedures against a side load (schematic drawing)

Notches

Planners should avoid notches wherever possible.

Certainly according to DIN EN 1995-1 a maximum of 50% of the support height may be cut out. Reinforcing the cut-away may not be necessary if certain conditions are met, but is definitely to be recommended! Even usage class 3 allows notches, but reinforcements must be applied.

Fasteners should be inserted below 90 degrees to the grain and with the minimum possible gap between the fasteners and the notch. Multiple fastener rows in the longitudinal direction are not permitted.

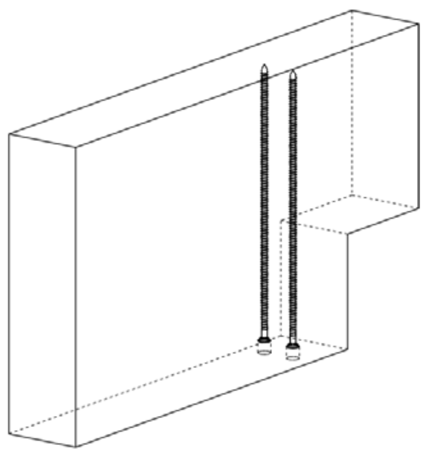

Figure 3: Reinforcement of a notch (schematic drawing)

Support reinforcement

Slim and high cross sections often offer inadequate support surface area to take up prevailing forces. In such cases, fasteners should be inserted under 90 degrees to the grain. Dimensioning is done at two levels: firstly the actual support face and secondly the internal plane liable to transverse forces at the tips of the fasteners. In order that the load can be taken up by the screws, the contact surface must be flat and exhibit adequate stiffness. Steel plates should be used to support these requirements, because the forces under the screw heads can reach the plastic deformation limit of steel. Countersunk fasteners are preferable to proud round-headed bolts (1)

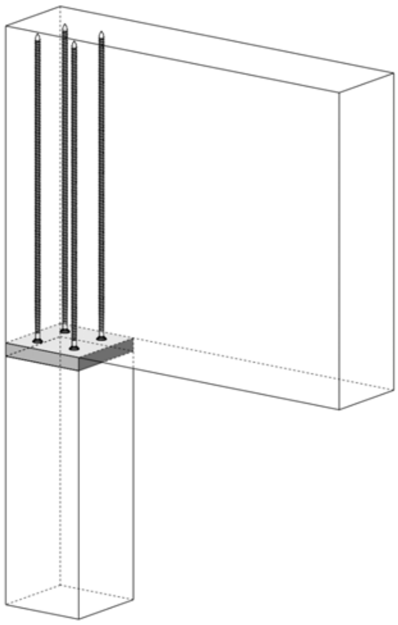

Figure 4: Support reinforcement (schematic drawing)

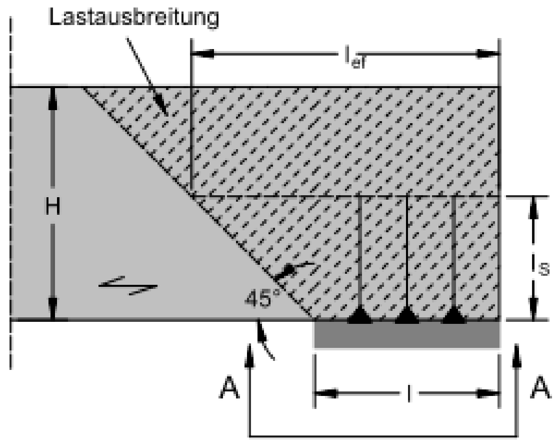

Figure 5: Load displacement under 45 degrees (schematic drawing); sketch from [1]

Cut out

Not least because of the relatively low price, whole-wall supporting plate girders are often used for spans from 15 to 25 m. Because of their static characteristics it is often sensible to feed pipework straight through them. The required break-throughs must be reinforced. Fasteners should be inserted as close as possible to the cavity at right angles to the timber grain. Similarly to the situation with cut-outs, only one fastener is deemed to be offering support in the transverse direction.

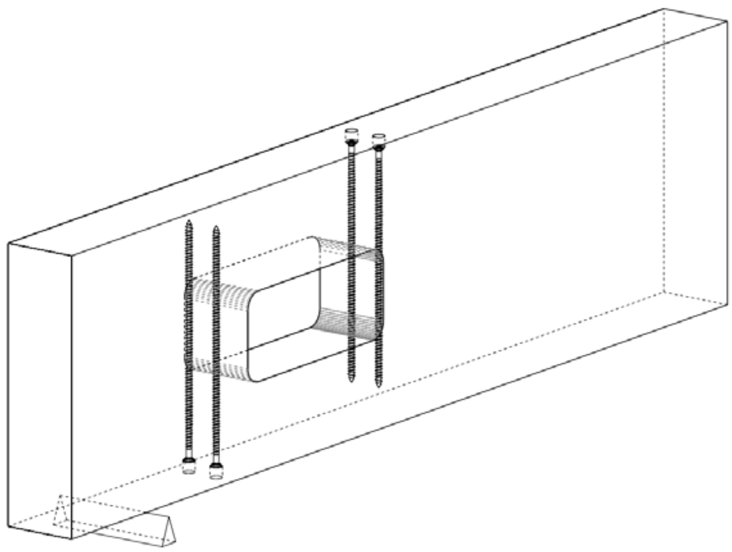

Figure 6: Cut out reinforcement (schematic drawing)

Changes to the load bearing structure

Where local conditions admit of it, matching the support structure to intended future use can be an economical option, as opposed to strengthening existing members. The critical advantage of such adaptations where new members are fitted to the existing structure is that the new additions can be carefully designed using CAD and carefully prepared in the factory prior to installation. The job of the fasteners is merely to fit the new components to the existing structure.

There is, moreover the possibility of integrating the fasteners as a constituent of the revised support structure. This is demonstrated in the diagram. In this example it was not possible to generate a conglomerate section with old and new parts in direct contact. This occasioned a new static model leading to the conversion of the original single support into a multi-facet solution.

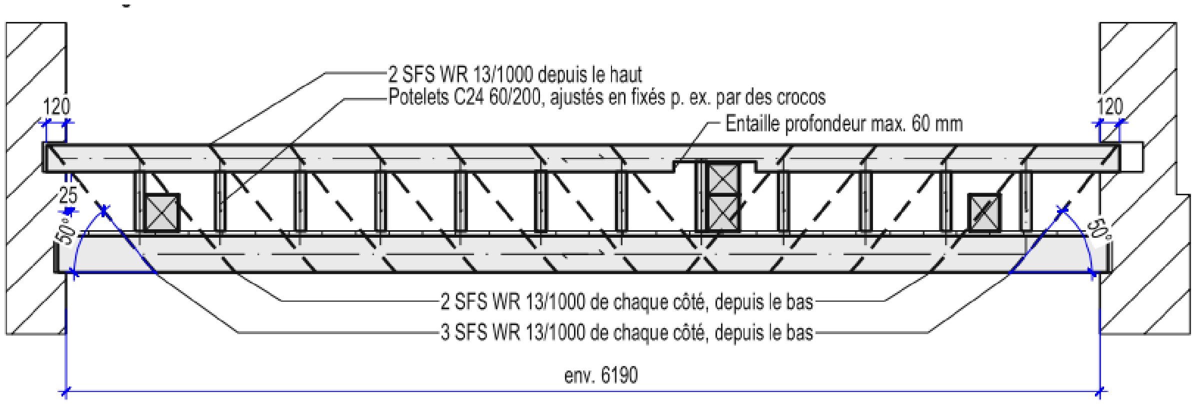

Figure 7: Using self-tapping screws to fit a new component to an existing beamed ceiling using the screws as system tie bars. (Concept: Aleksis Dind, Lutz Architectes Sàrl, Martin Geiser Conception Bois Sàrl)

Reinforced connections with dowel-type fasteners

The load strength of steel plate-to-timber assemblies if governed primarily by the timber bore stability and the yield point of the fasteners and of course the physical dimensioning of the set-up.

Modern timber engineering often utilises large section timbers with dimensions of up to 40 x 40 cm and more coupled to steel supports via steel inserts and retaining bolts. The danger with such arrangements is the risk of timber splitting or a block shearing. This can be avoided by the right use of long fully-threaded bolts. Used correctly these increase the overall support strength and ensure a ductile failure mode if the load is too excessive.

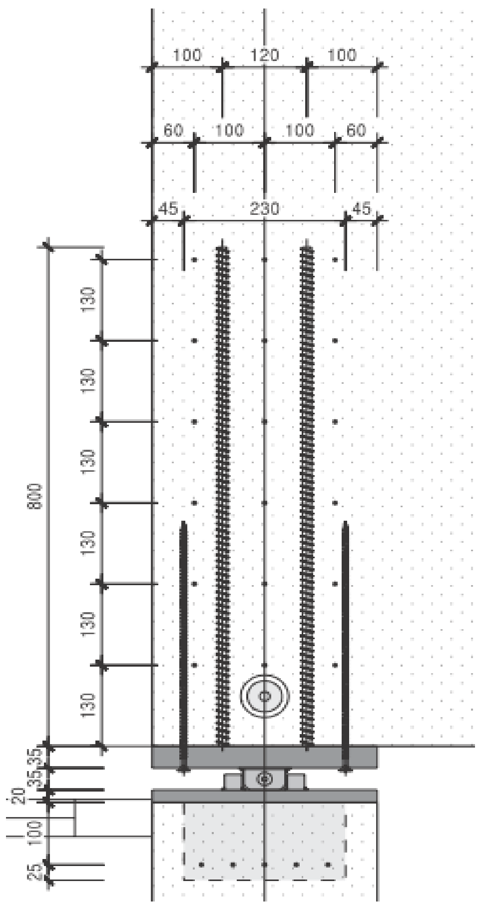

Figure 8: Reinforced connections with dowel-type fasteners for a support reinforcement

3 Results and discussion

Strengthening with self-tapping, fully-threaded screws and fasteners opens up enormous possibilities for creative solutions in new build, whether for design, technical or cost-driven reasons.

When it comes to renovation work, reinforcing a support structure can mean that a historical timber feature can be retained following change of use, or a support system damaged by external influences can be repaired.

Dimensioning reinforcements using self-tapping, fully-threaded fasteners and bolts is much more demanding in restoration work on existing support structures as compared to new build design because of the basic lack of data. Nevertheless reinforcements readily available on the market for new build are by no means uniformly standardised. In both cases it goes without saying that a generous and demonstrable safety margin must be calculated into the plan.

Proof of self-tapping fully-threaded fastener strength characteristics and the requirement to fix surface-mounted strengthening plates are governed by the relevant European technical approval documents. There is indeed no variation in the demands made by the various ETA's, but there is no overall standard agreement in force. Likewise when it comes to requirements for the reinforcement of break-throughs which, on the basis of regulations contained in standard DIN 1052 2008-12, have been adopted in the national appendix to Eurocode 5 within Germany: likewise in Austria. There is, however, no single Europe-wide governing directive on this issue.

In implementing static reinforcements of timber beams, it is vital that the timber make-up of the existing section is known if an economical solution is to be achieved. An appropriate stiff reinforcement support against sagging and breakage can then be determined for the section in question, with new components fixed to the existing structure as required. There is a pressing need for non-destructive test procedures for existing timbers – at least to determine the information required for static verification.

When a support system is being renovated, among other considerations where new timber components are introduced, it is vital that the new parts represent a close match to the existing elements. There are various measuring systems available on the market for analysing existing supporting timber work.

4 Conclusion and future work

New building offers opportunities to explore all design possibilities, but cut-outs and break-throughs are to be avoided. The potential for reinforcing timbers during renovation work is clearly apparent, but it is also a useful tool in new build scenarios. What is missing is a Europe-wide standardisation; albeit such directives do exist at the national level.

Solutions for the refurbishment of existing buildings are inevitably individual and project-specific. Where self-tapping, fully threaded fasteners and fixings are employed, there are possibilities to individualise surface finishes, dimensions and materials – perhaps turning to carbon or stainless steels. It is vital to establish the requirements of the fasteners (so that the best possible choice can be made) early on in the planning phase if an economical and technically sound solution is to be arrived at. Again, the suitability of the chosen fasteners in terms of working with them in existing structures has to be evaluated: in historic buildings, exotic timber varieties no longer in common use today may have been used.

References

[1] Blaß, H.J.; Bejtka, I. " Querzug- und Querdruckverstärkungen - Aktuelle Forschungsergebnisse" Forschungsbericht 2004. Versuchsanstalt für Stahl, Holz und Steine, Universität Karlsruhe

[2] Blaß, H.J.; Bejtka, I. "Querzugverstärkungen in gefährdeten Bereichen mit selbstbohrenden Holzschrauben" Forschungsbericht 2003. Versuchsanstalt für Stahl, Holz und Steine, Universität Karlsruhe

Engineering design of CFRP reinforced single span timber beams loaded in bending

Martin Lehmann[1]

Summary

As the reinforcement of timber structures using carbon fibre reinforced polymer (CFRP) plates is still regarded as special application the design calculations are not well documented. Furthermore the long-term behaviour is not fully understood yet. In this paper a short review of the calculations method for retrofitted concrete and timber structures is given. Furthermore some details on the engineering design of CFRP reinforced timber beams loaded in bending are presented.

Key words: CFRP reinforcement, Composite section, SIA 265, EC 5

1 Introduction

As the reinforcement of timber structures using carbon fibre reinforced polymer (CFRP) plates is still regarded as special application the design calculations are not well documented. Furthermore the long-term behaviour is not fully understood yet. This is in contrary to other applications such as reinforcement of concrete structures where numerous research projects were carried out and substantial experience was gained in the last 30 years. Therefore the design calculations are well documented for such application [25] and the adhesives are optimised for concrete structures. As the loadbearing behaviour and especially the failure behaviour of timber is quite different to the one of steel reinforced concrete the well-known engineering models for CFRP reinforced concrete structures are not valid for CFRP reinforced timber structures and the for the concrete optimised adhesives are may not the optimum for timer structures [16].

2 Calculations methods for reinforced members

The Swiss construction standard SIA 166 "externally bonded reinforcement" [1] is valid for various construction materials such as concrete, masonry, steel and timber. However the presented calculation models are mainly based on experiments done using concrete as substrate and experience gained from reinforcing concrete structures. Therefore the presented calculations models are based on concrete behaviour and of limited use to design a timber structure. The SIA 166 [1] distinguishes between different zones over the length of the reinforcement lamella: the active zone and the anchorage zone. The ultimate limit state of the active zone is given by the tension capacity of the lamella. The ultimate limit state of the anchorage zone strongly depends on the means used for the anchorage and in case of adhesively bonded anchorage its length and the pull-off strength of the substrate. In concrete structures the anchorage zone has to be located in the theoretically not fissured part (at ultimate load) of the structure. The region containing cracks is regarded as active zone. For timber structures no explanations concerning the distinguishing between the anchorage and the active zone are given. However the tension behaviour of timber is more or less linear elastic up till failure therefore one could assume that the standard theories for composite beams are applicable. This means that for adhesively bonded lamellas the stress in the anchorage zone is equal to the shear stress present in the adhesive determined using standard equations for linear elastic composite sections. For concrete structures SIA 166 [1] recommends using equations (1) to (3) to determine the anchoring resistance.

$$l_{b,0d} = \frac{\pi}{2}\sqrt{2 \cdot \frac{G_{Fbd} \cdot E_l \cdot t_l}{\tau_{l0d}^{\,2}}} \tag{1}$$

$$F_{b,Rd} = b_l \cdot \sqrt{2 \cdot G_{Fbd} \cdot E_l \cdot t_l} \cdot sin\sqrt{\frac{\tau_{l0d}^{\,2} \cdot l_{bd}^{\,2}}{2 \cdot G_{Fbd} \cdot E_l \cdot t_l}} \qquad if\ l_{bd} \le l_{b0d} \tag{2}$$

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$$F_{b0,Rd} = b_l \cdot \sqrt{2 \cdot G_{Fbd} \cdot E_l \cdot t_l} \qquad if\ l_{bd} \geq l_{b0d} \tag{3}$$

Where:
$l_{b,d}$ = design value of the anchorage length
$l_{b,0d}$ = design value of the maximal effective anchorage length
G_{Fbd} = design value of the fracture energy of the substrate
E_l = MOE of the lamella
t_l = thickness of the lamella
τ_{l0d} = design value of the maximal shear strength of the substrate
$F_{b,Rd}$ = design value of the anchorage resistance
$F_{b0,Rd}$ = design value of the maximal anchorage resistance
b_l = width of the lamella

Holzenkämpfer presents in his dissertation engineering models for the bond behaviour of adhesively bonded reinforcement for concrete structures [11]. He presents the solution of the differential equation for different bond behaviour (linear, bilinear and nonlinear). Holzenkämpfer conducted experiments on concrete specimens with externally bonded steel lamellas to verify his models. Furthermore he developed design models for reinforced concrete structures. The models for bending consider debonding in the anchorage and/or in the zone with high bending moments (active zone). The models for debonding in the active zone are based on the bending or shear crack size present in the concrete. The model for the anchorage zone is based on the concrete tension strength and the fracture energy of the concrete. The model allows the determination of the maximal force that can be anchored using adhesively bond. The equations presented by Holzenkämpfer [11] for the anchorage strength is quite similar to the equation presented in the Swiss standard [1]. Holzenkämpfer uses a hyperbolic tangent function instead of a sinus function and therefore the equation is valid for every anchoring length. The equations used to estimate the shear strength and the fracture energy of the concrete are different in the two models. However both models base the estimation on the tension strength of the concrete. Holzenkämpfer uses a modifications factor for different width of the lamella and the substrate. Holzenkämpfer [11] recommends to use 80% of the result of equation (4) as the characteristic value. This leads to significant difference in the characteristic resistance calculated using the two models. Holzenkämpfer estimates higher values for short anchoring length. The maximal resistance is equal for 104 mm wide lamellas. For smaller lamellas Holzenkämpfer estimates higher and for wider lamellas lower values than the SIA model. He also presents the lower limit of equation (4) valid for short anchoring length (equation (5)). Furthermore the upper limit of equation (4) valid for long anchoring length is presented (equation (6)) this equation is identical with the one presented in SIA 166 [1]. There is no description about the range of the anchoring length were equation (5) can be applied.

$$F_{anch} = b_l \cdot \sqrt{2 \cdot G_F \cdot E_l \cdot t_l} \cdot tanh \sqrt{\frac{{\tau_{l1}}^2 \cdot l^2}{2 \cdot G_F \cdot E_l \cdot t_l}} \tag{4}$$

$$F_{anch} = b_l \cdot \tau_{l1} \cdot l \tag{5}$$

$$F_{anch} = b_l \cdot \sqrt{2 \cdot G_F \cdot E_l \cdot t_l} \tag{6}$$

Where:
G_F = fracture energy of the substrate
El = MOE of the lamella
l = anchorage length
τ_{l1} = maximal shear strength of the substrate
F_{anch} = anchorage resistance
b_l = width of the lamella

Equation (5) represents the maximum concerning the strength of the substrate where equation (4) considers the fracture mechanics and equation (6) presents the limit given by fracture mechanics.

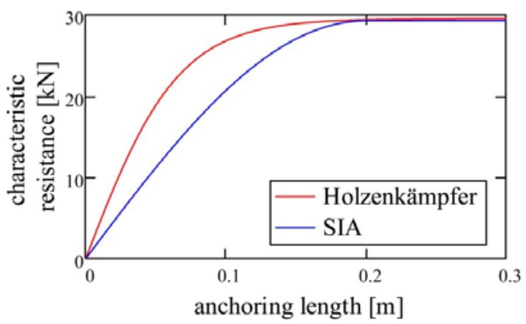


Figure 1: Comparison of the characteristic anchoring resistance calculated using the equations provided in SIA 166 (blue) and Holzenkämpfer (red). The calculation is based on a C25/30 concrete and a CFRP-lamella with an MOE of 165 GPa, a thickness of 1.2 mm and a width of 100 mm.

Plevris and Triantafillou developed a calculation model for FRP reinforced timber in bending and bending combined with compression for different failure modes [18]. The model was calibrated with experiments carried out on small clear specimens with various volume fractions of FRP reinforcement. The results showed that even small area fractions of FRP leads to a significant improvement of the load bearing capability. The calculation model developed by Plevris and Triantafillou [18] considers four different failure modes of small clear timber specimens reinforced with FRP. The timber is regarded as linear elastic until failure on the tension side and also linear elastic up to yield strain on the compression side. For strain higher than yield strain the stress strain curve is described by a falling branch (Figure 2).

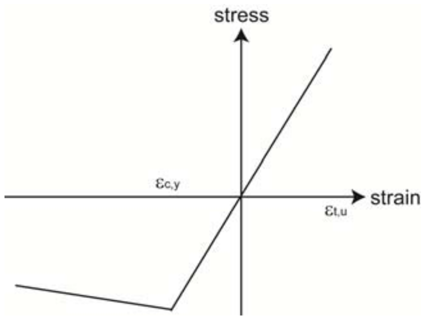


Figure 2: Sketch of the stress strain relationship as used by Plevris et al. [18]

Tension strain in timber is taken as failure criteria of the composite section. The considered modes are:

1. Timber and FRP remains in its linear elastic state
2. Timber remains in its linear elastic state and FRP has ruptured
3. Timber yields on the compression side and FRP remains in its linear elastic state
4. Timber yields on the compression side and FRP has ruptured

Rupture can occur in all four modes described above. However no report of tension failure in the CFRP before failure in timber has been found in literature considering structural sized specimens.

Dziuba tested timber beams reinforced with glued in steel wires in bending [10]. The area fraction of the reinforcement was varied between 0 % and 4 %, the used wires had an average yield stress of 535 MPa. The cross section of the beams was around 50x160mm the distance between the centre of gravity of the reinforcement and outermost tension fibre of the timber was 30 mm. For the beams with a higher percentage of reinforcement Dziuba reports failure of the beam due to compression failure of the timber without failure of the tension side. This leads to his failure model that includes a bilinear stress strain relationship for timber under axial load. The tension side is linear-elastic up until failure, the compression side has a linear-elastic branch followed by an ideal plastic branch, which is limited by an ultimate strain ($\varepsilon_{c,u}$) where failure occurs.

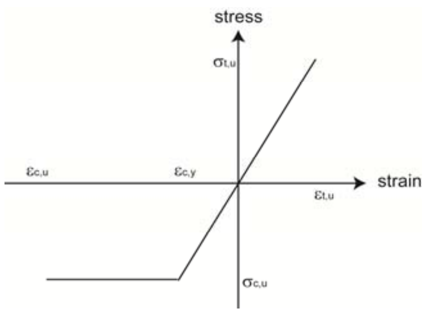


Figure 3: Sketch of the stress strain relationship as used by Dziuba [10], Blass and Romani [3, 19]

Romani and Blass [19] developed a design model for FRP reinforced beams with a facing timber lamella on top of the FRP-lamella. The model allows for plasticity in timber under compression. The published equations allow the calculation of the ultimate load bearing capacity based on the ultimate tension and compression strength of the timber. Different failure mode depending on the ratio between the ultimate stress in compression in timber and tension in timber as well as in the FRP are considered. Blass and Romani [3] extended the design model for FRP reinforced beams without facing timber lamella. The design model does allow the calculation of the ultimate loadbearing capacity without iteration. However in case of active plasticity the calculation of the stress distribution based on a loading situation is only possible with multiple iterations.

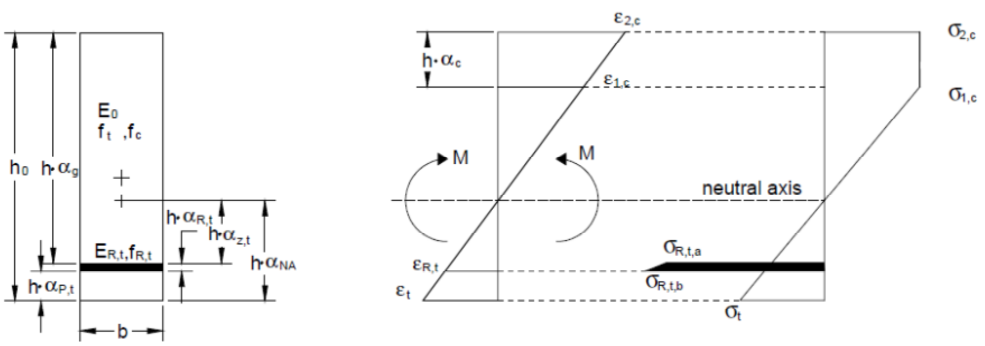


Figure 4: Stress and strain distribution as used for the calculation model of Romani and Blass [19]

3 Calculation methods for prestressed members

Depending of the prestressing system the applied prestress force (P) is reduced due to elastic deformation of the structure. This elastic loss (ΔP_{el}) occurs if a system where the jack is mounted on a pretensioning frame is used (Figure 5). The prestress force (P_0) present in the member is given by equation (7). Most post-strengthening methods for concrete do not have elastic losses because the prestressing jack is attached to the concrete and the elastic deformation occurs whilst building up the prestress force P (Figure 6).

$$P_0 = P - \Delta P_{el} \tag{7}$$

Where:
P_0 = prestress force present in the member
P = prestress force applied to the system (jacking force)
ΔP_{el} = elastic prestress force loss

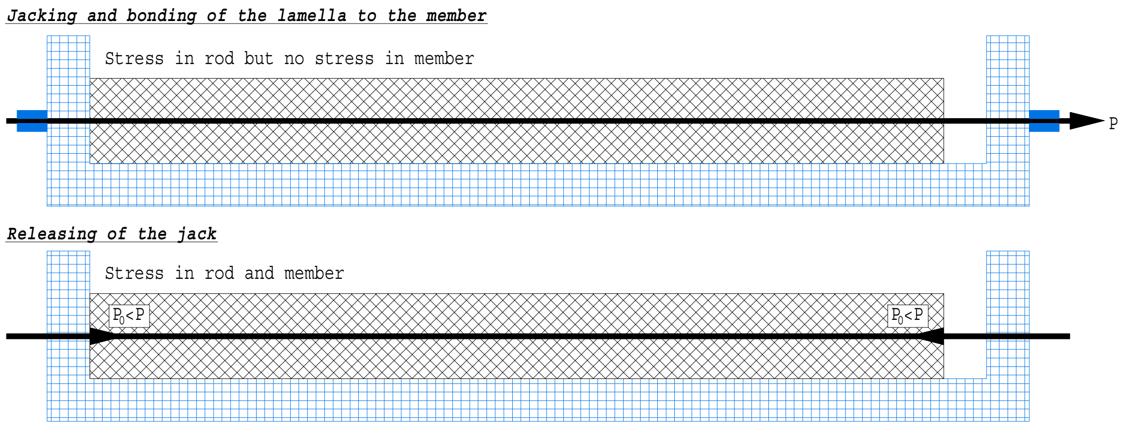


Figure 5 Sketch of prestressing a member using a pretensioning frame or something simular were the jack is mounted on a frame and therefore elastic loss does occur.

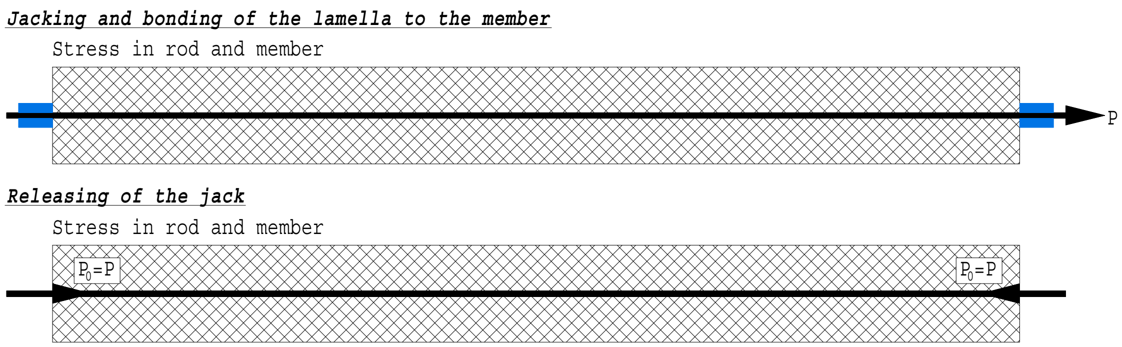


Figure 6: Sketch of prestressing where the jack is mounted on the member and therefore no elastic loss does occur.

In reference to Thomsing [22] the prestress force and the moment due to eccentricity act on the conceptual cross section. The moment due to eccentric prestressing is determined by the multiplication of the prestress force with the eccentricity.

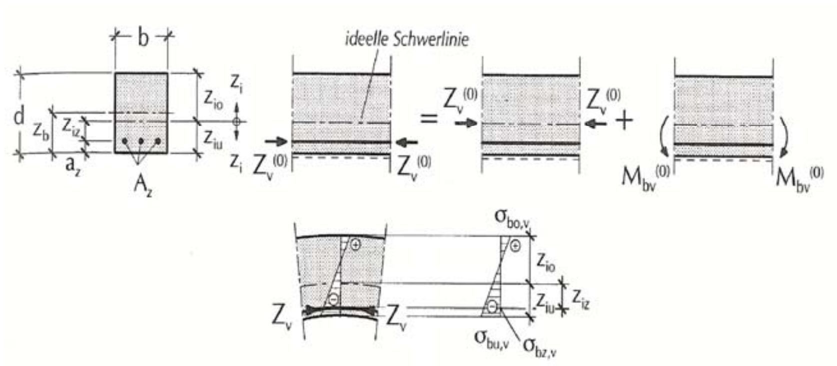


Figure 7: Determination of the prestress in case of eccentric prestressing [22]

Triantafillou and Deskovic [23] investigated the maximal anchoring force for adhesively bonded prestressed CFRP-lamellas assuming concrete or adhesive failure. The analyses they presented assumed linear-elastic materials and the governing deformation mode in the adhesive layer as shear. The stress strain relationship is assumed as bilinear model where the first part is linear-elastic followed by a perfect plastic plateau until failure (Figure 8).

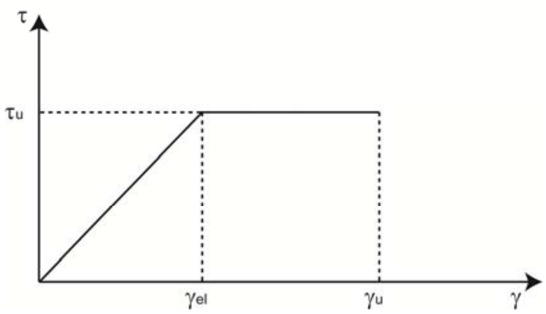


Figure 8: Shear stress-strain relationship for epoxy adhesive as used by Triantafillou et al

The shear stress in the elastic domain of the adhesive can by determined using equation (9) [23]

$$\tau(x) = \frac{\gamma_{el,a} \cdot G_a}{sinh \cdot \left(\frac{\omega \cdot l_{el}}{2}\right)} \cdot \sinh(\omega \cdot x) \tag{8}$$

$$\omega = \sqrt{\frac{G_a}{h_a \cdot h_{cf}}\left(\frac{1}{E_{cf}} + \frac{4 \cdot h_{cf}}{h_s \cdot E_s}\right)} \tag{9}$$

Where:
$\tau(x)$ = shear stress due to prestressing in the adhesive as a function of x
$\gamma_{el,a}$ = maximal elastic shear strain (Figure 8)
G_a = shear modulus of the adhesive
l_{el} = length of the elastic domain
h_a = thickness of the adhesive layer
h_{cf} = thickness of the CFRP-lamella
E_{cf} = modulus of elasticity of the CFRP-Lamella
h_s = height of the substrate
x = position over the length of the beam
E_s = modulus of elasticity of the substrate

The shear stress in the area were the adhesive has passed its elastic limits is equal to the ultimate shear stress of the adhesive ($\tau_{u,a}$) (Figure 8).

Triantafillou et al assumed that the equation (10) for the shear deformation is valid for the whole length of the reinforced beam. This assumption is consistent with the commonly hypothesis that the strains in the inelastic regime are approximately described by the same relationship characterising elastic response (e.g. bending beam).

$$\gamma(x) = \frac{\gamma_{el,a}}{sinh \cdot \left(\frac{\omega \cdot l_{el}}{2}\right)} \cdot \sinh(\omega \cdot x) \tag{10}$$

Where:
$\gamma(x)$ = shear strain due to prestressing in the adhesive as a function of x
$\gamma_{el,a}$ = maximal elastic shear strain (Figure 8)
l_{el} = length of the elastic domain
ω = is given by equation (9)

The equations presented by Triantafillou and Deskovic [23] lead to the following shear stress and strain distribution .

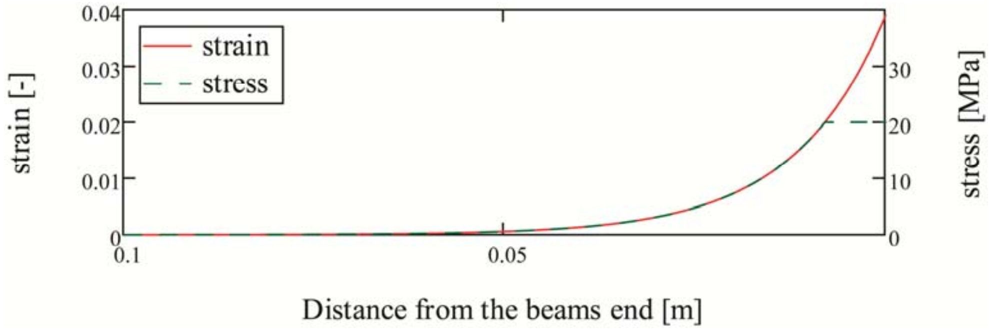


Figure 9: Shear stress and strain distribution (immediately before delaminating due to prestress force) in the adhesive layer towards the end of the beam calculated using the equations presented by Triantafillou and Deskovic [23].

For the ultimate anchorage load the shear strain at the end of the beam is equal with the ultimate shear strain ($\gamma_{u,a}$). This condition allows the calculation of the length of the elastic domain (l_{el}) (equation).

$$l_{el} = \frac{2 \cdot ln\left(\frac{\beta + \sqrt{\beta^2 + 4}}{2}\right)}{\omega} \tag{11}$$

$$\beta = \frac{2 \cdot \gamma_{el,a}}{\gamma_{u,a}} \cdot sinh\left(\frac{\omega \cdot l}{2}\right) \tag{12}$$

Where:
$\gamma_{u,a}$ = ultimate shear strain of the adhesive (Figure 8)
$\gamma_{el,a}$ = maximal elastic shear strain of the adhesive (Figure 8)
l_{el} = length of the elastic domain
ω = is given by equation (9)

Based on the shear stress distribution in the elastic domain the normal stress in the CFRP can be determined. Triantafillou et al. assumed the normal stress distribution in the inelastic zone as linear. Knowing the normal stress distribution in the CFRP allows the calculation of the ultimate pretension force ($P_{0,u}$) (equation (13)).

$$P_{0,u} = EA_{cf} \cdot h_a \cdot \gamma_{el,a} \cdot \omega \cdot \left(coth\left(\frac{\omega \cdot l_{el}}{2}\right) + \frac{\omega \cdot (l - l_{el})}{2}\right) \tag{13}$$

Where:
$P_{0,u}$ = ultimate pretension force
EA_{cf} = tension stiffness of the CFRP
h_a = thickness of the adhesive layer
$\gamma_{el,a}$ = maximal elastic shear strain of the adhesive (Figure 8)
l_{el} = length of the elastic domain
ω = is given by equation (9)

Furthermore Triantafillou and Deskovic [23] present an equation to calculate the ultimate pretension force ($P_{0,u}$) assuming concrete failure (equation (14)).

$$P_{0,u} = \frac{\tau_{u,c} \cdot (l - l_{el})}{4h_{cf}} \cdot \left(A_{cf} + \alpha \cdot \frac{EA_{cf}}{E_c}\right) + \frac{EA_{cf} \cdot h_a \cdot \tau_{u,c} \cdot \omega}{G_a} \cdot coth\left(\frac{\omega \cdot l_{el}}{2}\right) \tag{14}$$

Where:
$P_{0,u}$ = ultimate pretension force
EA_{cf} = Tension stiffness of the CFRP
h_a = thickness of the adhesive layer
$\gamma_{el,a}$ = maximal elastic shear strain of the adhesive (Figure 8)
l_{el} = length of the elastic domain
ω = is given by equation (9)

Triantafillou and Deskovic [24] investigated the maximal anchor resistance for prestressed CFRP laminates on European beach (*Fagus Silvatica*) using adhesively bond and assuming timber shear failure. The shear stress strain behaviour of wood was assumed as bilinear with an elastic start followed by an ideal plastic branch (Figure 8). The equations for timber failure are the same as they presented for adhesive failure [23] (equations (8) to (13)) only the stress-strain relationship of wood has to be used instead of the one of the adhesive. In case the CFRP-lamella is not as wide as the timber beam equation (9) has to be modified (equation (15))

$$\omega = \sqrt{\frac{G_a \cdot b_s}{h_a \cdot A_{cf}}\left(\frac{1}{E_{cf}} + \frac{4 \cdot A_{cf}}{AE_s}\right)} \tag{15}$$

Where:
G_a = shear module of the adhesive
h_a = thickness of the adhesive layer
A_{cf} = cross section area of the CFRP-lamella
E_{cf} = modulus of elasticity of the CFRP-Lamella
b_s = width of the substrate
AE_s = tension stiffness of the substrate (area multiplied by the module of elasticity)

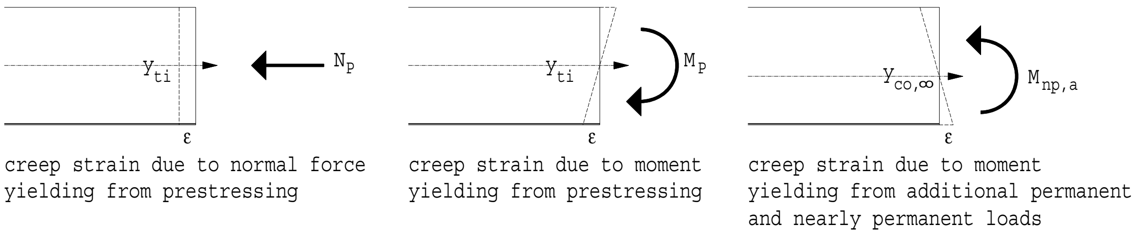
Triantafillou and Deskovic [23, 24] report that the theoretical investigations were satisfyingly verified with experiments using rather small and clear specimens. Furthermore they tested three timber beams in bending in order to determine the influence of prestressed CFRP-lamellas on the bending capacity [24]. They conclude that in order to avoid sudden collapse the members have to be designed to yield in the compression first before fail by tensile fracture.

4 Long-term behaviour

Long term loads lead to creep in timber on the other hand CFRP-lamellas and the used epoxy adhesive are not prone to creep. In designing timber structures creep is regarded as an increase of strain and mostly only considered in the service limit state. In a composite composed of materials with different creep behaviour the creeping leads to strain increase in all materials and therefore in case of different creep behaviour to load redistribution over the time.

In case of retrofitting the load history must be taken in account for long term design and calculation. In reference to Navi and Heger [17] creep can be regarded as linear viscoelastic behaviour for loads up to 40% of the short term load bearing capacity. Therefore the long term loads which were already present in the timber beam prior to intervention[2] do not contribute to the long term effects that occur after intervention.

The creep relevant loads in buildings are the dead loads, the nearly permanent part of the live loads and the forces due to prestressing. Normally creep leads to an elevated strain. On the bottom side of the timber where the CFRP-lamella is bonded to the beam strain difference in the timber leads also to strain difference in the CFRP which leads to a difference in the forces present in the CFRP. The creep due to prestressing leads to an increase of the compression strain in the timber and therefore leads to a reduction of the prestress. This reduction reduces the stress in the timber and therefore reduces also the strain. The internal forces must always be in equilibrium. The permanent and nearly permanent loads also lead to creep in the timber. Therefore the strain on the tension side of the timber increases which leads to higher strain and stress in the CFRP-lamella and therefore to a load redistribution from the timber to the CFRP and a movement of the neutral axis of the section. This shift is only valid for additional permanent and nearly permanent loads[3]. Therefore the composite section has two different neutral axes one for creep relevant external loads and one for the other external loads. Again the action and the reaction of the composite beam are always in equilibrium. The two effects described above take place at the same time and place but go in the opposite direction. Figure 10 presents a sketch of the different creeping strains present in a prestressed timber beam. The long term prestress force (P_∞) can be expressed by equation (16). For beams reinforced without prestressing the long-term effects lead to load redistribution from the timber to the CFRP. This additional force in the CFRP can be handled like a prestress force for long-term engineering design.



creep strain due to normal force yielding from prestressing

creep strain due to moment yielding from prestressing

creep strain due to moment yielding from additional permanent and nearly permanent loads

Figure 10: Systematic sketch of the creep relevant forces and the yielding strain of the composite section.

[2] It is assumed that the creep of the long term loads present prior to intervention has already reached its limit at the time the retrofitting takes place which is regarded as time = 0.

[3] Additional permanent and nearly permanent loads stands for the difference between the permanent and nearly permanent loads present in the system prior to intervention and the permanent and nearly permanent loads present in the system after renovation

$$P_{\infty}(x) = P_0(x) + \Delta P_{\infty}(x) \tag{16}$$

Where:

$P_{\infty}(x)$ = prestress force present in the system after the long term effect have taken place (time = ∞) as a function of x

$P_0(x)$ = prestress force at time = 0 as a function of x

$\Delta P_{\infty}(x)$ = difference in prestress force due to long term effects as a function of x

The creep strain at the bottom face of the timber and the strain in the CFRP-lamella can be regarded as proportional as long as the adhesive is sufficiently rigid. Therefore the creep in the timber leads to a strain increase or decrease in the CFRP-lamella, this leads also to a different stress in the CFRP. This stress increase or decrease occurs without any adjustment of the loads; therefore the stress difference is regarded as a decrease or increase of the prestress. The adhesive layer experiences also a strain increase or decrease due to the creep of timber. The prestress force at the time = ∞ can be calculated using equation (17).

$$P_{\infty}(x) = P_0(x) + \frac{n_{ti\infty} \cdot \frac{\left(\psi_2 \cdot M_{ll}(x) + M_{dl}(x) - M_{np,pi}(x)\right) \cdot z_{P\infty}}{I_{y,co,\infty}} - P_0(x) \cdot \left(\frac{1}{A_{ti}} + \frac{a^2}{I_{y,ti}}\right)}{\frac{MOE_{ti}}{\varphi \cdot \left(EA_{cf} + EA_{ad}\right)} + \frac{1}{A_{ti}} + \frac{a^2}{I_{y,ti}}} \tag{17}$$

Where:

$P_{\infty}(x)$ = difference in prestress force due to long term effects as a function of x

$n_{ti\infty}$ = ratio between the long term MOE of the timber and the reference MOE

ψ_2 = reduction factor for the nearly permanent value of a variable action as defined in SIA 260 [13]

$M_{ll}(x)$ = Moment due to live loads after intervention as a function of x

$M_{dl}(x)$ = Moment due to dead loads after intervention as a function of x

$M_{np,pi}(x)$ Moment due to dead loads prior intervention as a function of x

$z_{P\infty}$ = distance between the neutral axis of the composite section at time = ∞ and the centre of gravity of the prestress force

$I_{y,co\infty}$ = moment of inertia of the composite section at time =

$P_0(x)$ = prestress force after removing the in the height adjustable prop (time = 0) as a function of x

A_{ti} = area of the timber cross section

a = eccentricity of the prestress force

$I_{y,ti}$ = moment of inertia of the timber section

φ = creep factor

MOE_{ti} = Modulus of elasticity of the timber

EA_{cf} = MOE of the CFRP-lamella multiplied with the cross section area of the CFRP-lamella

EA_{ad} = MOE of the adhesive multiplied with the cross section area of the adhesive layer

5 Verification of the composite section

The building codes require verifying the compression and the tension stress resulting from bending. In this particular case the Swiss timber construction standard (SIA 265) [12] requests considering the bending in combination with the compression resulting from prestressing. This verification is done using equation (18) [12].

$$\left(\frac{\sigma_{c,0,d}(x)}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,d}(x)}{f_{m,d}} \leq 1 \tag{18}$$

Where:

$\sigma_{c,0,d}(x)$ = design stress in compression parallel to the grain as a function of x

$f_{c,0,d}$ = design value in compression parallel to the grain according to SIA 265 [12]

$\sigma_{m,d}(x)$ = design stress in bending as a function of x

$f_{m,y,d}$ = design value in bending according to SIA 265 [12]

The verification of the compression side of a reinforced bending beam leads in timber construction to a very conservative approach. The experiments described by various authors [2-8, 14, 16, 19-21, 24] showed that a beam reinforced with one CFRP lamella fails because of tension failure. Furthermore a violation of the design value at the compression side leads to crushing of the timber fibres and larger deformation but not to a complete failure, which is similar to the behaviour of reinforced concrete. Eurocode five [9] allows considering of the ductile behaviour of timber in compression. Unfortunately in the Swiss standard this is not explicit mentioned. Therefore the author suggests using the tension stress in the timber to verify the reinforced timber beam. The verification is done using equation (19).

$$\frac{\sigma_{d,b,ti}(x)}{f_{m,d}} \leq 1 \tag{19}$$

Where:

$\sigma_{d,b,ti}(x)$ = design normal stress at the bottom of the timber section due to prestressing and loading as a function of x

$f_{m,d}$ = design value in bending according to SIA 265 [12]

The use of equation (19) leads to a significant higher design bending capacity then the use of the conservative method requested by the SIA 265 [12]. Figure 11 shows the significant difference between the two methods of verifying a reinforced timber beam.

The equations (18) and (19) are also valide for the verification at time = ∞. As mentioned above the creeping of the timber leads to a shift of the neutral axis. This shift is valid only for the additional permanent and nearly permanent loads for all other loads the neutral axis calculated at time = 0 is still valid. Therefore the ultimate limit state design at time = ∞ is done using the neutral axis at time = 0 and at time = ∞. Furthermore the internal and external forces cannot just be added at Time = ∞ but the stresses have to be added in order to validate the cross section. The maximal tension stress at the bottom of the timber is calculated using equation (20).

$$\sigma_{b,ti,\infty}(x) = \frac{\left(M_d(x) - M_{np,a}(x)\right) \cdot z_{b,ti}}{I_{y,co}} + \frac{M_{np,a}(x) \cdot z_{b,ti,\infty}}{I_{y,co,\infty}} + \frac{M_{P\infty,ti}(x) \cdot h_{ti}}{2 \cdot I_{y,ti}} \qquad (20)$$

Where:

$\sigma_{b,ti,\infty}(x)$ = normal stress at the bottom of the timber due to loading and prestressing at time = ∞
$M_d(x)$ = design moment as a function of x
$M_{np,a}(x)$ = moment due to additional permanent and nearly permanent loads as a function of x
$I_{y,co}$ = moment of inertia of the composite section at time = 0
$z_{b,ti}$ = distance between the neutral axis of the composite section and the bottom of the timber section at time = 0
$z_{b,ti,\infty}$ = distance between the neutral axis of the composite section and the bottom of the timber section at time = ∞
$I_{y,co,\infty}$ = moment of inertia of the composite section at time = ∞
$M_{P\infty,ti}(x)$ = internal moment in timber yielding from eccentric prestressing as a function of x at time = ∞
h_{ti} = height of the timber section
$I_{y,ti}$ = moment of inertia of the timber section

The verification on the compression side of the prestressed bending beam can be done using equation (18), where $\sigma_{m,d}$ is the sum of the bending stresses yielding from the creep relevant and creep irrelevant loads. Again the verification of the compression side of the timber is a conservative approach (Figure 11) and should be questioned. Unfortunately not many long term bending tests with adjacent determination of the ultimate load are done so far. Lehmann [15] presents mechanical properties of an historic timber beam the properties were determined using small clear specimens cut from one beam. The results allow the assumption that historic timber has also a quite extended non-linear behaviour in compression parallel to the grain. Failure on the tension side leaded to complete failure of the bending specimen. Not all bending specimen showed extensive non-linear behaviour towards the end. The results presented by Lehmann [15] underline that a verification of the tension side only should at least be considered.

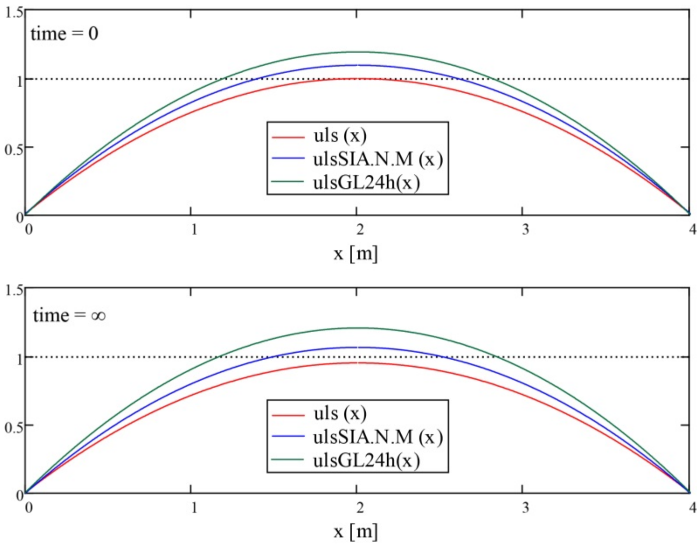


Figure 11: Results of equation (18) (blue) and (19) (red) for a reinforced beam at time = ∞ (160mm x 120mm GL24h reinforced with one 1.2mm thick 50mm wide CFRP-lamella). The green line represents an unreinforced glulam beam (160mm x 120mm). A value of one means the design capacity of the beam is reached. (All beams are equally loaded.)

References

[1] Arbeitsgruppe SIA 162-8 Klebebewehrungen 2004: Vornorm SIA 166:2004, Klebebewehrungen; SIA Schweizerischer Ingenieur- und Architektenverein, Zürich

[2] Blass H. J., Krams J. und Romani M. 2002: *Verstärkung von BS-Holz-Trägern mit horizontal und vertikal angeordneten CFK-Lamellen;* Bautechnik, **79**(10) p.684-690

[3] Blass H. J. und Romani M. 2000: *Trag- und Verformungsverhalten von Verbundträgern aus Brettschichtholz und faserverstärkten Kunststoffen;* Forschungsbericht, Universität Fridericiana Karlsruhe, Karlsruhe

[4] Blass H. J. und Romani M. 2001: *Tragfähigkeitsuntersuchungen an Verbundträgern aus BS-Holz und Faserverbundkunstoff-Lamellen;* Holz als Roh- und Werkstoff, **59** p.364-373

[5] Blass H. J. und Romani M. 2002: *Biegezugverstärkung von Brettschichtholz mit CFK- und AFK-Lamellen;* Bautechnik, **79**(4) p.216-224

[6] Borri A., Corradi M. und Grazini A. 2003: *FRP Reinforcement of Wood Elements under Bending Loads*; in proceedings of the *Structural Faults and Repair* London.

[7] Borri A., Corradi M. und Grazini A. 2005: *A method for flexural reinforcement of old wood beams with CFRP materials;* Composites: Part B, **36**(2) p.143-153

[8] Brunner M. und Lehmann M. 2008: *Strengthening glulam beams with prestressed multilayer FRP carbon laminates*; in proceedings of the *Final Conference of COST E34 Bonding of Timber*; Sopron.

[9] CEN/TC 250 "Structural Eurocodes" 2005: SN EN 1995-1-1:2004, Eurocode 5 Bemessung und Konstruktion von Holzbauten - Teil 1-1: Allgemeine Regeln für den Holzbau; SIA Schweizerischer Ingenieur- und Architektenverein, Zürich

[10] Dziuba T. 1985: *The Ultimate Strength of Wooden Beams with Tension Reinforcement;* Holzforschung und Verwertung, **37**(6) p.115-119

[11] Holzenkämpfer P. 1994: *Ingenieurmodelle des Verbunds geklebter Bewehrung für Beton Bauteile*; Dissertation, Technischen Universität Braunschweig, Braunschweig

[12] Kommision SIA 164 "Holzbau" 2003: SIA 265:2003, Holzbau; SIA Schweizerischer Ingenieur- und Architektenverein, Zürich

[13] Kommission SIA 160 "Einwirkungen auf Tragwerke" 2003: SIA 260:2003, Grundlagen der Projektierung von Tragwerken; SIA Schweizerischer Ingenieur- und Architektenverein, Zürich

[14] Lehmann M. 2006: *Renforcement sur site de poutres en bois avec lamelles de carbone précontraintes*; MSc. Thesis, Université Henri Poincaré, Nanncy

[15] Lehmann M. 2008: *LGA Holzbalken, Bestimmung von einigen mechnaischen Eigenschaften eines Historischen Holzbalkens;* Prüfbericht (confidential), 7774-SB-01; Berner Fachhochschule, Biel/Bienne

[16] Lehmann M. 2012: *Investigations of the Load Bearing Behaviour of Timber Bending Beams Reinforced Using Prestressed CFRP-Lamellas*; Dissertation (submitted for approval), Bauhausuniversität Weimar, Weimar

[17] Navi P. und Heger F. 2005: *Comportement thermo-hydromécanique du bois: applications technologiques et dans les structures*, Presses Polytechniques et Universitaires Romandes, Lausanne

[18] Plevris N. und Triantafillou T. C. 1992: *FRP-Reinforceed Wood as Structural Material;* Journal of Materials in Civil Engineering, **4**(3) p.300-317

[19] Romani M. und Blass H. J. 2001: *Design Model for FRP Reinforced Glulam Beams*; in proceedings of the *CIB-W18/34-12-3*; Venice.

[20] Schnüriger M., Brunner M. und Lehmann M. 2007: *Gradientenvorspannung; Verstärkung von Holzbalken mit vorgespannten CFK-Lamellen welche in Gradienten Verankert sind;* Forschunungsbericht, Berner Fachhochschule, Biel

[21] Schober K.-U. und Rautenstrauch K. 2006: *Post-strengthening of timber structures with CFRP's;* Materials and Structures, **40**(1) p.27-35

[22] Thomsing M. 1998: *Spannbeton; Grundlagen - Berechnungsverfahren - Beispiele*, B.G. Teubner, Stuttgart

[23] Triantafillou T. C. und Deskovic N. 1991: *Innovative Prestressing with FRP Sheets: Mechanics of Short-Term Behavior;* Journal of Engineering Mechanics, **117**(7) p.1652-1672

[24] Triantafillou T. C. und Deskovic N. 1992: *Prestressed FRP Sheets as External Reinforcement of Wood Members;* Journal of Structural Engineering, **118**(5) p.1270-1284

[25] Triantafillou T. C., Matthys S., Audenaert K., et al. 2001: *fib bulletin 14; Externally bonded FRP reinforcement for RC structures*, International Federation for Structural Concrete (fib), Lausanne

Fibre Reinforced Polymers for the Strengthening of Wood Structures: A General Overview

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Summary

Fibre reinforced polymer (FRP) systems are widely used in structural strengthening, with application on reinforced concrete, masonry, wood and other substrates. For wood strengthening, carbon fibre reinforced polymer (CFRP) plates are often used for flexural strengthening, and carbon fibre fabrics offer an interesting solution for shear strengthening. Carbon fibres are well suited for the strengthening of wood, because they are small and easy to install, but have a large stiffness difference to wood and usually don't need special protection.

While there is only little information on the design given in standards, planning of a wood strengthening project is possible with the available guideline and the experience of the many reference projects.

Key words: FRP, Wood, Structural strengthening, Carbon fibres

1 Introduction

Composites made from fibre reinforced polymers (FRP) have been used in structural strengthening since over 20 years. Different fibre and matrix resin types are used on various substrates, and the most common application is the strengthening of reinforced concrete with carbon fibre reinforced polymer (CFRP)

Strengthening solutions with different FRP are widely used and considered standard practice in many cases. They are used more and more also on other substrates than reinforced concrete – such as steel, masonry, natural stone and wood.

Many different structural problems can be approached and solved with a FRP strengthening solution. Reasons for strengthening are for example different use or improvement of a structure, repair of damaged members, mistakes in design or construction, fulfillment of stricter standards or the ability to take higher loads.

In wood strengthening, usually the stiffening of an element, a reduction in deflection or shear strengthening is the goal. In such cases, CFRP composites are very well suited, as they generally have a small volume and offer a big stiffness difference compared to wood. The CFRP materials have very constant, closely monitored mechanical properties, are installed quickly and easily, they are lightweight, and (in case of carbon fibres) not subject to corrosion. Additionally, one can fall back on the experience of the last two decades, both in research projects and applications worldwide.

Figure 1: Wooden bridge, Sins, bottom view

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2 Overview strengthening solutions

In structural strengthening, different fibres (carbon, glass, aramid, basalt, natural fibres) and matrix resin systems are used. Carbon and glass fibres (see

Table 1) are most widely spread, usually with an epoxy based matrix and adhesive.

The fibres are used either as prefabricated composites such as rods or plates, or as dry fabrics, pre-pregs or fibre strings. The dry fibre products are impregnated with the corresponding matrix resin on site and applied to the substrate directly, forming the final composite during application. The preformed composites are applied using an adhesive and, if specified, mechanical fastenings.

The different systems can either be externally applied or put into slits in the substrate (near surface mounted application). The substrate preparation is important for both installation types. For externally applied systems, the application area of the substrate must be free of oil, grease, dust and any loose material. Coatings such as paints or protective layers can affect the bond negatively and also have to be removed before application. If a fabric is applied around a corner, the edge has to be rounded to avoid the fibres from bending and breaking.

FRP profiles that are installed into slits or grooves in the substrate surface need a different substrate preparation. Paints and coatings are not a problem in this case, but every cut slit must be cleaned from sawdust to guarantee a good bond.

Epoxy resins are most often used for the application of FRP plates and for the impregnation of dry fibre fabrics. These resins consist of two components which need to be mixed thoroughly before application, and they are suitable for the application on concrete, masonry, steel and wood. For the installed strengthening system to be able to take load, the adhesive or impregnation resin needs to be fully cured, which can take anywhere from a few hours to several days, depending on the product and the ambient temperature.

For the use on wood substrates, mainly flexural and shear strengthening are of interest.

Table 1: Mechanical properties of Carbon and glass fibres

		Carbon		Glas
		T700S	**T800S**	**E-Glas**
Tensile strength	MPa	4900	5490	2300
E-Modulus	GPa	230	294	73
Elongation at break	%	2.1	1.9	2.2 – 2.5
Density	g/cm3	1.8	1.8	2.6

Figure 2: Substrate preparation

Flexural Strengthening

FRP plates can be used for flexural strengthening of wood in two different ways, either as externally applied plates, or as embedded plates or rods.

The externally applied plates are installed in exactly the same was as known from reinforced concrete: The plate is cut to length, coated in adhesive and pressed onto the prepared substrate. By carefully pressing it on using a rubber roller, air voids are eliminated and a good connection between plate and substrate is achieved.

Near surface mounted plates and rods can be installed on the bottom or at the sides of beams. Especially older wooden beams can be strongly curved, and installation on the bottom is not always possible. The main advantage of near surface mounted plates compared to externally applied plates is that they are nearly invisible after installation. Only an adhesive line of a few millimeters width which can easily be coated or otherwise covered remains.

Shear Strengthening

FRP in general and fabrics in particular are well suited for shear strengthening, because they can be applied to any shape of substrate, and installed so the fibres are loaded in tension. In case of shear strengthening, fabrics are better suited than plates as the larger bond area helps transfer the load from the substrate into the strengthening system. Additionally, the fabrics can be applied around the beam edges for a better end-anchorage. The fabrics are either impregnated directly on the substrate or placed on it pre-impregnated, and the impregnation resin also serves as adhesive for the system. Since no primer or extra adhesive is necessary, the installation is very fast and simple. Unlike with the FRP plates, this strengthening system is clearly visible and can either be left as is, or be painted or coated as needed.

Figure 3: Externally applied plates

Figure 4: Fabric on wooden beam

3 Selected projects

Other than for residential buildings, the strengthening of wood structures is mainly interesting for historical buildings and structures with monument protection.

One of the first projects in Switzerland where CFRP plates were used is the **wooden bridge over the Reuss river in Sins, AG**. This wooden bridge was built in 1807 and used frequently. Until 1996, vehicles up to 28 tons were allowed to cross the bridge, which lead to significant deflections of the transverse beams. As a part of the general refurbishment and improvement of the whole structure, the affected beams were retrofitted with Sika CarboDur plates. The repair and refurbishment works were not only carried out to preserve the structure, but also to improve it so vehicles up to 40 tons could cross it safely. The application of CarboDur plates has led to a reduction in deflection without changing the appearance of the structure. The system has been left without protective coating or further maintenance needed for over 20 years.

Another typical application of CFRP on wood is the strengthening of the wood floors in the **Schönhof Görlitz, Germany**. This building is the oldest renaissance building in Görlitz and was renovated to be used as a museum. To accommodate the higher service load, the wooden beams of the original ceiling were strengthened with CarboDur plates. This was only a small change to the buildings appearance, but made the different use of the building possible.

Figure 5: Wooden bridge, Sins

Figure 6: Schönhof Görlitz

4 Special conditions and protection

While the CFRP strengthening systems are corrosion resistant and are not sensitive to heat and humidity, sometimes additional protection is necessary or advisable.

If a CFRP system is exposed to direct sunlight, a UV-protective coating should be applied, as the matrix can get damaged over the years. The same is true for standing water and chemical attacks. If the matrix is damaged, the load cannot be transferred from fibre to fibre effectively, and the efficiency of the system decreases dramatically. In most cases the systems are applied underneath bridges or in buildings however, where the environment is not aggressive or the system will be covered. There, a special protective coating is not necessary.

In some cases, the reaction to fire is also an issue. Since the glass transition temperature of the adhesive often lies around 50° C, the strengthening system will not be effective at high temperatures. It is very rarely the case that a strengthening system must be able to take the full load in case of a fire, so extra protection is usually not necessary. This has to be verified on a project basis.

5 Research and Design

Different universities in Europe have been involved in research regarding the strengthening of wood with FRP composites. Most commonly investigated is flexural strengthening (with externally applied plates and near surface mounted plates) or pre-stressed systems, in fewer cases shear strengthening is chosen as research focus. Simulations and design basics have also been proposed.

For design and calculation, the Italian guideline CNR-DT 201/2005, [1] is usually used as basis, as the local standards don’t give much information.

Selection of References and Reports:

[1] http://www.cnr.it/sitocnr/Englishversion/CNR/Activities/RegulationCertification.html

[2] J. acob, 2007: „Flexural Strengthening of Glued Laminated Timbre Beams with Steel and Carbon Fibre Reinforced Polymers”, Chalmers University of Technology, Sweden, Masters Thesis

[3] M. Lehmann et al., 2006: “Strengthening of timber structures using pre-stressed carbon fibre lamellas”, BFH-AHB, Switzerland

[4] Q. Xu, 2009: “Strengthening Timber Beams with Near Surface Mounted Carbon Fiber Reinforced Polymer Rods”

[5] A. Wäger, 1996: “Verstärken von Biegeträgern aus Holz mit Faserverbundwerkstoffen”, Diplomarbeit SISH, Biel

[6] T. Triantafillou, 1997: „Shear Reinforcement of Wood Using FRP Materials“, Journal of Materials in Civil Engineering / May 1997 / 65

Strengthening with wood and wood based panels

Robert Widmann[1]

Summary

In this paper the use of glued wood based panels, solid timber members and additional glulam lamellae for strengthening and reinforcement of glulam members is being discussed. The content of the paper is practically oriented. It is shown how this strengthening technique is being applied and also respective existing design models are proposed.

Key words: Glulam, Strengthening, Reinforcement, Wood based panel, Additional lamellae, Screw-press bonding

1 Introduction

The strengthening of elements of glulam with the help of wood based panels and/or with additional (wood) lamellae is a standard procedure in timber engineering. It is used both in the course of the renovation as well as for manufacturing of new components. This normally results in the required capacity and / or stiffness is being reached or restored by a (subsequent) enlargement of the original cross section. Most often, the strengthening is applied in strip and / or plate-like shape. In general it is possible to apply the wood based panels with metal fasteners like screws and nails or with the use of adhesives. Due to stiffness requirements the bonded version of the strengthening is preferred as this kind of strengthening can be attributed as being very stiff. On the other hand, as it is visible, it is often not first choice if architectural aspects also have to be considered. In this paper the strengthening of glulam members with glued wood based panels and with glued additional lamellae is being discussed with a practical background.

2 Material and method: Wood based panels

Wood-based panels applied by screw-press bonding often come in first line to be used when reinforcements acting on the surface of a member are needed. Main applications are, for repair as well as for new members, strengthening and reinforcements regarding:

- Tension perpendicular to grain
- Compression perpendicular to grain
- Shear
- Embedment

Apart in the case of refurbishment of cracked or delaminated (glulam) members most often this kind of reinforcement is being used for (Figure 1):

- Notched members
- Members containing holes
- Double tapered beams
- Curved beams
- Pitch cambered beams

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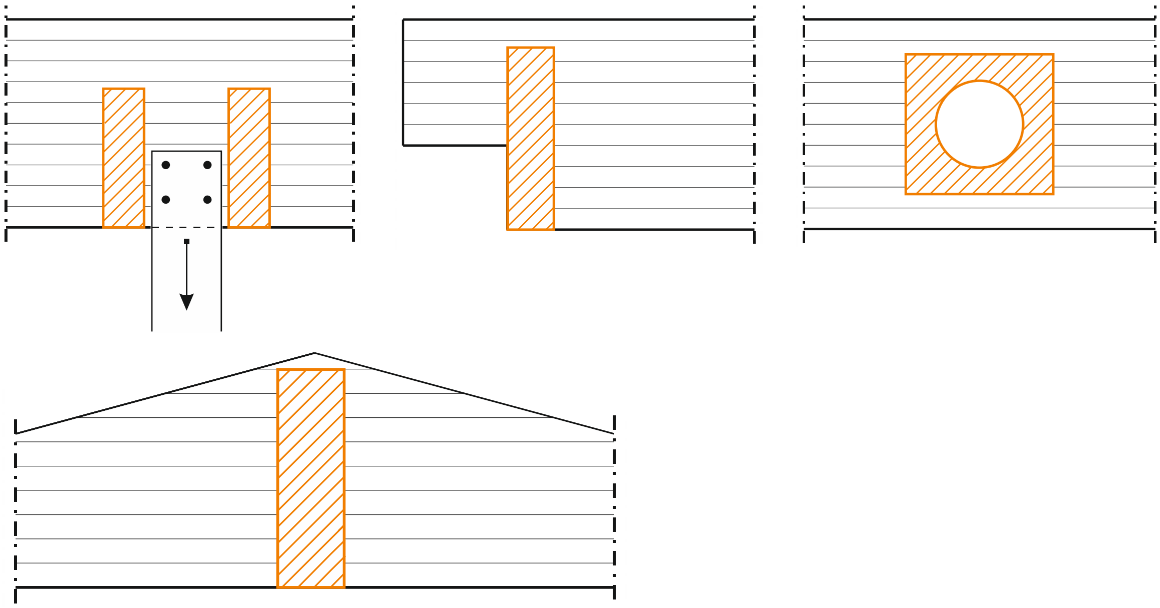

Figure 1: Classical application of wood based panels for strengthening of cross-connections, notched members, members that contain holes as well as double tapered, curved or pitch cambered beams.

Wood based panels are also used to reinforce nodes or connection areas of wooden structures. Most often the application is done from the outside and the strengthening is therefore visible. Recently, however, such reinforcements were also increasingly designed as inside members (e.g. node TA Media, Figure 2).

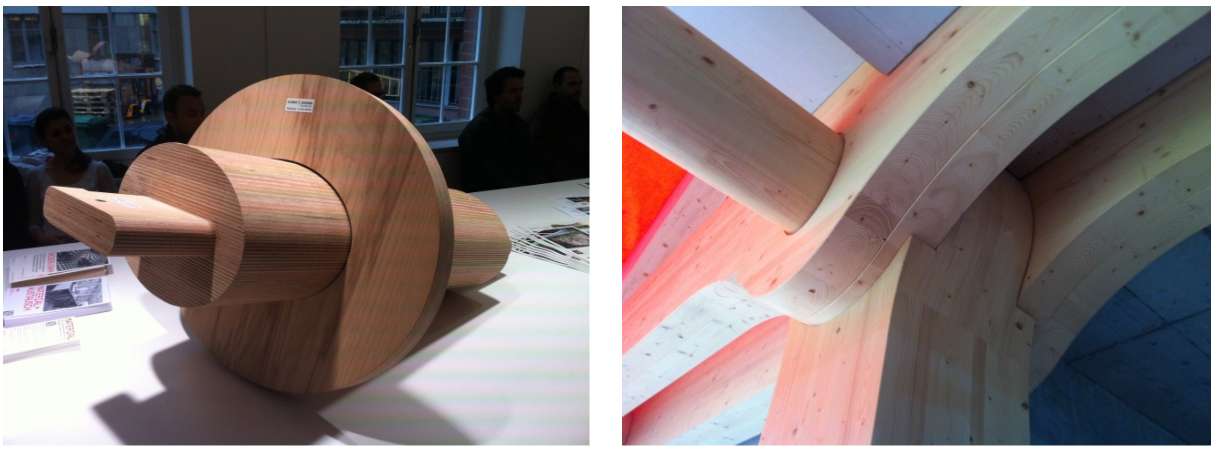

Figure 2: Node at the Shigeru Ban building for TA Media in Zürich (Engineering sjb kempter, fitze AG, Realization: Blumer Lehmann AG). The special LVL-type element shown on the left hand side is completely embedded in the node (right hand side) and guarantees the transfer of the loads of the horizontal elements into the columns without exceeding the perpendicular to the grain strength of the members.

Material: Strengthening members

Solid members as well as wood based panels are suitable for these kinds of strengthening. If solid wood is going to be used, its depth should not exceed a maximum of 35 mm. Solid members most often will be single board (lamellae) of a high strength class. In general softwood as well as hardwood boards (lamellae) are suitable. Regarding wood based panels, most often high strength products are being used like CLT, LVL or Kerto. Also for these products some limitations apply. Layer thickness of LVL and Kerto may not exceed 5 mm and a minimum total thickness of 10 mm is required whereas layer thickness in CLT should be 30 mm at maximum.

Material: Adhesives

The choice of a suitable adhesive is a crucial point for this kind of strengthening. The leaflet "Rehabilitation of timber glulam elements" of the Studiengemeinschaft Holzleimbau eV [5] proposes the use of adhesives that fulfil certain requirements.

Aminoplast- or phenoplast- resins can be used if they meet the corresponding requirements. This is given, if they comply with e.g. a general technical approval (in Switzerland not necessary). Common in many rehabilitation works is the use of two-component epoxy adhesives. This resin has a high viscosity during the reaction and thus fills cavities very well. When gluing, no clamping pressure is required because this adhesive - unlike polycondensation-resins – does not show any shrinking. The high viscosity of epoxies results in the possible leaking of the adhesive which must be prevented by suitable measures. During the renovation of remaining visible components the used adhesives can be mixed with a dye in order to achieve the desired colour. Also in this case the properties of the coloured adhesive have to be tested and found to be suitable. If adhesives, e.g.in order to be to thickened, are mixed with extenders, this is only permitted if the thickened adhesive has been tested and has been classified as suitable. All used adhesives shall correspond to Type I according to EN 301: 2013 [6].

Recently also two-component polyurethane adhesives are increasingly being used for such reinforcements. For the purposes of screw-press bonding several providers in Switzerland and Europe have corresponding products available. An important criterion for the selection of the adhesive is, inter alia, the glue line thickness. The adhesive should be tested and have successfully shown to be appropriate for glue line thicknesses of at least 2 mm.

3 Material and methods: Additional lamellae

With the application of additional lamellae (Figure 3 and Figure 4) the bending strength and stiffness of existing glulam beams can be improved significantly. If there is no restriction in depth of a beam this most often is the most efficient way to strengthen a glulam beam. Otherwise CFRP or steel lamellae have to be considered. One advantage of using timber lamellae over (thin) steel or CFRP lamellae is of course the activation of geometric parameters, in particular increasing depth which effects bending strength with the power of two and bending stiffness with the power of three. In order to achieve a maximum strengthening effect it is recommended to use high quality lamellae for this purpose, e.g. such made of hardwood. Another possibility is to additionally strengthen the lamellae e.g. with glued in steel rods. However, where the normal case "additional lamellae" is easy to calculate and to design there are no standard procedures for the design of "high strength lamellae" (e.g. reinforced with glued-in steel rods) available and it might be necessary to confirm the performance by tests.

The application of the additional lamellae is relatively easy and usually the necessary compression in the glueline is being guaranteed by screws.

The enlargement of the cross-section also leads to a slightly increase in shear resistance. However, this is disregarded in most cases, among other reasons, because the additional lamellae usually cannot be applied over the whole length (including the support area, see Figure 3). On the other hand, due to this fact a notch will exist in the strengthened beam and the effect this notch must be taken into account for the static design.

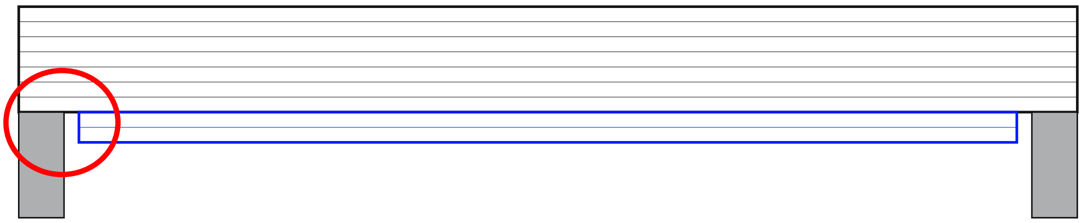

Figure 3: Principle of the application ff additional lamellae in order to reinforce or strengthen a beam regarding its bending performance. It has to be regarded that with this addition of lamellae also a notch is being introduced (red circle).

Figure 4: Application of (reinforced) additional lamellae for the strengthening of beams. Right hand side: laboratory tests, left hand side: application in practice.

4 Design

The design and calculation is a relatively easy procedure. From the stiffness conditions it results for most of the applications that the strengthening members have to carry 100% of the total load. This means that the load will be transferred from the wood over the glue line into the wood based panel and then back over the glue line into the wood member. Consequently the strengthening member has to be designed for carrying the respective load (most often tension and/or shear) and the glue line has to be designed for shear.

Despite being relatively "easy" there is no information on the design of these reinforcements available in Eurocode 5 [1] and only some National Annexes (e.g. the German Annex DIN EN 1995-1-1/NA:2010 [2]) provide the necessary data.

Based on the load that has to be transferred, the necessary cross section of the wood based panel(s) can be determined. Often it is not possible to account for 100% of the characteristic strength values of such a panel, but the strength (e.g. tension strength $f_{t,k}$) has to be reduced to 2/3rds for reinforcements of lateral connections or to 50% for reinforcements of notches and holes (Table 1). No reduction has to be made for the strength of wood based panels that are being used for strengthening of arched beams or beams with an inclined compression zone. The main reason for the reduction of strength is the uneven distribution of the stresses within such a panel which has to be accounted for in design. In addition there are limitations to the size of wood based panels that are being used for reinforcements or strengthening. Often a strengthening is needed only at a locally limited area of a member and therefore it does not make sense to increase the size of wood based panel used for this purpose to any dimension. This limit in size is often a decisive factor for the design of reinforcements with the use of wood based panels as the size restriction for the wood based panel at same time limits the available glued area.

Regarding the strength of the glue line the uneven stress distribution within the bonded area leads to similar – additional – limitations. The characteristic value for the glue line strength has also to be reduced to 50% of the "standard value".

Table 1: Characteristic glue line strength for different kinds of reinforcements and factor for the strength reduction of wood based panels (DIN EN 1995-1-1/NA:2010 [2], no values available in Swiss standard SIA 265:2012[3]).

Reinforcement of	**Charakteristic bond line strength $f_{k,k}$ [N/mm^2]**	**Factor for reduction of characteristic tension strength $f_{t,k}$ of strengthening member**
Notches and holes	0.75	0.5
Lateral connections	0.75	0.67
Double tapered, curved and pitch cambered beams	1.50	1

Already the "standard value" with $f_{k,k}$ = 1.5 N/mm^2 represents a conservative value and stays well below the characteristic shear strength of structural timbers and as well below the shear strength of bond lines in glulam that is being determined according to EN 14080:2013 [4]. On the other hand it is clear why and that such reduced values have to be used. The production or application of such a kind of strengthening involves different conditions compared to a glulam production. This comprises important parameters as bond line thickness, bond line pressure, surface conditions, on site application, application temperature and moisture content, cleanliness, and several more.

5 Application

The performance of the strengthening is essentially dependent on that the large bonding area is being applied according to the manufacturers (of the adhesive) instructions and requirements. The preparation contained of surface of existing glulam members can represent a significant amount of work especially for older elements.

Figure 5: Screw-press bonding for the application of additional lamellae for a glulam beam. Left hand side: application of the PUR adhesive, right hand side: setting of the screws in a predefined pattern.

For screw-press-bonding it has to be ensured that the required compression of the adhesive can be introduced into the bond line. The force that can be applied on the bond line depends strongly on the type, number and distribution of the screws over the whole strengthening section. Neither in Eurocode 5 [1] nor in Swiss standards [3] there is a requirement regarding these screws and their properties. According to the leaflet "Rehabilitation of timber glulam elements" of the Studiengemeinschaft Holzleimbau eV [5] at least one screw with a diameter greater than 4 mm per 150 cm^2 (with a maximum edge length of the screw grid of 150 mm) has to be set (Figure 5). In the glued plate or the glued lamella no screw thread may be present in order to prevent blocking of the compression effect. The length of the thread in the timber member that contains the screw tip must be at least 40 mm, however, be at least equal to the plate or lamella thickness. In case of a multi-layer strengthening the screw pattern should be arranged in a staggered layout. The screws can be removed after curing, as they don't contribute to the load transfer. On the other hand, if they are not being removed, their presence can stabilize the glued zone to a certain degree, e.g. regarding stresses perpendicular to the grain.

In principle, it is possible to apply the specified adhesive stress in other ways than by screw-press bonding, but for example by clamps, hydraulic or electric actuators, vacuum devices and similar techniques. However, in practice, screw-press bonding has been established as the most economical solution.

6 Acknowledgement

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References

[1] EN 1995-1-1/A1:2008: Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings.

[2] DIN EN 1995-1-1/NA:2010: National Annex - Nationally determined parameters - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings

[3] SIA 265:2012 Timber Structures.

[4] EN 14080:2013: Timber structures - Glued laminated timber and glued solid timber - Requirements.

[5] Studiengemeinschaft Holzleimbau e.V, (2010): Leaflet "Rehabilitation of timber glulam elements" in German: Merkblatt: Sanierung von BS-Holzbauteilen, Wuppertal, Deutschland, www.brettschichtholz.de.

[6] EN 301:2013: Adhesives, phenolic and aminoplastic, for load-bearing timber structures - Classification and performance requirements

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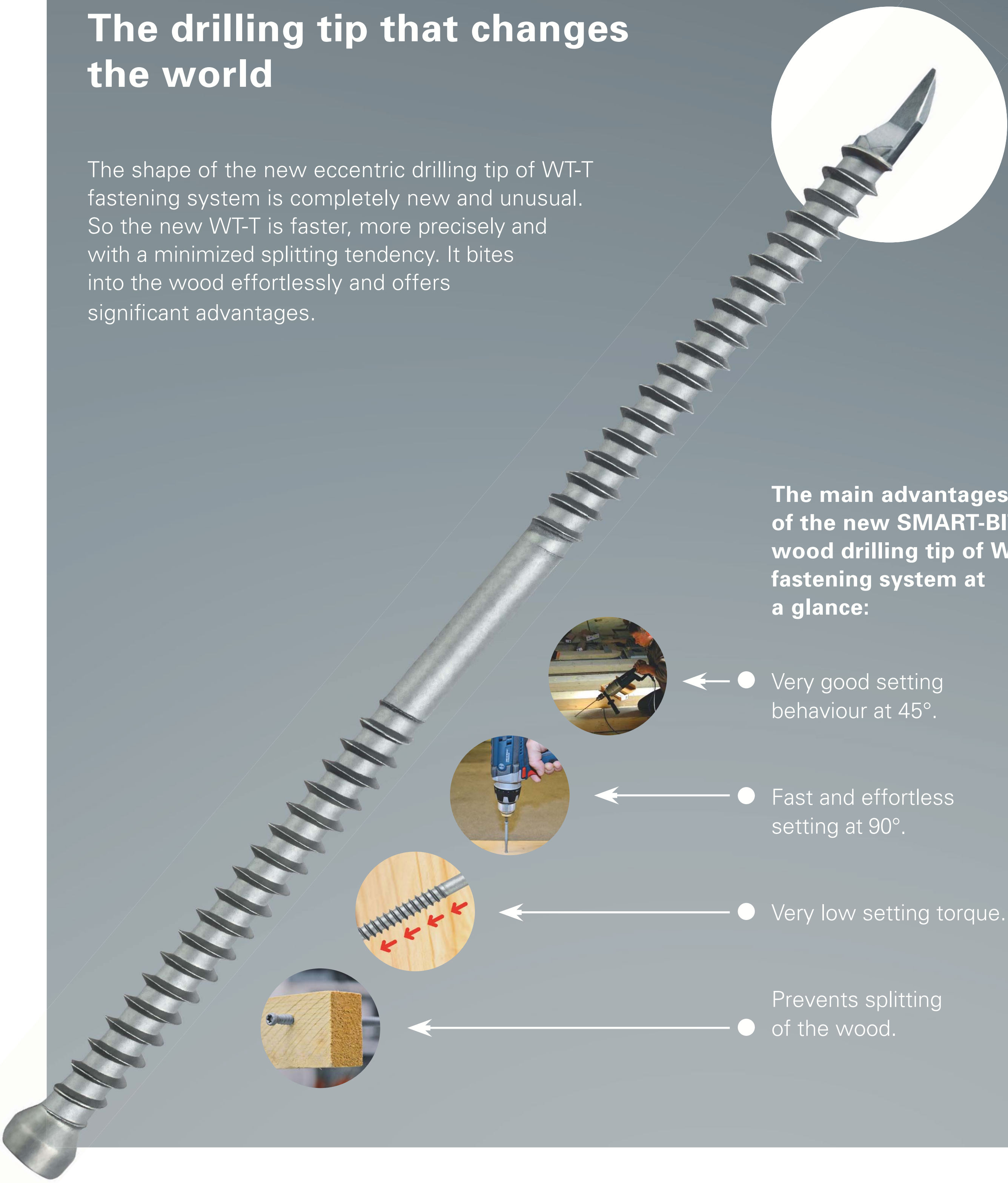

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