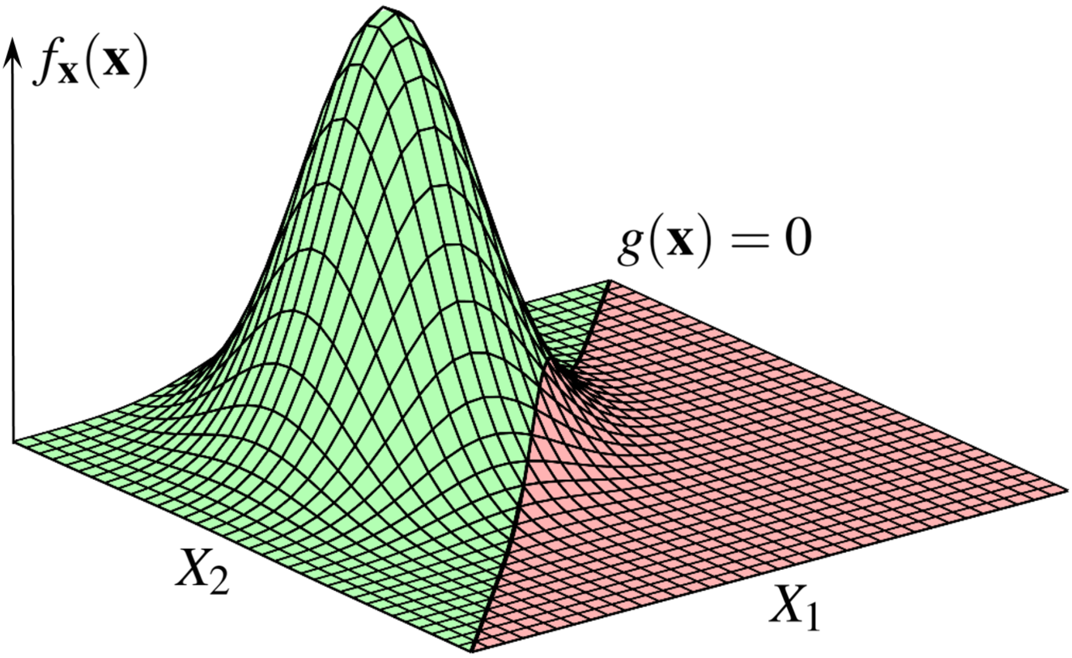
Basis of Design Principles for Timber Structures



Editors:
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Basis of Design Principles for Timber Structures

A state-of-the-art report by
COST Action FP1402 / WG 1

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Foreword

This report is a publication of the European Network COST FP1402 *Basis of Structural Timber Design – from research* to standards.

The COST Action FP1402 (website: http://www.costfp1402.tum.de/home) is a research network established under the aegis of the COST domain "Forests, their Products and Services". The aim of the Action was to overcome the gap between broadly available scientific results and the specific information needed by designers, industry, authorities and code committees, providing transfer for practical application in timber design and innovation.

This report represents the results of the activities performed in working group 1, *Basis of Design*. The most important task of working group 1 was the defragmentation and harmonization of techniques and methods that are necessary to prove the reliable, safe and economic application of timber materials or products in the construction industry.

This report is structured into five parts. At first general principles regarding the design formats are addressed (Part I). Afterwords timber specific aspects regarding code calibration (Part II) and serviceability (Part III) are summarized. In Part IV other demanding issues for the implementation into Eurocode 5 are addressed. Here also summaries of joint activities with other working groups on cross laminated timber and timber connections are presented. The report concludes with a guideline for data analysis (Part V).

Gratitude is addressed to the COST Office for funding the production of this report. The commitment and contributions of all working group members towards this report are greatly appreciated.

Gerhard Fink, Jochen Kohler, Chairs of working group 1, COST FP1402
Philipp Dietsch, Chair, COST FP1402

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The adressing of the principles of EN 1990 by EN 1995-1-1

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Review of Principles given in main body and annex A1 of EN1990

In the tables below each of the Principles in the main body and annex A1 of EN1990 is reviewed from the perspective of whether it has been followed through in EN1995-1-1. Bold (non-italic) print in the third column of the table highlights where EN1995-1-1 might be considered not to have fully implemented a Principle of EN1990 and bold-italic print highlights where the proposed work activities of COST FP1402-WG1 are relevant to the interface between EN1990 and EN1995-1-1.

In previous National Standards (certainly British Standards) the rules for satisfying safety, serviceability and durability criteria were given in individual material design codes and therefore concerned specific materials used in particular constructions. In the Eurocodes many of the 'head' rules are now given in EN1990 and are material-independent. As EN1990 is to be used in conjunction with the material Eurocodes (e.g. EN1995-1-1), a number of the Principles in EN1990 do not require and do not have a supplementary clause in EN1995-1-1. In these instances a comment along the lines 'Material-independent Principle not requiring a supplementary clause in EN1995-1-1' is given in the third column of the table below.

Also in the third column of the table below, the term 'Grandfather clause' has been used for those clauses whose text is very fundamental but also so general as to in itself give no specific guidance to the designer.

Table 1: Section 2 – Requirements.

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
2.1	**Basic requirements**	
2.1(1)P	A structure shall be designed and executed in such a way that it will, during its intended life, with appropriate degrees of reliability and in an economic way - sustain all actions and influences likely to occur during execution and use, and - meet the specified serviceability requirements for a structure or a structural element.	Grandfather clause, which for timber structures, is addressed by the overall implementation of EN1995-1-1 The complimentary clauses in EN1995-1-1 are 2.1.1(1)P and 2.1.1(3).
2.1(2)P	A structure shall be designed to have adequate: - structural resistance, - serviceability, and - durability.	Grandfather clause which in EN1995-1-1 is addressed in: - sections 6, 8 & 9 (structural resistance) - section 7 (serviceability) - section 4 (durability)
2.1(3)P	In the case of fire, the structural resistance shall be adequate for the required period of time.	Grandfather clause, which for timber structures, is addressed by the overall implementation of EN1995-1-2
2.1(4)P	A structure shall be designed and executed in such a way that it will not be damaged by events such as: - explosion, - impact, and - the consequences of human errors, to an extent disproportionate to the original cause.	Grandfather clause which is primarily addressed by the implementation of EN1991-1-7.
2.1(5)P	Potential damage shall be avoided or limited by appropriate choice of one or more of the following: - avoiding, eliminating or reducing the hazards to which the structure can be subjected; - selecting structural form which has low sensitivity to the hazards considered; - selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localised damage; - avoiding as far as possible structural systems that can collapse without warning; - tying the structural members together.	This clause is primarily addressed by Annex A of EN1991-1-7 with the only input from EN1995-1-1 being the magnitude of the partial material factor for accidental design situations. This is less input than for some other material Eurocodes and **for example one area where EN1995-1-1 could provide guidance is the magnitude of reduced tie forces for lightweight (i.e. timber) structures.**

Table 1: Section 2 – Requirements (continued).

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
2.2	**Reliability management**	
2.2(1)P	The reliability required for structures within the scope of EN 1990 shall be achieved: - by design in accordance with EN 1990 to EN 1999 and - by appropriate execution, and quality management measures.	Grandfather clause part a) of which is met by the implementation of EN1995. **However an Execution Standard for timber structures is required to provide guidance to designers in respect of meeting part b). This is fully recognised by CEN/TC250/SC5.**
2.4	**Durability**	
2.4(1)P	The structure shall be designed such that deterioration over its design working life does not impair the performance of the structure below that intended, having due regard to its environment and the anticipated level of maintenance.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1, though of course section 4 gives rules for durability.
2.4(3)P	The environmental conditions shall be identified at the design stage so that their significance can be assessed in relation to durability and adequate provisions can be made for the protection of the materials used in the structure.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1.

Table 2: Section 3 – Principles of ultimate limit state design.

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
3.1	**General**	
3.1(1)P	A distinction shall be made between ultimate limit states and serviceability limit states.	EN1995-1-1 treats ultimate limit states and serviceability limit states separately in sections 6 and 7.
3.2	**Design situations**	
3.2(1)P	The relevant design situations shall be selected taking into account the circumstances under which the structure is required to fulfil its function.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1.
3.2(2)P	Design situations shall be classified as follows: - persistent design situations, which refer to conditions of normal use; - transient design situations, which refer to temporary conditions applicable to the structure, e.g. during execution; - accidental design situations, which refer to exceptional conditions applicable to the structure or its exposure e.g. to fire, explosion, impact or the consequences of localised failure; - seismic design situations, which refer to conditions applicable to the structure when subjected to seismic events.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1.
3.2(3)P	The selected design situations shall be sufficiently severe and varied so as to encompass all conditions that can be reasonably be foreseen to occur during the execution and use of the structure.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1.
3.3	**Ultimate limit states**	
3.3(1)P	The limit states that concern: - the safety of people, and/or - the safety of the structure shall be classified as ultimate limit states.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1.
3.3(4)P	The following ultimate limit states shall be verified where they are relevant: - the loss of equilibrium of the structure or any part of it, considered as a rigid body; - failure by excessive deformation, transformation of the structure or any part of it into a mechanism, rupture or loss of stability of the structure or any part of it, including supports and foundations; - failure caused by fatigue or other time-dependent effects.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1. **Currently the only guidance on fatigue effects is in EN1995-2. The need for EN1995-1-1 to give guidance on fatigue should be reviewed, though the current status quo may well be appropriate.**

Table 2: Section 3 – Principles of ultimate limit state design (continued).

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
3.4	**Serviceability limit states**	
3.4(1)P	The limit states that concern: - the functioning of the structure or structural members under normal use; - the comfort of people; - the appearance of the construction works, shall be classified as serviceability limit states.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1.
3.4(2)P	A distinction shall be made between reversible and irreversible serviceability limit states.	Whilst this is a sound Principle, **it is questionable whether EN1995-1-1 has implemented it (e.g. no distinction between reversible and irreversible serviceability limit states in deflection limits of Table 7.2 of EN1995-1-1).**
3.5	**Limit state design**	
3.5(1)P	Design for limit states shall be based on the use of structural and load models for relevant limit states.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1.
3.5(2)P	It shall be verified that no limit state is exceeded when relevant design values for - actions - material properties, or - product properties, and - geometric data are used in these models.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1.
3.5(3)P	The verifications shall be carried out for all relevant design situations and load cases.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1.
3.5(6)P	The selected design situations shall be considered and critical load cases identified.	Material-independent Principle, though for timber structures the identification of critical load cases is a more onerous task than for other materials on account of the load-duration characteristics of wood-based materials. This aspect is alluded to, if not fully spelt out, in clause 3.1.3(2) of EN1995-1-1.
3.5(8)P	Possible deviations from the assumed directions or positions of actions shall be taken into account.	**Unlike some other material Eurocodes no guidance is given in EN1995-1-1 for the evaluation of equivalent horizontal forces.**

Table 3: Section 4 – Basic variables.

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
4.1	**Actions and environmental influences**	
4.1.1	**Classification of actions**	
4.1.1(1)P	Actions shall be classified by their variation in time as follows: - permanent actions (G), e.g. self-weight of structures, fixed equipment and road surfacing, and indirect actions caused by shrinkage and uneven settlements; - variable actions (Q), e.g. imposed loads on building floors, beams and roofs, wind actions or snow loads; - accidental actions (A), e.g. explosions or impact from vehicles.	Within scope of EN1991 not EN1995
4.1.1(4)P	Actions shall also be classified - by their origin, as direct or indirect, - by their spatial variation, as fixed or free, or - by their nature and/or the structural response, as static or dynamic.	Within scope of EN1991 not EN1995
4.1.2	**Characteristic values of actions**	
4.1.2(1)P	The characteristic value F_k of an action is its main representative value and shall be specified: - as a mean value, an upper or lower value, or a nominal value (which does not refer to a known statistical distribution) (see EN 1991); - in the project documentation, provided that consistency is achieved with methods given in EN 1991.	Within scope of EN1991 not EN1995
4.1.2(2)P	The characteristic value of a permanent action shall be assessed as follows: - if the variability of G can be considered small, one single value G_k may be used; - if the variability of G cannot be considered small, two values shall be used: an upper value $G_{k,sup}$ and a lower value $G_{k,inf}$.	Within scope of EN1991 not EN1995
4.1.2(7)P	For variable actions, the characteristic value (Q_k) shall correspond to either: - an upper value with an intended probability of not being exceeded or a lower value with an intended probability of being achieved, during some specific reference period; - a nominal value, which may be specified in cases where a statistical distribution is not known	Within scope of EN1991 not EN1995

Table 3: Section 4 – Basic variables (continued).

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
4.1.3	**Other representative values of variable actions**	
4.1.3(1)P	Other representative values of a variable action shall be as follows: - (a) the combination value, represented as a product $\psi_0 Q_k$, used for the verification of ultimate limit states and irreversible serviceability limit states (see section 6 and Annex C); - (b) the frequent value, represented as a product $\psi_1 Q_k$, used for the verification of ultimate limit states involving accidental actions and for verifications of reversible serviceability limit states.	Within scope of EN1991 not EN1995
4.1.6	**Geotechnical actions**	
4.1.6(1)P	Geotechnical actions shall be assessed in accordance with EN 1997-1.	Within scope of EN1991 not EN1995
4.1.76	**Environmental influences**	
4.1.7(1)P	The environmental influences that could affect the durability of the structure shall be considered in the choice of structural materials, their specification, the structural concept and detailed design.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1
4.2	**Material and product properties**	
4.2(4)P	Material property values shall be determined from standardised tests performed under specified conditions. A conversion factor shall be applied where it is necessary to convert the test results into values which can be assumed to represent the behaviour of the material or product in the structure or the ground.	Determination of wood-based material property values is undertaken in EN Product Standards and not in EN1995-1-1. It is not clear whether the reference to conversion factors is intended to refer to the factors in EN384 for example or is a reference to the various modification factors in EN1995-1-1.

Table 3: Section 4 – Basic variables (continued).

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
4.2(10)P	Where a partial factor for materials or products is needed, a conservative value shall be used, unless suitable statistical information exists to assess the reliability of the value chosen.	It is perhaps doubtful that statistical information exists verifying that the partial material factors in Table 2.3 of EN1995-1-1 are conservative and indeed Kohler and Fink (2012) paper indicate that for some stress types the EN1995-1-1 partial material factors are non-conservative. As most National Annexes do not give exactly the same values as EN1995-1-1, ***this further indicates that the activities of COST FP1402-WG1-TG2 "Timber specific code calibration" are needed.***
4.3	**Geometrical data**	
4.3(1)P	Geometrical data shall be represented by their characteristic values, or (e.g. the case of imperfections) directly by their design values.	**Clause 2.4.2(1) of EN1995-1-1 is perhaps at odds, though sensibly so, with this Principle stating 'Geometrical data for cross-sections and systems may be taken as nominal values from product standards or drawings for the execution'.**
4.3(5)P	Tolerances for connected parts that are made from different materials shall be mutually compatible.	**This Principle can certainly be applicable to timber connections and the question is whether guidance on addressing it should be incorporated into EN1995-1-1 or left to technical manuals.**

Table 4: Section 5 – Structural analysis and design assigned by testing.

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
5.1	**Structural analysis**	
5.1.1	**Structural modelling**	
5.1.1(1)P	Calculations shall be carried out using appropriate structural models involving relevant variables.	The complimentary clauses in EN1995-1-1 are 2.2.1(1)P and 5.1(1)P.
5.1.1(3)P	Structural models shall be based on established engineering theory and practice. If necessary, they shall be verified experimentally.	The primary complimentary clause in EN1995-1-1 is again 5.1(1)P.
5.1.2	**Static actions**	
5.1.2(1)P	The modelling for static actions shall be based on an appropriate choice of the force-deformation relationships of the members and their connections and between members and the ground.	Clauses in EN1995-1-1 applying this Principle include: 2.2.2(1)P, 2.3.2.2(2), 5.1(2), 5.1(5) and 5.3(2)P ***COST FP1402-WG1-working activity 1 is looking at the "impact of stiffness on the results in ultimate limit state design".***
5.1.2(2)P	Boundary conditions applied to the model shall represent those intended in the structure.	Clauses in EN1995-1-1 applying this Principle include: 5.4.1(1)P and 5.4.2(1)P
5.1.2(3)P	Effects of displacements and deformations shall be taken into account in the context of ultimate limit state verifications if they result in a significant increase of the effect of actions.	Clauses in EN1995-1-1 applying this Principle include: 5.1(4)P, 5.4.4(1)P and 5.4.4(2)
5.1.2(4)P	Indirect actions shall be introduced in the analysis as follows: - in linear elastic analysis, directly or as equivalent forces (using appropriate modular ratios where relevant); - in non-linear analysis, directly as imposed deformations.	
5.1.3	**Dynamic actions**	
5.1.3(1)P	The structural model to be used for determining the action effects shall be established taking account of all relevant structural members, their masses, strengths, stiffnesses and damping characteristics, and all relevant non-structural members with their properties.	The only reference to 'dynamic actions' in the whole of EN1995-1-1 occurs in clause 10.6 'transportation and erection'.
5.1.3(2)P	The boundary conditions applied to the model shall be representative of those intended in the structure.	The only reference to 'dynamic actions' in the whole of EN1995-1-1 occurs in clause 10.6 'transportation and erection'.

Table 4: Section 5 – Structural analysis and design assigned by testing (continued).

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
5.1.4	**Fire design**	
5.1.4(1)P	The structural fire design analysis shall be based on design fire scenarios (see EN 1991-1-2), and shall consider models for the temperature evolution within the structure as well as models for the mechanical behaviour of the structure at elevated temperature.	This Principle is being addressed by Annexes A and B of EN1995-1-2, which are currently under development under the auspices of CEN/TC250/SC5.
5.2	**Design assisted by testing**	
5.2(1)P	Design assisted by test results shall achieve the level of reliability required for the relevant design situation. The statistical uncertainty due to a limited number of test results shall be taken into account.	There are a number of important timber components (e.g. wall diaphragms) where the amount of test data is always likely to be limited, ***affirming the need for the activity of COST FP1402-WG1-TG3 "Experimental & numerical data analysis".***

Table 5: Section 6 – Verification by the partial factor method.

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
6.1	**General**	
6.1(1)P	When using the partial factor method, it shall be verified that, in all relevant design situations, no relevant limit state is exceeded when design values for actions or effects of actions and resistances are used in the design models.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1.
6.1(5)P	Design values directly determined on statistical bases shall correspond to at least the same degree of reliability for the various limit states as implied by the partial factors given in this standard.	Material-independent Principle not requiring a supplementary clause in EN1995-1-1.
6.3	**Design values**	
6.3.2	**Design values of the effects of actions**	
6.3.2(3)P	Where a distinction has to be made between favourable and unfavourable effects of permanent actions, two different partial factors shall be used ($\gamma_{G,inf}$ and $\gamma_{G,sup}$).	Within scope of EN1991 not EN1995
6.3.4	**Design values of geometric data**	
6.3.4(2)P	Where the effects of deviations in geometrical data (e.g. inaccuracy in the load application or location of supports) are significant for the reliability of the structure (e.g. by second order effects) the design values of geometrical data shall be defined by: $a_d = a_{nom} + \Delta a \quad (6.5)$ Δa takes account of: - the possibility of unfavourable deviations from the characteristic or nominal values; - the cumulative effect of a simultaneous occurrence of several geometric deviations.	The design of most ordinary timber structures, analysed using a linear material model, would not be effected by this Principle (refer also to EN1995-1-1, 2.4.2(1).

Table 5: Section 6 – Verification by the partial factor method (continued).

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
6.4	**Ultimate limit states**	
6.4.1	**General**	

6.4.1(1)P	The following ultimate limit states shall be verified as relevant:
	- (a) EQU: Loss of equilibrium of the structure or any part of it considered as a rigid body, where:
	  - Minor variations in the value or spatial distribution of actions from a single source are significant, and
	  - The strengths of construction materials or ground are generally not governing;
	- (b) STR: Internal failure or excessive deformation of the structure or structural member, including footings, piles, basement walls, etc, where the strength of construction materials governs;
	- (c) GEO: Failure or excessive deformation of the ground where the strengths of soil or rock are significant in providing resistance;
	- (d) FAT: Fatigue failure of the structure or structural members.
		Material-independent Principle not requiring a supplementary clause in EN1995-1-1.

6.4.1(2)P	The design values of actions shall be in accordance with Annex A.	Within scope of EN1991 not EN1995. Also there appears an inherent contradiction between this clause being assigned a Principle and the National Choice allowed in Annex A.
6.4.2	**Verifications of static equilibrium and resistance**	
6.4.2(1)P	When considering a limit state of static equilibrium of the structure (EQU), it shall be verified that: $E_{d,dst} \leq E_{d,stb}$ (6.7) where: - $E_{d,dst}$ is the design value of the effect of destabilising actions; - $E_{d,stb}$ is the design value of the effect of stabilising actions.	Clause in EN1995-1-1 applying this Principle is 2.4.4(1).

Table 5: Section 6 – Verification by the partial factor method (continued).

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
6.4.2(3)P	When considering a limit state of rupture or excessive deformation of a section, member or connection (STR and/or GEO), it shall be verified that: $E_d \leq R_d$ (6.8) where: - E_d is the design value of the effect of actions such as internal force, moment or a vector representing several internal forces or moments; - R_d is the design value of the corresponding resistance.	The equation embodied in this Principle is applied numerous times in sections 6 and more sparingly in sections 8 and 9.
6.4.3	**Combination of actions (fatigue excluded)**	
6.4.3.1(1)P	For each critical load case, the design values of the effects of actions (Ed) shall be determined by combining the values of actions that are considered to occur simultaneously.	Within scope of EN1991 not EN1995
6.4.3.1(4)P	Where the results of a verification are very sensitive to variations of the magnitude of a permanent action from place to place in the structure, the unfavourable and the favourable parts of this action shall be considered as individual actions.	Within scope of EN1991 not EN1995
6.5	**Serviceability limit states**	
6.5.1	**Verifications**	
6.5.1(1)P	It shall be verified that: $E_d \leq C_d$ (6.9) where: - E_d is the limiting design value of the relevant serviceability criterion; - C_d is the design value of the effect of actions specified in the serviceability criterion, determined on the basis of the relevant combination.	The equation embodied in this Principle is applied in section 7.3 'Vibrations'.
6.5.3	**Combination of actions**	
6.5.3(4)P	Effects of actions due to imposed deformations shall be considered where relevant.	

Table 6: Annex A1 – Applications for buildings.

EN 1990 clause	Text of Principle	Clauses in EN 1995-1-1 addressing the Principles of EN 1990
A 1.4	**Serviceability limit states**	
A 1.4.2	**Serviceability criteria**	
A 1.4.2(3)P	The serviceability criteria for deformations and vibrations shall be defined: - Depending on the intended use - In relation to the serviceability requirements in accordance with 3.4; - Independently of the materials used for supporting structural member.	**It is doubtful that the serviceability criteria in the various Eurocodes (or their National Annexes) have been determined independently of the materials used and it may not be in the interests of industries to too actively pursue this Principle.**

Table 7: Summary of EN1990 Principles possibly not fully followed through in EN1995-1-1.

Topic	EN1990 clause	Commentary
Design for accidental damage	2.1(5)P	There is less input on accidental damage design in EN1995-1-1 than for some other material Eurocodes and for example one area where EN1995-1-1 could provide guidance is the magnitude of reduced tie forces for lightweight (i.e. timber) structures.
Reliability management achieved by appropriate execution/quality management	2.2(1)P b)	As with the other materials, an Execution Standard is needed for timber structures to fulfil this Principle. This is in hand within CEN/TC250/SC5.
Fatigue	3.3(4)P	It should be re-affirmed that it remains appropriate that no guidance (in addition to that in EN1995-2) on fatigue is given in EN1995-1-1.
Serviceability criteria. Distinction between reversible and irreversible states.	3.4(2)P	Whilst this is a sound Principle, it is questionable whether EN1995-1-1 has implemented it (e.g. no distinction between reversible and irreversible serviceability limit states in deflection limits of Table 7.2 of EN1995-1-1). However as serviceability criteria are set in National Annexes, each country has the option of implementing the Principle (e.g. setting lower deflection limits for flexural members supporting plastered or plasterboard linings).
Serviceability criteria. Independent of materials used.	A.1.4.2(3)P	It is doubtful that the serviceability criteria in the various Eurocodes (or their National Annexes) have been determined independently of the materials used though it may not be in the interests of industries to too actively pursue this Principle.
Equivalent horizontal forces	3.5(8)P	As in the past timber structures have mostly involved lightweight construction there has been less need to determine equivalent horizontal forces than for other construction materials and consequently timber design codes (including the current version of EN1995-1-1) do not contain any associated guidance. However with the increasing use of heavier constructions (e.g. timber-concrete composites) and taller timber buildings, there is an increasing need for guidance within EN1995-1-1 on determining equivalent horizontal forces to account for sway imperfections.
Geometrical data. Values to be used in design.	4.3(1)P	EN1995-1-1 (use of nominal dimensions) is at odds, though probably sensibly so, with EN1990 (use of characteristic values for dimensions).
Geometrical data. Tolerances for connected parts made from different materials.	4.3(5)P	This Principle can certainly be applicable to timber connections (particularly if considered in conjunction with moisture-related movements) and the question is whether guidance on addressing it should be incorporated into EN1995-1-1 or left to technical manuals.

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Eurocode Load Combination Rules and Simplified Safety Formats

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1 State of the Art – Eurocode Load Combination Rules

The safety assessment of structural elements according to Eurocode 0 (EN 1990, 2002) is based on a comparison of the resistance design value r_d with the design value of the effect of actions e_d where the former has to be larger than the latter in order to provide appropriate safety $r_\mathrm{d} > e_\mathrm{d}$. Following the requirements of Eurocode 5 (EN 1995-1-1, 2004) for structural timber design, the resistance design value r_d has to be determined for ultimate limit state design as presented in Eq. (1) with the characteristic value of the resistance r_k, the partial safety factor γ_M and the timber specific modification factor k_mod.

$$r_\mathrm{d} = k_\mathrm{mod} \frac{r_\mathrm{k}}{\gamma_\mathrm{M}} \tag{1}$$

The modification factor k_mod takes the load duration effect and moisture content of timber into account and is influencing the determination of the design load e_d or rather the decisive load combination, which will be described herein later. The design load e_d for persistent or transient design situations according to Eurocode 0 equation 6.10 can be calculated with

$$e_\mathrm{d} = \sum_{j \geq 1} 1.35 \cdot g_{\mathrm{k},j} + 1.5 \cdot q_{\mathrm{k},1} + \sum_{i > 1} 1.5 \cdot \psi_{0,i} \cdot q_{\mathrm{k},i} \tag{2}$$

(or two alternative formulas: equation 6.10a and 6.10b in Eurocode 0 for STR and GEO limit states) where ψ_0 is the load combination factor. For each relevant load case the design effect of action shall be determined by combining the loads that can occur simultaneously.

Due to the linear resistance models of the material property, the design check can be rewritten as in Eq. (3), where the resistance side is independent of k_mod.

$$r_\mathrm{d} > e_\mathrm{d} \rightarrow \frac{r_\mathrm{k}}{\gamma_\mathrm{M}} > \frac{e_\mathrm{d}}{k_\mathrm{mod}} \tag{3}$$

As it can be seen, the load case with the highest ratio of e_d/k_{mod} is decisive for design. The value for k_{mod} has to be chosen as the one corresponding to the load with the shortest duration considered in the combination. This circumstance requires the examination of a larger number of load combinations compared to other construction materials where the combination giving the largest design load is automatically decisive. Thus, the engineering effort is significantly higher, especially when hand calculations are performed, which is often the case for simple structures. For this reason, the following simplified safety formats where discussed and examined within Working Group 1 (basis of design) and a Short-term Scientific Mission in COST Action FP1402.

2 Simplified Safety Format I (SFI)

This simplified safety format is based on these two load combinations from Colling and Mikoschek (2016).

$$e_{d,1} = \gamma_F \cdot \left(\sum_{j\geq 1} g_{k,j} + \sum_{i\geq 1} q_{k,i} \right) \tag{4}$$

$$e_{d,2} = \sum_{j\geq 1} 1.35 \cdot g_{k,j} + 1.5 \cdot q_{k,1} \tag{5}$$

where γ_F is a global partial safety factor and $q_{k,1}$ is the leading live load. As usual, the design loads $e_{d,1}, e_{d,2}$ have to be divided with the corresponding values of k_{mod} (with the shortest load duration considered in the combination) in order to determine the decisive design load:

$$\frac{e_{d,1}}{k_{mod}} > \frac{e_{d,2}}{k_{mod}} \rightarrow e_{d,1} \text{ is the decisive design load,}$$
$$\frac{e_{d,2}}{k_{mod}} > \frac{e_{d,1}}{k_{mod}} \rightarrow e_{d,2} \text{ is the decisive design load.}$$

Generally, this simplified safety format is not thought to replace the current load combination rules according to Eurocode 0. Instead, it could provide an alternative for a quicker and more economic design of simple structures. However, the application should be only permitted for load cases with less than 60% permanent loads and only with imposed loads from categories A, B, C and D (not E with load duration = long).

3 Simplified Safety Format II

The additional effort for finding the decisive load combination in structural timber design is caused by the large number of different modification factors k_{mod}. Therefore, it was proposed in Baravalle et al. (2017) to use a fixed value of k_{mod} (hereinafter referred to as k'_{mod}) for load cases with dominating permanent loads and for

Table 1: Calibrated values of global factor γ_F in SFI and fixed k'_{mod} in SFII.

Case		permanent loads dominating	variable loads dominating
1	γ_{F}	2.14	1.46
	k'_{mod}	0.63	0.89
2	γ_{F}	2.17	1.48
	k'_{mod}	0.62	0.84
3	γ_{F}	2.35	1.56
	k'_{mod}	0.63	0.92
4	γ_{F}	2.38	1.58
	k'_{mod}	0.62	0.86

load cases with dominating variable loads. This simplification is leading to a number of load combinations which is equal to the ones considered for any other construction material.

4 Calibration of Reliability Elements

Simplifications have to provide a satisfying level of safety. For this reason, both simplified safety formats needed to be calibrated by established techniques. At first, the reliability indices β associated with the simplified safety formats were calculated and compared with the safety level given by the Eurocodes. Then, the reliability elements (global safety factor γ_{F} in SFI and k'_{mod} in SFII) were calibrated in order to satisfy the objective of minimizing the reduction of structural efficiency without compromising the structural safety level.

The calibration was restricted to service class 1 and 2, only three load types (self-weight, snow, wind), two materials (solid timber and glulam) and three failure modes (bending, tension and compression parallel to the grain). For the comparison of climatic cases, four types of climate were regarded by combining snow and wind actions with different characteristics. These cases might represent the climates and load durations of Germany, Austria, Denmark and Norway.

Different load scenarios were included in the study, too. They are characterized by the proportions between the different loads expressed as $\chi_G = g_k/(g_k + q_{1,k} + q_{2,k})$ and $\chi_Q = q_{1,k}/(q_{1,k} + q_{2,k})$. The load scenarios are divided into two domains: dominating permanent loads with $\chi_G \geq 0.6$ and $0 \leq \chi_Q \leq 1$; dominating variable loads with $0 \leq \chi_G \leq 0.6$ and $0 \leq \chi_Q \leq 1$.

The results of the calibration process published in Baravalle et al. (2017) are summarized in Table 1.

5 Conclusion

The load duration factor k_{mod} can cause a large number of load combinations in the design of timber structures. Therefore, two simplified safety formats have been proposed and calibrated. Both proposed safety formats with the calibrated reliability elements meet the requirement of simplifying design without decreasing the level of safety. The question, of which type of simplification is to be preferred, has to be discussed in further investigations or expert groups. More profound and detailed calculations and results are described in Colling and Mikoschek (2016); Baravalle et al. (2017).

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Timber specific code calibration[1]

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Abstract

The European timber design standard is under development and a new version will be issued at the end of this decade. In this chapter the present design standard is critically assessed in regard to its ability to identify design solutions with a consistent level of reliability. The main issues to enhance the current standards are identified and discussed. Thereunder, the influence of different material properties in different load directions, the quality of the grading process, the application of the current safety concept on non-linear design equations, duration of load effects, effects due to moisture induced stresses, volume and length effects, reliable and uniform design equations for joints and the adoption of consequence classes for associated situations are considered.

1 Introduction

Sustainable development is the important requirement and goal for modern society and the international research community is in demand to find solutions that provide the foundation for this aim. The role of structural engineering research is thereby of significant importance. The development of methodologies and principles that allows for the optimal allocation of resources into the structural performance and their implementation into the daily engineering practice constitute the major challenge for ongoing and future research in the field of structural engineering.

The broad implementation of newly developed principles requires their proper transition into rules and regulations that constitute the basis for the daily work of practicing engineers. Thus, rules and regulations as structural design codes constitute the mayor interface between structural engineering research and practical application and it is of utmost importance that structural design codes are up to date with the best

[1]The work is based on the article *Aspects of code based design of timber structures*, published at the 12th International Conference on Applications of Statistics and Probability in Civil Engineering (Kohler and Fink, 2015).

scientific information available and, at the same time, are simple enough for straight forward application.

This challenge outlined above is general for the entire structural engineering research and professional community. Here, timber and timber based materials might be attributed with a special status since timber as a natural grown material plays an important role in the safe, cost efficient and sustainable development of our future build environment because of its beneficial properties. Timber is an efficient building material, not least in regard to its mechanical properties but also because it is a highly sustainable material considering all phases of the life cycle of timber structures: production, use and decommissioning.

Timber is a widely available natural resource; e.g. with proper management, there is a potential for a continuous and sustainable supply of raw timber material in the future. Because of the low energy use and the low level of pollution associated with the manufacturing of timber structures the environmental impact is much smaller than for structures built in other materials.

However, besides the beneficial properties of timber the confident use of timber as a load-bearing material is particularly challenging compared to other common structural materials as steel and concrete. One of the main reasons for this is that timber is a highly complex material; the proper use in structures actually requires a significant amount of expertise in structural detailing.

Another main reason is that any prediction of the structural performance of timber is associated with large uncertainties. Timber is by nature a very inhomogeneous material. The material properties depend on the specific wood species, the geographical location and furthermore on the local growing conditions over the entire lifetime of the tree. Timber is an orthotropic material, i.e. it consists of "high strength" fibres/grains which are predominantly oriented along the longitudinal axis of a timber log/tree and packed together within a "low strength" matrix. After a log is sawn into pieces of structural timber, irregularities, such as grain direction or knots, become, in addition to the orthotropic characteristics mentioned above, highly decisive for the load-bearing capacity of a timber structural element. Consequently, the properties of solid timber cannot be designed or produced by means of some recipe but may be ensured to fulfil given requirements only by quality control procedures implemented during the production process for sawn timber. Timber material for structural purpose is generally associated to a certain grade or strength class. However, there are various different ways how quality control is implemented in the production process and the properties of timber of a certain strength class are highly sensitive to the quality control scheme applied to the timber.

Timber is a viscoelastic and hygroscopic material. When using timber as a load-bearing element in a structure it is of high interest how the load-bearing performance is developing over time, i.e. how the building environment with its variable loads, temperature and moisture is influencing the timber structural element.

The high importance of structural timber and timber products for the sustainable development of our build infrastructure together with the fact that many features of the structural behaviour of timber are not known with accurate precision underlines the urgent need for extensive and coordinated research in this field. Furthermore it is necessary that current and future knowledge about timber and timber based materials load- bearing behaviour is represented in the current design standards in a sensible way.

In Europe the design of structures is regulated by the Eurocodes, a suite of consistent standards for structural design covering all relevant load scenarios and building materials. They were developed under the supervision of the European Committee of Standardization (CEN) and regulate to a large extent the performance criteria of the build environment being reliability, serviceability and safety of structures. The Eurocodes had been introduced in the 1980s and are by now compulsory for structural engineering design in most European countries. Until 2020 a revision and update of the Eurocodes is planned. Thus, this constitutes an excellent opportunity to critically reflect the design procedures prescribed in the Eurocode 5 – "Design of Timber Structures" in the light of recent scientific developments.

2 Basic principles of reliability based code calibration

2.1 General

Modern design codes, such as the Eurocodes (2002), are based on the so-called load and resistance factor design (LRFD) format. Next, the principle of LRFD is explained for the case of two loads; one that is constant and one that is variable over time. The LRFD equation is given in Eq. (1). Here R_k, G_k and Q_k are the characteristic values of the resistance R, the permanent load G, and the time variable load Q. γ_m, γ_G and γ_Q are the corresponding partial safety factors. z is the so-called design variable, which is defined by the chosen dimensions of the structural component.

$$z\frac{R_\mathrm{k}}{\gamma_m} - \gamma_G G_\mathrm{k} - \gamma_Q Q_\mathrm{k} = 0 \qquad (1)$$

The characteristic values for both load and resistance are in general defined as fractile values of the corresponding probability distributions. In Eurocode 5 (2004) the following characteristic values are defined: R_k is the 5% fractile value of a Lognormal distributed resistance, G_k is the 50% fractile value (mean value) of the Normal distributed load (constant in time), and Q_k is the 98% fractile value of the Gumbel distributed yearly maxima of the load (variable in time).

The corresponding partial safety factors can be calibrated to provide design solutions (z) with an acceptable failure probability P_f (Eq. 2). Here R, G, and Q are resistance and loads represented as random variables, $z^* = z(\gamma_m, \gamma_G, \gamma_Q)$ is the design solution identified with Eq. (1) as a function of the selected partial safety factors, and

X is the model uncertainty.

$$\begin{aligned} &P_f = P\{g(X,R,G,Q) < 0\} \\ &\text{with} \quad g(X,R,G,Q) = z^* XR - G - Q = 0 \end{aligned} \tag{2}$$

Often the structural reliability is expressed with the so-called *reliability index* β (Eq. 3). A common value for the target reliability index is $\beta \approx 4.2$ which corresponds to a probability of failure $P_f \approx 10^{-5}$ (JCSS, 2001).

$$\beta = -\Phi^{-1}(P_f) \tag{3}$$

In general, different design situations are relevant; i.e. different ratios between G and Q. This can be considered using a modification of Eq. (1)–(2) into Eq. (4)–(5). α_i might take values between 0 and 1, representing different ratios of G and Q. $\hat{R}$, $\hat{G}$, and $\hat{Q}$ are normalized to a mean value of 1. For each α_i one design equations exists, thus altogether n different design equations have to be considered.

$$z_i \frac{\hat{R}_\mathrm{k}}{\gamma_m} - \gamma_G \alpha_i \hat{G}_\mathrm{k} - \gamma_Q (1-\alpha_i) \hat{Q}_\mathrm{k} = 0 \tag{4}$$

$$g_i(X,\hat{R},\hat{G},\hat{Q}) = z_i^* X\hat{R} - \alpha_i \hat{G} - (1-\alpha_i)\hat{Q} = 0 \tag{5}$$

Afterwards, the partial safety factors (γ_m, γ_G, and γ_Q) can be calibrated by solving the optimisation problem given in Eq. (6).

$$\min_{\gamma} \left[\sum_{j=1}^{n} \left(\beta_\text{target} - \beta_j \right)^2 \right] \tag{6}$$

The reliability based code calibration is briefly introduced to illustrate the influence of uncertainties (load and resistance), in respect to codes. Please find more information in (e.g. JCSS, 2001; Faber and Sørensen, 2003).

The application of the above sketched framework constitutes the basis for reliability based calibration of the partial safety factors of a load and resistance factor design format. And it entirely depends on a realistic representation of loads, resistances and model accuracy by the random variables R, Q, G, and X.

2.2 Example

The design equation for a structural component can be calibrated according to the procedure described in above. The chosen variables of Eq. (4) and Eq. (5) are summarized in Table 1. Using this values the situation could represent a solid timber bending beam loaded by constant (e.g. self-weight of beam and installations) and variable (e.g. live load).

Table 1: Chosen representation of the model uncertainty X, the bending strength R, the permanent load G and the variable load Q.

	X	R	G	Q
Mean value	1	1	1	1
Standard deviation	0.1	0.25	0.1	0.4
Distribution type	Lognormal	Lognormal	Normal	Gumbel
Fractile	-	0.05	0.5	0.98
Characteristic value	-	0.647	1	2.037

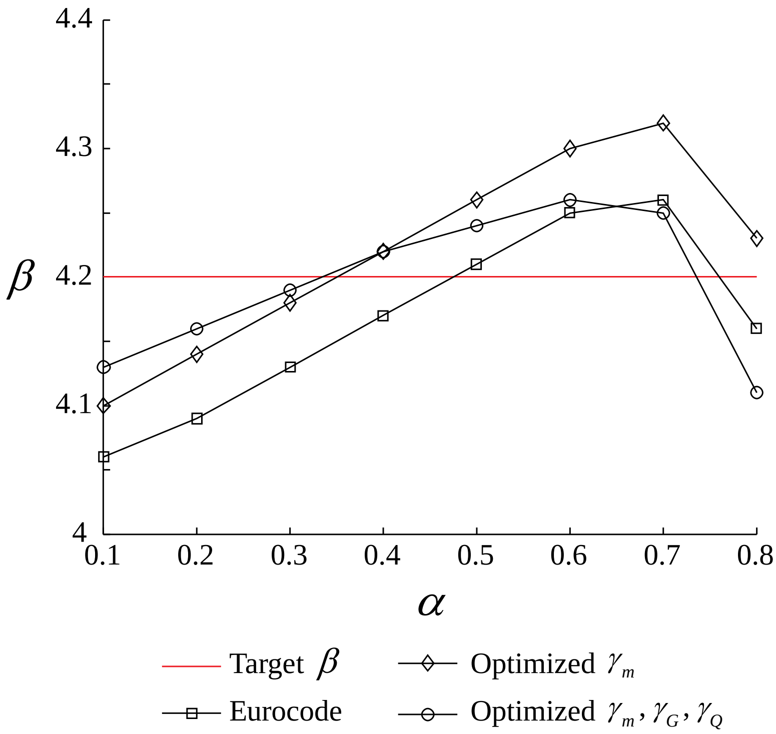

Figure 1: Reliability Index over different design situations alpha for solid timber in bending. The different lines represent different sets of partial safety factors (Kohler and Fink, 2012).

In the presented example, which is explained in more detail in Kohler and Fink (2012), the range $\alpha = [0.1, 0.2, ..., 0.8]$ is chosen, to exclude rather unrealistic design situations. The calculations were performed with the software CodeCal (JCSS, 2001). In Figure 1 the chosen target reliability index of $\beta = 4.2$ (red line) is compared with the design solutions for the structural component obtained according to the current version of the Eurocode ($\gamma_m = 1.3, \gamma_G = 1.5, \gamma_Q = 1.5$); represented by the line with squares. The reliability indices of the design solutions according to the Eurocode tend to be too low compared to the target reliability index, especially for small α. The line with the diamonds is obtained when all partial safety factors are subject to optimization: $\gamma_m = 1.29, \gamma_G = 1.30, \gamma_Q = 1.57$. However, it is the philosophy of the Eurocodes that the partial safety factors for the loads are material independent. Thus, γ_G and γ_Q are fixed and the optimization is performed only subject to γ_m. The line with the circles in Figure 1 is representing the corresponding result ($\gamma_m = 1.33$).

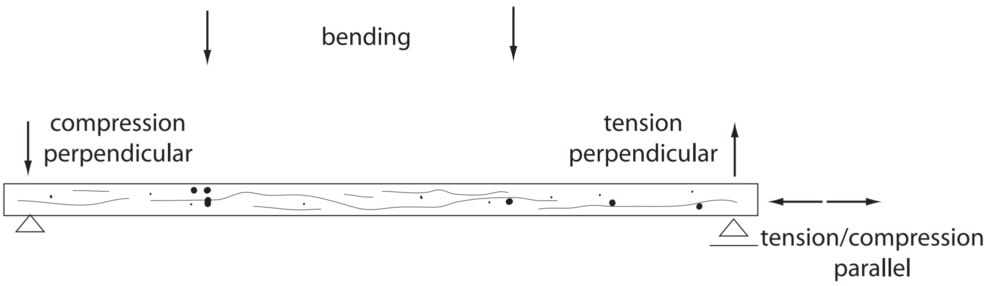

Figure 2: Different "material properties" dependent on the loading mode.

The above example demonstrates the validity of the Eurocode 5 (2004) design safety concept for timber load-bearing elements under the assumption that the parameters given in Figure 1 represent the real situation with sufficient accuracy. In the following it will be discussed in which ways the actual load load-bearing behavior deviates from the assumptions in Table 1. It is demonstrated and quantified how the corresponding deviation affects the reliability of design situations and it is discussed how recent research results might integrated in the further developed issue of Eurocode 5 (2004).

3 Particularities in timber material modelling

In the following section the main issues to enhance the current standards are identified and discussed. That includes the influence of different material properties in different load directions, the quality of the grading process, the application of the current safety concept on non-linear design equations, duration of load effects, effects due to moisture induced stresses, volume and length effects, reliable and uniform design equations for joints and the adoption of consequence classes for associated situations.

3.1 Different "material properties"

Timber is a rather complex building material. Its properties are highly variable, spatially and in time. In structural engineering, material properties of timber are in general understood as the stress and stiffness related properties of standard test specimen under given (standard) loading and climate conditions and the timber density. Test configurations are prescribed in e.g. ISO 8375 (1985) and any statement about stress and stiffness related properties of structural timber is conditional to the corresponding test configuration. In general it is distinguished between the different loading modes and "material properties" are given corresponding to the loading direction relative to the main fiber direction of a beam shaped element (Figure 2).

The "material properties" have different statistical properties and when using the design criterion introduced before and applying the same partial safety factor γ_m, as it is practiced in the Eurocode, the reliability of the corresponding design solutions differ.

Table 2: Calibrated partial safety factors for the resistance, for constant $\gamma_G = 1.35$ and $\gamma_Q = 1.50$ (from Kohler and Fink, 2012).

Ultimate limit state	γ_m
Bending strength	1.33
Tension strength parallel to the grain	1.40
Tension strength perp. to the grain	3.05
Compression strength parallel to the grain	1.24
Compression strength perp. to the grain	1.20
Shear strength	1.33

The influence of different "material properties" was investigated in Kohler and Fink (2012). There, the distribution functions and the associated variability for different types of "material properties" were chosen, as recommended in the Probabilistic Model Code JCSS (2006), see also Köhler (2006) for background information. The results are summarized in Table 2. The obtained scatter in partial safety factors suggests a rather differentiated treatment of the different design situations in future developments of design codes.

The most extreme deviation from the values proposed in the Eurocode $\gamma_m = 1.30$ is obtained for the load case tension perpendicular to the grain $\gamma_m = 3.05$. This also indicated that if a structural element for this load case is designed with the current safety factor of $\gamma_m = 1.30$, very low reliability indices, in the order of magnitude of 3.1 are obtained. However, the results concerning this particular load case have to be considered with special care. In fact the material capacity under this loading mode is specified by EN 338 (2010) with a nominal value that does not correspond to the 5%-fractile value taken from the statistical distribution that is derived from test data for the same loading mode. It is rather a value well below the 5%-fractile value. Furthermore, in best practice timber engineering design this loading mode at its limit is avoided due to the high sensibility to aspects that are not directly controlled in design, as e.g. moisture induced stresses and macro and micro cracks in the timber.

On the other hand the current design for compression strength perpendicular to the grain seems to be rather conservative $\gamma_m = 1.20$. Here, it has to be considered that the consequences that are resulting from failure of the compression strength perpendicular to the grain are rather low (see also Chapter 3.7). Thus, the conservative approach has to questioned.

3.2 Timber as a graded material

Due to the special way timber material properties are ensured by means of grading in the production line, special considerations must be made when modeling their probabilistic characteristics. Previous work on this subject is reported in (e.g. Rouger, 1996; Pöhlmann and Rackwitz, 1981). Further assessment of the probabilistic modelling on the properties of graded timber material was presented in Faber et al. (2004); Sandomeer et al. (2008). In the latter references it is reported that the scatter of strength related material properties is highly sensitive to the grading procedure applied and to the properties of the original ungraded material. This observation is confirmed by a large experimental campaign that took place recently in Europe in connection to the Gradewood project[2]. Here a large number of graded samples have been tested and a large between sample variability has been observed. Furthermore it has been shown that it is highly uncertain whether a sample that is graded to a specific grade actually meets the corresponding requirements in regard to minimum 5% fractile values of strength properties.

It is continued along the example introduced above, assuming that the grading accuracy directly affects the coefficient of variation of the timber bending capacity. The material partial safety factors are calibrated for different grading schemes that correspond to different coefficients of variation in the range from 0.2 – 0.4. The corresponding partial material safety factors rage between $\gamma_m = 1.2 - 1.65$ depending on the applied grading procedure. These results suggest a better differentiation of the grading procedure in future design codes. Alternatively, if no information about the accuracy of timber grading is utilized a larger coefficient of variation for representing the bending capacity should be used.

3.3 Non linear design equations

For common design equations a linear comparison of load effects and component resistance as in Eq. (1) is not sufficient. One example is the design of slender columns where strength and stiffness properties and creep effects play an important role for assessing the stability. For the analysis of single members, standards generally give simplified calculation models that do not require a 2$^{\text{nd}}$ order ultimate limit state analysis. However, for the analysis of more complex systems like unbraced frame structures, a 2$^{\text{nd}}$ order structural analysis is more appropriate and accurate and an alternative design procedure is given e.g. in the Eurocode (2004). Compared to the simple design format as presented in Eq. (1), the design equations for slender columns are more complex containing uncertain properties as strength, stiffness and load eccentricity in non-linear combination. The problem was addressed in Köhler et al. (2008) and quite uneven reliabilities for different column slenderness have been reported. In Figure 3 the reliability index of design solutions with different slenderness-ratio are presented. The different colors correspond to different design frameworks; I. EN 1995-1-1 (2004), 2$^{\text{nd}}$ order method with the stiffness considered as the mean modulus of elasticity;. II. DIN 1052 (2004), 2nd order method with the stiffness considered as the 5% fractile of the modulus of elasticity, and; III. EN 1995-1-1 (2004) / DIN 1052 (2004) according to the so called simplified equivalent length approach. For more details compare Köhler et al. (2008).

[2]http://www.woodwisdom.net/wp-content/uploads/2014/08/Gradewood_final_report.pdf

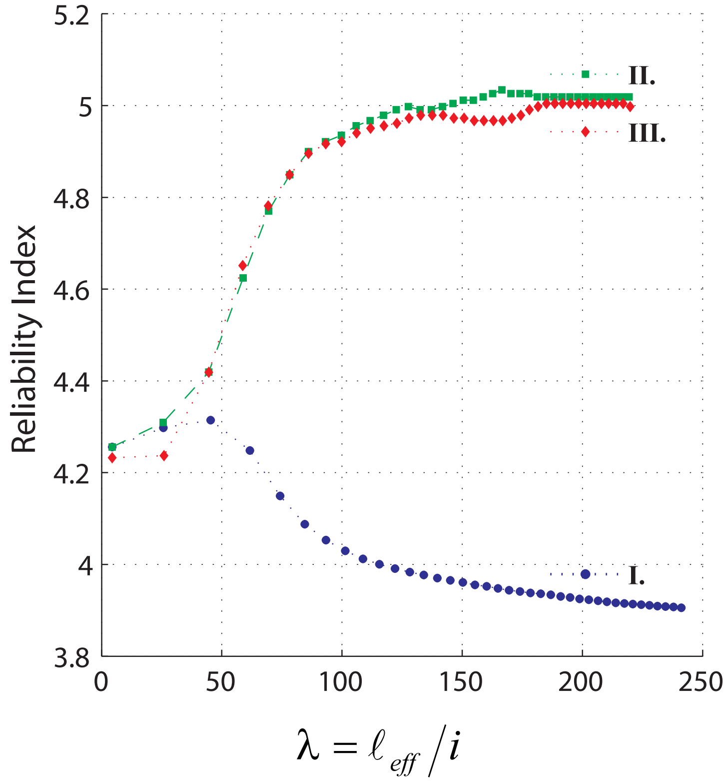

Figure 3: Reliability Index over slenderness for design solutions according to different design formats (modified from Köhler et al., 2008).

In future code safety formats design strength and stiffness should be clibrated in order to obtain consistent reliability levels for different design situations.

3.4 Duration of load effect and moisture induced stress

The capacity of a timber structural element is highly dependent on the time duration of the load effect to which it is exposed to. As an example the capacity of a bending beam continuously loaded is only 60% of that of a similar beam exposed to an instant load (Wood, 1947).

Timber is a hygroscopic material, i.e. it adsorbs and desorbs moisture from the surrounding air. Variations in moisture content in the surrounding air will, with a corresponding time lack, lead to variations in moisture content in the timber, this affects the mechanical properties of the timber but more importantly it will induce stresses due to shrinkage and swelling alongside the moisture gradients in the timber. These moisture induced stresses have been a matter of intensive discussion in the timber engineering community in the last years.

Both, duration of load effect and moisture induced stresses are highly relevant phenomena to take into account in structural design. They are also challenging phenomena since the underlying physical mechanisms are not fully understood and empirical evidence is scarce. However, in practical design, as in the Eurocode 5 (2004), the effect of moisture on the duration of load effect is considered with the joint modification factor k_{mod} which is given for different climate exposures in design codes.

Values for this factor are prescribed in a matrix for three different so-called service classes, i.e. different climate scenarios, and five different load classes, i.e. load scenarios.

This format appears to be oversimplified and further research and enhancement of the level of detail in structural design should be developed.

3.5 Volume and length effects

One major topic that is continuously discussed within the research community is the appropriate representation of size effects on strength in solid timber. For most loading modes as tension parallel or perpendicular to grain, shear or bending, timber predominately presents brittle failure behaviour. A (perfect) brittle material is defined as a material that fails if a single particle fails (see e.g. Bolotin, 1969). The strength of the material is thus governed by the strength of the 'weakest' particle; therefore the model for ideal brittle materials is also called the weakest link model (Weibull, 1939). This model was applied to the different failure modes in timber, the model parameters have been calibrated based on experimental evidence on the different failure modes. A literature review can be found e.g. in Kohler et al. (2013). There it is concluded that the size effects in timber are better represented with a model that takes into account the multi scale variability of structural timber and a corresponding model framework is suggested.

In present code formats size effects are often not completely taken into account or neglected. This is particularly critical when large scale engineered timber sections are used in modern timber construction. In a revision of the codes this aspect should earn appropriate attention and current research results should be implemented.

3.6 Joints

For timber structures, the structural performance depends to a considerable part on the connections or joints between different timber structural members; joints can govern the overall strength, serviceability and fire resistance. Despite their importance timber joint design frameworks are not based on a consistent basis compared to the design regulations of timber structural components.

Explanations for this difference in progress of design provisions for members and joints can be found in the relative simplicity of characterizing mechanical behaviour of members, as compared to connections. A diversity of joint types is used in practice and these types have infinite variety in arrangement. This usually precludes the option of testing large numbers of replicas for a reliable quantification and verification of statistical and mechanical models. The main and most important group of joints corresponds to the joints with dowel type fasteners, i.e. joints with dowels, nails, screws and staples belong to this group.

Different failure modes can be observed for dowel type fastener joints and the modes are partly captured by a simple mechanical model based on the works of Johansen (1949); Meyer (1957). These models build the basis for the current European design framework for dowel type connectors in the Eurocode. However, different failure modes correspond to different failure behavior and consequences (brittle or ductile). In Köhler (2006) is has also been observed that model uncertainty and model bias for the different failure modes is significantly different. This is not considered in the current version of the European design standard and should be subject for further investigation.

3.7 Consequence classes

In the previous chapter it was mentioned that different failure modes in dowel type fastener joints lead to different magnitudes of consequences. This is in principle true for all failure modes in timber structure. In Chapter 3.1 different failure modes of timber components have been compared to the same target reliability, implying that the consequences for all failure modes are classified uniformly. However, if a failure scenario for tension or bending failure is visualized and compared with a typical failure scenario for compression perpendicular to the grain, it might be agreed that the consequences are quite different and correspondingly the target reliability should be defined separately for the different cases.

4 Conclusion

Timber will play an important role in the future developments towards a more sustainable building sector. However, many stakeholders are still skeptical when it comes to the technological maturity of the material, especially compared to concrete and steel. The structural design regulations in general can be seen not only as the main interface connecting the state of knowledge in the engineering research community with the implementation of the real build environment; design standards are also the precondition for the implementation of building material on a high technological level.

In this chapter the major challenges for the future development of timber design standards have been highlighted from a European perspective; i.e. taking the Eurocodes as references. The challenges are hereby related to both, the further development of the knowledge basis for the behavior of timber in structures and the implementation of this knowledge into practicable rules in the future standards.

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Serviceability limit states in structural timber design

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Abstract

Consideration of serviceability issues is important in both, the design of new and the refurbishment of existing timber buildings, since they cannot be completely avoided. Moreover, buildings without serviceability problems might even be impractical, if one sees them as some kinds of indicator about the increased probability of structural failure. However, there is no direct relationship between safety and serviceability. Furthermore, it is not necessarily obvious how to deal with serviceability and what are the underlying principles behind the recommendations in structural design codes. A brief overview about relevant aspects concerning Eurocodes is given in this contribution.

1 Introduction

When designing timber (or any other) structures usually two types of limit states are considered: ultimate limit states (ULS) and serviceability limit states (SLS). There are several design situations, where the acceptable performance of structures is defined by requirements related to serviceability rather than safety. For example, in case of large span timber floor joists the structural dimensions are more likely to be determined by the stiffness and springiness of the structure rather than the strength of the structural elements and the load-bearing capacity of connections. To estimate the reliability in SLS, i.e. the probability that the limit state will not be exceeded, three main design aspects should be considered: 1) the relevant exposures (e.g. loads, temperature, relative humidity etc.); 2) factors affecting the structural response (e.g. boundary conditions, geometrical dimensions, material properties, level of structural modelling etc.); and 3) the performance criterion itself (Honfi, 2013).

There is, however, a fourth aspect, related to the material behaviour on a structural level and material specific design approaches and construction methods. For engineers being familiar with steel or concrete, the switch to special rules only related to timber structures might become a severe barrier, especially in the case of composites and building structures made of several building materials. In many cases, SLS also becomes decisive for the final dimension of structural members or connections and therefore has a significant impact on the competitiveness of building materials and building systems.

2 Why do we need serviceability limits?

2.1 General

Structural serviceability refers to the adequacy of a structure to fulfil its original design function or in other words serve as it is intended. The most fundamental requirement for structures is to be safe and fail in a predictable way. However, even sufficiently safe structures may not serve as intended; e.g. excessive vibrations (e.g. due to human activity, wind gusts) might cause annoyance to building occupants; large deflections of timber girders (e.g. due to creep) might hinder operation of machinery attached to them or cause occupants to feel unsafe. Common serviceability issues include (Galambos and Ellingwood, 1986):

- Damage to non-structural elements due to excessive deflections;
- Excessive distortion of connections under service loading
- Reduced functionality of furniture or equipment due to deflections;
- Discomfort of occupants due to noticeable (typically vertical) deflections;
- Discomfort or annoyance of occupants due to building motion, e.g. vibrations caused by normal use or wind;
- Deterioration of the structure due to age and use (if not impacting safety);
- Extensive damage to non-structural elements due to extreme natural events (if not impacting safety).

Serviceability is traditionally regarded less important than safety due to its consequences related to human life and injuries; however, the consequences of serviceability failure could be significant in terms of economic costs and the efforts required for dealing with the occupants' claims. The major differences between ULS and SLS are not only characterized by the distinct difference in perception of consequences of non-performance. Some other fundamental differences are listed below (Reid, 1981):

- Problems related to safety failure are usually clearly defined, whereas the definition of serviceability limit state is not always straightforward, since there might be a progressive transition between satisfactory and unsatisfactory behaviour.
- The failure boundary (i.e. the limit state) might not only be difficult to define, but is often subjective, i.e. dependent on the user/occupant.
- Safety problems are usually irreversible, whereas serviceability problems can often be reversible.

Figure 1: Jammed door due to excessive timber deflection and illustration of the deflection as deviation from a straight line (Courtesy of NCREP).

- The main design objective associated with safety is to ensure sufficiently small probabilities of failure. In contrast, the purpose of designing for serviceability is to achieve an economic structure i.e. to minimize the total costs over the design lifetime.
- Safety regulations are often mandatory and serve to protect both the designer and the client. In turn, serviceability criteria in the codes are usually only suggestions and are subject of modification by the designer if agreed with the client.

2.2 Issues typical for timber structures

Serviceability is often the governing issue when verifying the structural behaviour of existing timber structures or when designing new ones. The deformations in structural timber might increase significantly over time due to the effect of "normal" and mechano-sorptive creep, thus they need to be considered during design, especially if considerable sustained loading and moisture changes are expected. Deformations often cause problems at timber floors and roofs. The effect and extent of excessive deflections can be observed in Figure 1 and Figure 2.

For timber floors perhaps the most relevant serviceability issue is related to human-induced vibrations due to the relatively light weight and typically low bending stiffness (compared to steel and concrete constructions). Wind-induced deflections and vibrations of tall timber structures are getting greater attention with recent design and construction of tall timber buildings.

It is common that the structural assessment of existing timber buildings leads to the substitution or to the heavy strengthening of the floor or/and the roof due to serviceability requirements, according to the methodology of EC5 (EN 1995-1-1, 2004) (and to the limits of specific National Annexes). However, many of these

Figure 2: Excessive timber floor deflection and effect on the supported partition wall (Courtesy of NCREP).

structures have shown a satisfactory behaviour for their lifetime (sometimes for more than 100-200 years). Based on this, an analysis of the verification of the guidance and requirements related to serviceability is required, to understand if the conditions considered in the codes are suitable or if they lead to overly conservative approaches.

It is important to note that, supposedly, the serviceability limits are not rigid and can be discussed between the designer and the owner of the building. However, the designers should have some confidence in the discussion of these values, otherwise they will certainly have a conservative approach or, in a worst case, a very liberal approach which can lead to a dangerous situation (for instance resonance of timber floors).

3 Serviceability limit states in the Eurocodes

The Eurocode design family employs the limit state principle. Serviceability limit states are defined as states that correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met. According to EN 1990 (2002), the verification of serviceability limit states is based on criteria concerning the following aspects:

- Deformations;
- Vibrations; and
- Damage (that is likely to adversely affect appearance, durability or functioning of the structure).

Serviceability limits states are distinguished as irreversible and reversible limit states. When exceeding an irreversible limit state certain effects of the actions will remain after they are removed, whereas for reversible limit states no effects will remain after the actions' removal. The combinations of actions to be considered in the relevant design situations should be appropriate for the serviceability requirements and performance criteria being verified.

The combinations of actions for serviceability limit states, specified in EN 1990 (2002), are:

- Characteristic combination (normally used for irreversible limit states);
- Frequent combination (normally used for reversible limit states);
- Quasi-permanent combination (normally used for long-term effects and the appearance of the structure).

Serviceability limit states in buildings are supposed to take into account criteria related to, for example, floor stiffness, differential floor levels, storey sway or/and building sway and roof stiffness. Stiffness criteria may be expressed in terms of limits for vertical deflections and for vibrations. Sway criteria may be expressed in terms of limits for horizontal (absolute or relative) displacements. According to the code, the serviceability criteria should be specified for each project and agreed with the client. However, commonly, the client is unlikely to be able to contribute to this discussion and would prefer to rely on the professionals. When checking the performance of e.g. timber floors the influence of the constructive elements of the floors on the deflection and vibration could be significant. Timber floors are simple structures with a complex behaviour that depends on the performance of the whole system: the beams, the struts, the floorboard and even the ceilings. The way the load applied to the floor is distributed to the beams, and the contribution that the different components have to the floor's global performance are important issues. In fact, the load distribution factor conferred by struts and floorboard, designated k_{sys} in EC5 (EN 1995-1-1, 2004), which accounts for their stiffening effect, is essential to understand the real behaviour of the floors regarding their deflections and vibrations. This factor, however, is not taken into account in SLS. Another important issue regarding the deflections and vibration is the boundary conditions (support of timber beams in the walls, usually stone or brick masonry).

4 Rational basis for serviceability requirements

4.1 Background

Various studies and reports from the late 80ies acknowledged that a better understanding and improved guidelines are required concerning structural serviceability, see e.g. ASCE Ad Hoc Committee on Serviceability Research (1986); Cooney and King (1988).

General remarks of a symposium on the serviceability of buildings in Ottawa, 1988 can be summarized as follows (Ellingwood, 1988):

- Serviceability is closely linked to the expectations of the owner and building occupants about the structure's performance as a "consumer product";

- The expectations of individuals differ, which makes it difficult to define specific serviceability limits to be used in structural standards;
- To ensure serviceable performance, the overall system behaviour must be considered;
- Serviceability problems typically occur during a mixed use of building materials and products;
- The connection between performance requirements and the structural response quantity used in design is not always clear.

It is concluded that, in principle, serviceability problems are easy to identify as requiring consideration in design; however, it is difficult to deal with it through specific prescriptive design. Therefore, serviceability requirements should be relatively flexible and enable engineers to apply various options. A similar meeting was held in Europe the same year (Gothenburg, 1988) organized by IABSE and CIB. Several contributions were focusing on serviceability requirements using e.g. probabilistic (Holický and Östlund, 1993) and risk-based approaches (Leicester, 1993); however, on a rather theoretical level.

According to Deák and Holický (1993), to specify limit values for serviceability parameters the following attributes should be stated:

- Considered serviceability requirement;
- Structure or structural element to be verified;
- Serviceability parameter and its limit value;
- Corresponding probabilistic measures (probability or unserviceability);
- Design situations to be considered;
- The load combinations to be taken into account;
- Recommended simplified rules (e.g. limiting span/depth ratio), possible structural solutions including detailing to reduce risk of unserviceability.

This is obviously not an easy task, therefore a simplistic categorisation and linking of structural requirements and performance criteria is needed and design format is proposed in the Eurocodes, see (Lüchinger, 1996).

4.2 Target reliabilities

The target reliability indicex suggested for irreversible serviceability limit states in EN 1990 (2002) is β=2.9 considering a reference period of 1 year for structures in reliability class 2, i.e. structures with medium consequences of failure. More sophisticated values are proposed by the JCSS (2001) based on the relative cost of "safety measure" ranging from 1.3 to 2.3 from high to low costs of improving serviceability.

It should be emphasized, that the reference period and associated target reliabilities for SLS cannot be purely derived from the properties of the structure (Tichý, 1993). They need to be determined by decisions. These decisions should be based on opinions and needs of individuals, groups, social entities and economic analyses and should reflect previous experience with similar structures.

Perhaps the most fundamental aspect when deciding on target reliabilities is the importance of the structure for individuals and the society. This is rather difficult in case of SLS, because limited studies exist compared to ULS. Also, aspects governing the probability of failure for SLS differ substantially from those related to ULS. The main difference is that whilst it is generally accepted that ULS shall never be reached, the attainment of SLSs can be sometimes tolerated. As mentioned before the importance of the structure is a primary factor to consider when establishing its serviceability criteria. Performance demands on floors under gymnasiums, dancing halls, assembly halls, and others are much more rigorous than on floors under and over apartments. The difference in importance is illustrated through another example adapted from Tichý (1993).

Example. *Consider a building with 1000 rooms each room used by one occupant allocated randomly. Assume that one of the 1000 occupants is sensitive to any crack in the ceiling, while no cracks are ever noticed by any of the remaining persons. Denote the occurrence of a sensitive person in a room as event E1 and the occurrence of cracks in a particular ceiling as event E2. Assume that the floors of the building have been designed in a way that the probability of crack occurrence in a slab during the life of the building is exactly* $P_{rt} = P(E_1) = 10^{-3}$. *The probability that the crack sensitive person will be the user of a particular room is* $P_{sp} = P(E_2) = 10^{-3}$. *Since* E_1 *and* E_2 *are independent, the serviceability failure probability will be:*

$$P_f = P_{sp} \cdot P_{rt} = 10^{-6} \tag{1}$$

Consider now the entrance of the building and assume that the ceiling has been designed with the same probability of failure, i.e. 10^{-3}. *Since all occupants of the building, including the sensitive one, pass through the entrance every day, if cracking occurs, the sensitive person will notice. We can thus assume that* $P_{sp} = 1$ *and the serviceability failure probability will be:*

$$P_f = P_{sp} \cdot P_{rt} = 10^{-3} \tag{2}$$

If for the two facilities the same level of reliability should be achieved, the entrance should be designed for $P_{rt} = 10^{-6}$.

In public buildings the possible discomfort of people is invariably greater, and thus higher reliability targets might be required for public buildings than for buildings used by individuals or small groups. It should be noted that the distinction between public areas (with many people) and residential areas (with occupation only by single humans) regarding the ULS, is less related to the difference in the perceptions of individuals. It is rather related to consequences of ultimate failure, i.e. that a structural failure in populated areas may cause more harm. Therefore, the target reliability indices in standards could perhaps be distinguished according to the different requirements, i.e. if they are related to e.g. damage, reduced functionality or human perception and specific consequence classes for serviceability could be developed.

4.3 Serviceability criteria

As it was discussed a rational basis for the determination of serviceability limits to be implemented in standards is missing. Serviceability limits in general can be considered as random variables and their values should be determined through probabilistic analysis as highlighted by an example from Tichý (1993).

Example. *Consider a lecture hall which is regularly visited by a group of N individuals. Assume that the deflections of the floor increase with time e.g. due to creep. At a certain value the mid-span deflection u of the floor one of the regular visitors begins to be concerned about the safety of the structure. This value of u is the personal constraint u_{lim} of that specific visitor. As the deflections continue to increase, the number of alarmed visitors n increases as well. Assuming, that the visitors' concerns are non-transferable, additional Δn visitors will be concerned about the deflections at each lecture. The probability that a randomly selected visitor will get annoyed by $u \leq u_{lim}$ is given by:*

$$P = \frac{n}{N} \tag{3}$$

Similarly, the probability that a randomly selected visitor will get concerned just when u_{lim} has been reached is:

$$\Delta P = \frac{\Delta n}{N} \tag{4}$$

Everyone has a personal threshold, whose exceedance causes discomfort and since psychological and emotional properties of humans are random, the personal constraint u_{lim} is also a random variable. Consequently, if a large number of people is considered, Eq. (3) *and Eq.* (4) *can be interpreted as the cumulative distribution function and the probability density function of u_{lim}. If the probability distribution of u_{lim} is known, the value of admissible deflection u_{adm} can be calculated for an intended probability P_{lim} based on:*

$$Pr(u_{lim} \leq u_{adm}) = P_{lim} \tag{5}$$

Experimental data on the random behaviour of serviceability constraints is very scarce (or non-existent), therefore they usually are based on tradition, rather than rational consideration of probability of failure.

As pointed out by Tichý (1993), in SLS often the "average" loading conditions might be of interest. Thus, it might be recommended to derive appropriate load levels for SLS based on the mean, mode, or median of the physical realizations of load, rather than the characteristic values (i.e. 0.95-fractiles of the respective probability distributions). Similar considerations can be made concerning material characteristics. Tichý (1993) argues that significant efforts have been made to refine calculation models e.g. for bending stiffness and creep; however, the determination of probability-based values to be used in those models is often neglected.

Several physical reliability requirements, formulated for various serviceability criteria, must be checked. Only in very simple cases, such as floor beams, floor slabs, etc., a single deflection check is sufficient. During the evaluation one must not forget that some requirements, e.g. deformations should be verified also for several stages of the construction process, not only for the operational stage. Further, it should be kept in mind the time-dependencies involved: first, those related to loads, then those related to material, and finally also the time-dependence of the constraints themselves.

There is a substantial dependence between the calculation models and serviceability constraints. When values of constraints are specified in structural codes, they must be considered valid only for the given calculation model. A change in the calculation model can substantially affect the results of design. Members which are considered acceptable according to one calculation model, might be unreliable when verified with another one. This "meta-dependence" between calculation model and criteria can lead to problems whenever neither calculation model nor constraints are specified. Contract documents should always be clear on acceptable deflections, which should be preferably specified on the performance basis, not on calculation model basis. This issue seems to be independent from the construction material and, if addressed by structural codes, it should be dealt in generic parts, like e.g. EN 1990 (2002).

Time might have a significant effect on serviceability requirements, i.e. the older the structure the less sensitive the occupant might be to e.g. excessive deflections. Consider an example of an old timber frame, where the deformations accumulated over several decades would be unacceptable if they would occur in the first day of service. Someone buying an old farm house to spend only a small portion of the year there is little sensitive to large deflections of floor beams and might consider it unavoidable. When the deflections increase slowly and steadily, the occupants usually do not become suspicious about safety of the building even when they are excessive. Since serviceability problems are often subjective, setting up appropriate serviceability criteria might be based on risk assessment.

5 Conclusions

The paper has highlighted that the design and verification of timber structures with regard to serviceability is a rather complex issue, which is perhaps underestimated by both, the developers of structural codes and practicing engineers.

To better deal with this complexity, further discussions and continued research efforts are required in a variety of directions dealing with e.g. structural dynamics, structural reliability, rheology of wood etc. Some specific recommendations are as follows:

- Performance based formulation of serviceability criteria, possibly independent of material and type of structural realization;
- Definition of serviceability limits in direct relationship to specification of non-structural elements, i.e. better connect overall structural performance with the performance of construction products;
- More transparent and straightforward verification of serviceability limits, which are also or better accessible by modern tools design tools;
- Guidance on choosing the appropriate level of structural assessment with regard to serviceability, to avoid application of too simplistic checks for complex structures going beyond the scope of application.
- Implementation of new research findings on material specific parameters related to creep for a better forecast of the long-term structural behaviour.

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Deflections in timber buildings

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Abstract

The contribution reviews the state of the art of structural timber design regarding deflections with a focus on the relevant European structural codes and their background. The overall aim is to provide insights for further development of Eurocode 5 and facilitate the incorporation of new scientific knowledge and methods.

1 Introduction

1.1 General considerations

A detailed description of how structural deflections can be categorized is given by Galambos et al. (1973). The primary categories are static and dynamic deflections. The fundamental difference between them is due to the load/excitation causing the deflection, the structural characteristics influencing the response, and the effects on occupants and the structural subsystem. The current section deals with static deflections. However, dynamic deflections of structures are very important from a serviceability point of view and will be discussed in the next chapter of this COST report.

"Static" deflections here refer to deflections caused by static loads. A load can be considered static if applied and released slowly, i.e. within a time comparable to the natural period of the structure. In case of timber "static" (or perhaps more precisely quasi-static) loads cause creep, i.e. deflections increase in time even if the load is unchanged.

1.2 Background to the deflection limits of timber structures

It was recognised, a relatively a long time ago, that besides strength, also the stiffness of structural elements should be considered during design. Therefore, specifications for the determination of the required size of e.g. beams included limitations on deflections. A historical overview about the deflection limits for wooden beams is given by Percival (1979). He mentions that even at the beginning of the 19th century deflection limits existed based on the experience of carpenters and experiments done by engineers. Reference is made to Tredgold (1885), where a limit of 1/480 of the span is described (1/40 inch to a foot). According to Percival the limit was increased to 1/360 at the end of the century and further refined later including a distinction made depending on the function and loading (dead vs live) of the structural element.

Simultaneously, similar limits were developed for structural members made of other materials, e.g. steel girders (Fleming, 1941), and ever stricter limits were introduced for certain design situations. E.g. for supporting certain types of machinery deflections of beams were limited to a very small fraction of the span, sometimes to 1/2000.

Another review on deflection limits is given by Saidani and Nethercot (1993). The main finding of the paper is that the limiting criteria spread diversely throughout design codes and standards, scientific papers and reports, and how they are applied in engineering practice. It is also noted by the authors that there is a significant difference in loading and modelling assumptions, not only in the design criteria. Thus, as it is highlighted in Honfi (2013), it is utterly important to understand that limitations of deflection are always associated with a certain loading situation, which is related to the actual design format. Therefore, a deflection limit suggested in one design code, might be inappropriate if used with loads taken from another one if they represent different probabilities of exceedance. Similarly, even using the incorrect load combination from the same code might be misleading.

It is clear, that deflection limits (and related load considerations) are rather based on tradition than rational economic optimisation. This sounds unreasonable for structural elements and system, where deflections and deformations often govern the design, such as e.g. timber roof structures.

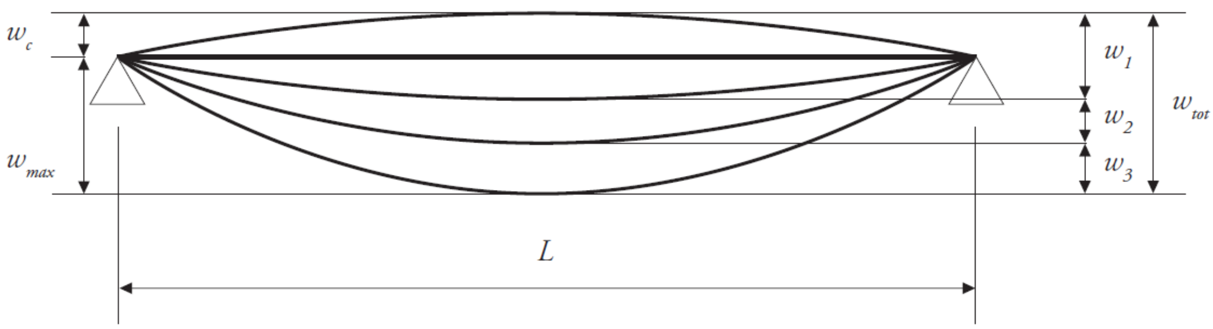

Figure 1: Definition of vertical deflections of beams according to EN 1990 (2002).

2 Eurocode recommendations

2.1 General recommendations (EC0)

Concerning vertical and horizontal deformations the appropriate combinations of actions should be taken into account with the relevant serviceability requirements. Special attention should be given to the distinction between reversible and irreversible limit states (EN 1990, 2002).

If the functioning or damage of the structure or to finishes, or to non-structural elements is considered, the verification for deflection should take into account those effects of permanent and variable actions that occur after the execution of the member or finish concerned.

If the appearance of the structure is being considered, the quasi-permanent combination should be used.

If the comfort of the user or the functioning of machinery are being considered, the verification should take account of the effects of the relevant variable actions. Long term deformations due to shrinkage, relaxation or creep should be considered where relevant, and calculated by using the effects of the permanent actions and quasi-permanent values of the variable actions.

The definition of vertical deflections according to EC0 (EN 1990, 2002) is shown in Figure 1, where w_c is the precamber of the unloaded beam; w_1 is the initial part of the deflection under permanent loads; w_2 is the long-term part of the deflection under permanent loads; w_3 is the additional part of the deflection due to the variable actions; w_{tot} is the total deflection as the sum of w_1, w_2, w_3; and w_{max} is the remaining total deflection taking into account the precamber.

Horizontal displacements of a building frame are presented schematically in Figure 2, with u denoting overall horizontal displacement over the building height H, and u_j representing the horizontal displacement over the height of one storey H_i.

2.2 Timber specific recommendations (EC5)

EN 1995-1-1 (2004) gives slightly different definition of deflections than EN 1990 (2002). The notation of deflections used in EC5 (EN 1995-1-1, 2004) are presented

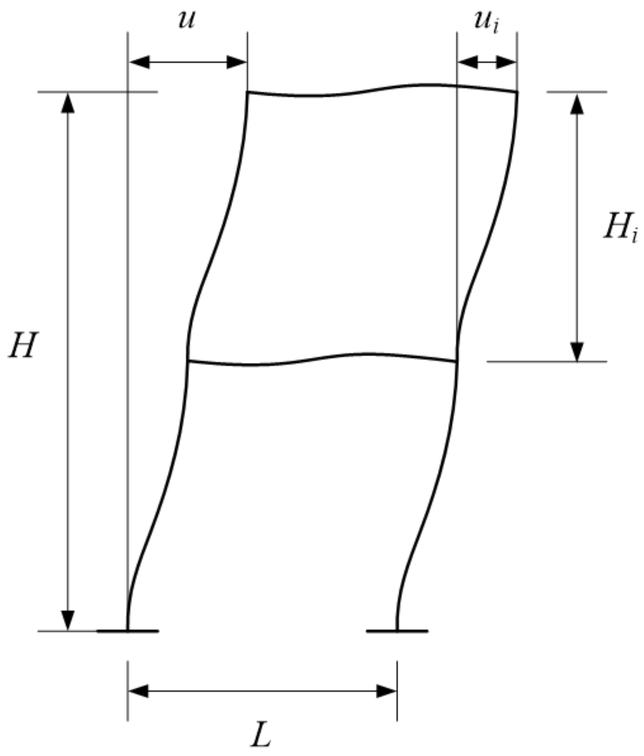

Figure 2: Definition of horizontal displacements according to EN 1990 (2002).

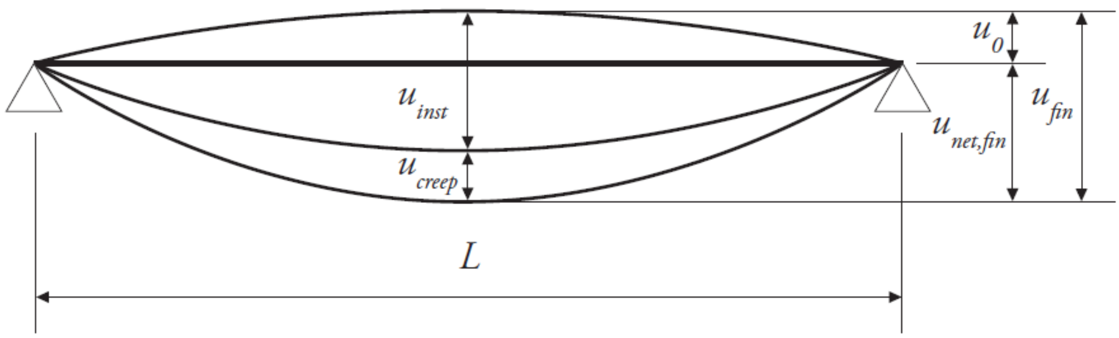

Figure 3: Definition of vertical deflections of beams according to EN 1995-1-1 (2004).

in Figure 3, where u_0 is the precamber; u_{inst} is the instantaneous deflection; u_{creep} is the creep deflection; u_{fin} is the final deflection; and $u_{net,fin}$ is the net final deflection. Already these different notations might create some confusion in the interpretation of the code prescriptions and make it difficult to check serviceability requirements in a consistent way.

The deformations of timber are time-dependent. The instantaneous deflections u_{inst} under an action should be calculated on the basis of mean values of the appropriate stiffness moduli. According to EC5 the final deformation u_{fin} of timber beams under long-term load should be calculated as:

$$u_{fin} = u_{inst} + u_{creep} = u_{inst} \cdot (1 + k_{def}), \text{ for permanent actions} \quad (1)$$

$$u_{fin} = u_{inst} + u_{creep} = u_{inst} \cdot (1 + \psi_2 k_{def}), \text{ for variable actions} \tag{2}$$

where k_{def} is the creep factor depending on the type of the wood-based material and the service class.

It should be noted that the deflections calculated in this way are not consistent with what is described in EC5 (EN 1990, 2002) for calculating the deflections from the quasi-permanent load combination. The reason for this is that Eq. (1) includes the deflections from the "rare" (characteristic) value of the variable action. This might lead to a significantly higher load level than e.g. the method used for the verification of concrete structures. This is illustrated in Eq. (3) and (4), by calculating the final and total deflections of a simply supported beam without precamber (and without the consideration of shear deformations) following EN 1990 (2002) and EN 1995-1-1 (2004) respectively.

$$\begin{aligned} u_{fin} &= \frac{5}{384} \frac{L^4}{E_{0,mean} I} \left[G_k \left(1 + k_{def}\right) + Q_k \left(1 + \psi_{2Q} k_{def}\right) \right] \\ &= \frac{5}{384} \frac{L^4}{E_{0,mean} I} \left(G_k + G_k k_{def} + Q_k + \psi_{2Q} Q_k k_{def} \right) \end{aligned} \tag{3}$$

$$\begin{aligned} w_{tot} &= \frac{5}{384} \frac{L^4}{E_{0,mean} I} \left(G_k + \psi_{2Q} Q_k \right) \left(1 + k_{def}\right) = \\ &\frac{5}{384} \frac{L^4}{E_{0,mean} I} \left(G_k + G_k k_{def} + \psi_{2Q} Q_k + \psi_{2Q} Q_k k_{def} \right) \end{aligned} \tag{4}$$

It should be noted that perhaps a meaningful deflection to limit would be the time dependent part of the permanent loads and the variable loads, i.e. Eq. (3) without the G_k part, see Thelandersson (1995). Limitations on deflections are recommended in EN 1995-1-1 (2004) and its national annexes. However, these values, as it is usual for deflection limits, are not compulsory.

It should also be mentioned that the EC5 recommendations are slightly different than the previous recommendations given in ENV 1995-1-1 (1993). The discrepancy between the EC0 and EC5 recommendations has been recognised and different deflection criteria based on full scale testing has been proposed e.g. by Bainbridge and Mettem (1997) and some National Annexes (see the next subsection). Furthermore, static deflections are sometimes limited in order to address issues related to dynamic deflections, i.e. vibrations. The latter issue is treated in another the part of this COST report.

2.3 National application rules

A detailed comparison of deflection limits from 29 European countries has been recently carried out by CEN/TC 250/SC 5 (2018). Detailed information on national

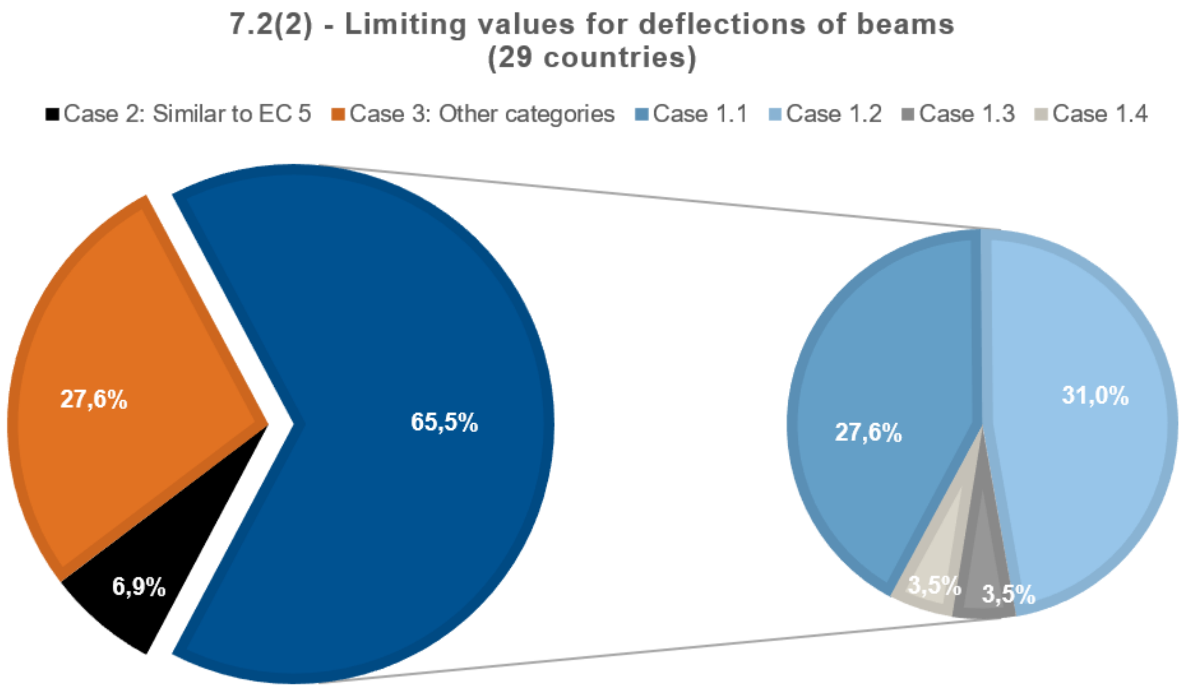

Figure 4: Overview of national choices for limiting values for deflections of beams in EC5; Case 1: Categories like in EC 5, Case 2: Similar to EC 5, Case 3: Other categories EN 1995-1-1 (2004).

choices may be found in the National Annexes. An overview about the national choices for limiting values for deflections of beams is given in Figure 4.
The different cases in Figure 4 are described below:

- Case 1 (65.5%, 19 countries): categories like in EC 5: The categories of the limiting values for deflections in the National Annex are the same like in or in the range of EC5;
  - Case 1.1 (27.6%, 8 countries): countries recommend the same ranges of limiting values like in EC5;
  - Case 1.2 (31%, 9 countries): countries recommend fixed limiting values for deflection;
  - Case 1.3 (3.5%, 1 country): countries recommend ranges of limiting values which differ from C5;
  - Case 1.4 (3.5%, 1 country): countries recommend fixed values as well as ranges of limiting values depending on the categories of the limiting values for deflections.
- Case 2 (6.9%, 2 countries): similar to EC 5, i.e. at least one category of the limiting values for deflections in the National Annex is the same like in or in the range of EC5. The other categories are different.
- Case 3 (27.6%, 8 countries): Other categories: The categories of the limiting values for deflections in the National Annex are different from the ones in EC5.

Table 1: Suggested deflection limits for simply supported beams in Austria.

Structures	Recommended values for deflection	
Deflections due to load combination	w_{inst}	$w_{net,fin}$
Impacts on the structure	not reversible (damage avoidance)	reversible (appearance)
Structures e.g. ceilings, accessible roofs and similar used structures	$L/300$	$L/250$
Structures, where the deflection negligible, e.g. not or just for maintenance accessible roofs, roof and ceiling constructions	$L/200$	-

To illustrate the differences between the National Annexes, a brief description about the verification procedure and the deflections limits is given in the following subsections.

2.3.1 Austria (Case 1.4)

Perhaps to avoid the confusion mentioned in the previous subsection, a detailed description about the calculation of the deflections is given in the Austrian National Annex. The instantaneous deflection w_{inst} is limited to avoid irreversible impacts on structures, e.g. to ensure the functionality of the components and to avoid damages on subordinate structures. The verification is calculated for the characteristic combination of actions according

$$w_{inst} = \sum_{j\geq 1} w_{inst,G,j} + w_{inst,Q,1} + \sum_{i>1} \psi_{0,i} w_{inst,Q,i} \tag{5}$$

The deflection due to the self-weight of the structure can be disregarded, if this deflection does not have any negative effects on subordinate structures (e.g. partition wall). The net final deflection $w_{net,fin}$ is limited to avoid reversible impacts on structures, e.g. appearance requirements and/or the wellbeing of the user. The verification is calculated for the quasi-permanent combination of actions:

$$w_{net,fin} = w_{inst,2} + w_{creep} - w_c = \left[\sum_{j\geq 1} w_{inst,G,j} + \sum_{i\geq 1} \psi_{2,i} w_{inst,Q,i} \right] \left(1 + k_{def}\right) - w_c \tag{6}$$

In the above equation $w_{inst,2}$ refers to the instantaneous deflection for the quasi-permanent combination of actions. The limiting values for simply supported beams are given in Table 1.

For cantilevering beams the length should be considered with the double length of the cantilevering beam.

Table 2: Deflection limits recommended in Germany.

	w_{inst}	$w_{net,fin}$	w_{fin}
Ordinary components	$L/300$ ($L/150$)	$L/300$ ($L/150$)	$L/200$ ($L/100$)
Pre-cambered elements; secondary components, such as for agricultural buildings; rafters and purlins	$L/200$ ($L/100$)	$L/250$ ($L/125$)	$L/150$ ($L/75$)

*For deflection-sensitive structures lower values could be applied.
The values given in brackets are applied for cantilevering beams.

Table 3: Deflection limits suggested in France.

	Common buildings			Agricultural and similar buildings		
	$w_{inst,Q}$	$w_{net,fin}$	w_{fin}	$w_{inst,Q}$	$w_{net,fin}$	w_{fin}
Rafters	-	$L/150$	$L/125$	-	$L/150$	$L/100$
Structural elements	$L/300$	$L/200$	$L/125$	$L/200$	$L/150$	$L/100$

2.3.2 Germany (Case 1.2)

The German National Annex essentially recommends the same principles for calculation as the one from Austria; however, in a more concise way. When determining the final deformation, the initial deformation u_{inst} and the proportion of creep in the quasi-permanent combination shall be taken into account. For structures comprising members, components and connections all having the same creep behaviour, the final deformation $u_{net,fin}$ may be calculated as follows, assuming a linear relationship between actions and deformations:

$$u_{net,fin} = \left(u_{inst,G} + \sum_{i \geq 1} \psi_{2,i} u_{inst,Q,i} \right) \left(1 + k_{def}\right) - u_c \tag{7}$$

Recommendations on deflection limits are based on the importance of the component or the structure as shown in Table 2.

2.3.3 France (Case 3)

According to the French National Annex, w_{inst} is calculated for the characteristic combination of actions, whereas w_{creep} is determined for the quasi-permanent combination of actions and with coefficient k_{def}. The deflection limits suggested in the French NA are given in Table 3.

$w_{inst,Q}$ in Table 3 refers to the part of the instantaneous deflection caused by variable actions. For floor panels or roof supports $w_{net,fin}$ should be less than $L/250$

under distributed load. For cantilevers values twice as much as for beams should be used, but not lower than 5 mm.

2.4 Limitations of current rules

2.4.1 Horizontal displacements

In EC0, horizontal displacements are only defined for multi storey frame structures. These inter storey drifts cannot be applied to high, single frame or arch structures. Further regulations could possibly be dropped by better knowledge about the structural requirements for the lining or cladding. For very tall buildings, dynamic interaction due to the dynamics of wind in dependency of critical masses should be investigated.

2.4.2 Wood based products

If optical criteria or expectations for some roofing structures with respect to outliers of the MOE are quite high, the risk of SLS performance failure should not be balanced by an increase of partial safety factor or extra high loading like the characteristic load combinations instead of most frequent loading, but better be handled by the right choice of the building material. Today, besides the basic type of solid wood, the engineer might choose among a wide range of wood-based products like glulam or LVL, which are characterised not only by improved strength parameters, but also by smaller scatter in the stiffness. High performance wood-based products should not be punished by low performance materials like members of solid wood with low grading class.

2.4.3 Importance of structural modelling

Some narrow limitations for timber structures may also be related to the practice of too simplified structural modelling especially in the EC5. The present regulations might be the result of neglecting limitations of the beam theory (e.g. spring stiffness due to compression perpendicular to the grain), the nonlinearity of connections and appropriate adoption of MOE to varying moisture contents.

Today, typically using FEM, the structural characterisation of both simple and complex structures is much better supported than in the past and even common practice with other materials like steel, glass or concrete. Missing or incorrect modelling should not be compensated by inflexible and not transparent package solutions in EC5.

More accurate structural modelling could be rewarded by less restrictive limits. However, it is not always straightforward to decide, if a more sophisticated model really provides more accurate and reliable results. This is because it might require more inputs, which might be difficult to obtain and involve large uncertainties. Nevertheless, such minimum requirements for acceptable structural modelling could be included in EC5, not only in terms of general purpose, common sense principles, but in terms of practical rules with consideration of the facilities of commercial modern engineering software.

3 Main difficulties for designers

In practical design situations some interesting questions arise for which the codes might not give sufficient guidance. Such a question is if the calculation of the deflection between an isolated beam and a beam that makes part of a floor should not be differentiated, like it happens in the ULS, namely in tension and bending, with the consideration of a factor k_{sys}? The contribution of the secondary elements (floorboards and struts) also occurs in deformation and therefore it would make some sense to take advantage of this increase in the stiffness of a floor.

Another concern relates to the service classes (SC). It would be important to have a better definition of the service classes in EC5, for a more rigorous selection of k_{def} (which can make a big difference between classes 1, 2 and 3). For instance, the timber roofs can be considered class 1 or class 2 and the results are quite different. As an example, the Portuguese NA indicates consideration of class 2 for roofs and basements.

4 Recent research on timber deformations

4.1 Long-term deflections in EC5

The long-term behaviour of wood-based structures is strongly affected by the surrounding conditions, such as loading (duration, history and direction), moisture content (MC) (level, history and variations) and temperature (T) and relative humidity (RH) of the surrounding air, see e.g. Morlier (1994). Despite extensive research efforts, the structural response of timber under these complex conditions is not yet fully understood. The accurate prediction of long-term deflection of timber structures is thus difficult, especially since many of the influencing factors might not be known at the design stage. The long-term deflections in EC5 are considered using a creep factor k_{def}. The value of k_{def} depends on the service class representing the environmental conditions.

- SC1 is characterised by a MC corresponding to a temperature of 20°C and RH of the surrounding air only exceeding 65% for a few weeks per year. In SC1 the average MC usually does not exceed 12%. The value of k_{def} for solid timber and glulam in SC1 is 0.6.

- SC2 is characterised by a MC corresponding to a temperature of 20°C and the RC of the surrounding air only exceeding 85% for a few weeks per year. In SC2 the average moisture content usually does not exceed 20%. The value of k_{def} for solid timber and glulam in SC2 is 0.8.

Table 4: Relative creep of beams in natural environments (Ranta-Maunus and Kortesmaa, 2000).

Conditioning	Type of member	Maximum bending stress [MPa]	Relative creep after 1 year	Relative creep after 7 years
In a sheltered environment with T ranging from -10^0C to $+20^0$C and RH ranging from 50% to 90%. Initial MC of the timbers were 8-14% and subsequent annual minimum and maximum MC were 14% and 20%.	50 x 150 Pine	7	0.62	0.97
	50 x 150 Spruce	7	0.66	n.m.
	50 x 150 Spruce	2	0.42	0.60
	90 x 180 Glulam	2	0.44	0.65
	51 x 200 Kerto	2	0.67	0.99
	I-joist with 45x45 timber flanges and 6.5 thick hardboard web	4	0.68	0.98

- SC3 is characterised by climatic conditions leading to higher moisture contents than in SC2. The value of k_{def} for solid timber and glulam in SC3 is 2.0.

The creep factors consider the various aforementioned aspects of long-term behaviour in a simplified way and are typically extrapolated from limited datasets, especially concerning the duration of tests and the uncertainties regarding material characteristics and environmental changes in real structures.

In the following a non-comprehensive, brief review of some creep studies on either wood-based members or joints between wood-based members, which all appear to indicate that the k_{def} values given in EN1995-1-1 are somewhat understated for uncontrolled environments where RH and T are allowed to fluctuate.

4.2 Creep of timber beams in natural environments

Several experimental studies have been carried out measuring long-term creep of structural sized timber beams under various circumstances, such as different types of wood species and products (pine, spruce, glulam, LVL), different treatment options (nil, painted, creosoted, salt treated) and different environments (sheltered, heated, outdoors), see e.g. Gowda et al. (1996), Ranta-Maunus and Kortesmaa (2000).

Ranta-Maunus and Kortesmaa (2000) measured creep of timber beams for eight years under both naturally varying and sheltered environments. Some of the results are presented in Table 4.

For solid timber, glulam and LVL the k_{def} value (50 years) given by EN1995-1-1 for Service Class 2 is 0.8. Although in the case of the I-joist the hardboard web has inferior creep properties to solid timber, the majority of I-joist deflection

Table 5: k_{def} values proposed by (Le Govic et al., 1994).

Material	***Load duration***	***Service class*** SC1	SC2	SC3
Solid timber	Medium term	0.25	0.35	1.4
	Permanent	0.9	1.3	1.7
Glulam	Medium term	0.35	0.38	0.7
	Permanent	1.4	1.25	1.7

(>80%) is attributable to bending deflection which predominantly depends on the flange material (i.e. solid timber). It can be seen that the relative creep after 7 years exceeds the EN1995-1-1 k_{def} value in the case of pine, LVL and the I-joist whereas in the case of spruce (lower stress level) and glulam the relative creep values appear to align reasonably with the EN1995-1-1 k_{def} value.

Le Govic et al. (1994) suggest an update of k_{def} values for SC1, SC2 and SC2 based on advanced numerical modelling for medium term (6 months) and permanent (50 years) loads. Results for solid timber and glulam are given in Table 5.

According to the model, the EC5 values for k_{def} are typically too low for SC1 and SC2, but perhaps too conservative for SC3.

4.3 Creep effects in timber joints

Feldborg and Johansen (1987) studied the behaviour of different types of timber joints under long-term loading (784 days) at constant and cycling RH. The studied joint types were a) nail plates, i.e. punched metal plate fasteners (PMPF joints), b) steel plates with annularly threaded nails, c) plywood gussets with square plain-shank nails and d) plywood gussets with annularly threaded nails. The applied load was about 40% of characteristic short-term strength (the estimated highest load with a slip not more than 15 mm.). A comparison of the measured relative creep values is given in Table 6.

For timber joints the k_{def} values given by EN1995-1-1 are 1.2 and 1.6 for SC1 and 2 respectively. For timber-plywood joints the k_{def} values given by EN1995-1-1 are 1.39 and 1.79 for SC 1 and 2 respectively. From Table 5 it can be seen that cycling the RH between 50% and 85% causes relative creep values, even after ≈2 years, well in excess of these values. Considering a design lifetime of 50 years, van de Kuilen (1999) proposed an updating of k_{def} values based on long-term (up to 5000 days) measurements and numerical modelling (Table 7). The load levels used in these studies was at 30, 40 and 50% of the average short-term strength.

Table 6: Relative creep of timber joints (Feldborg and Johansen, 1987).

Conditioning	Type of joint	Relative creep after 2.15 years
Constant RH of 65%	PMPF joint	0.6
	Steel plates and annularly threaded nails	0.4
	Plywood gussets and square plain-shank nails	1.2
	Plywood gussets and annularly threaded nails	0.7
Constant RH of 85%	PMPF joint	0.7
	Steel plates with annularly threaded nails	0.4
	Plywood gussets with square plain-shank nails	1.7
	Plywood gussets with annularly threaded nails	1.3
9.5 cycles of RH varying between 50% and 85%	PMPF joint	6.2
	Steel plates with annularly threaded nails	3.0
	Plywood gussets with square plain-shank nails	8.3
	Plywood gussets with annularly threaded nails	6.3

Table 7: k_{def} values proposed by Van de Kuilen (1999).

Conditioning	Type of joint	Creep factor k_{def} for permanent load duration
Based on measurements in an uncontrolled environment, whose climate could be regarded as SC2	Nailed joints	4.3
	Toothed-plate joints	4.4
	Split-ring joints	6.5

5 Proposal for changes

5.1 Consistency with other parts of Eurocode

In general, it would be beneficial, if issues related to serviceability could become independent from the materials and building systems on the level of performance-based regulations respective limitations. They could be compiled in EC0 as pool for basic principles of design, also for SLS. EC5 should only provide support, how to fulfil the requirements of EC0 in terms of material parameters and adequate structural modelling.

5.2 Probabilistic basis of SLS

As it was discussed a rational basis for the determination of deflection limits is missing. Deflection limits in general can be considered as random variables and their values should be determined through probabilistic analysis as highlighted e.g. by Tichý (1993).

Honfi et al. (2012) showed that the reliability of beams, including structural timber, in SLS is not consistent, with regard to the proportion of variable and permanent actions. Moreover, they might be well below the intended target reliability of EC0, i.e. $\beta = 2.9$ for irreversible limit states (considering a 1-year reference period) when the current load combinations, recommended deflection limits and stated creep factors are used.

5.3 Handling of non-structural elements

Some deflection criteria in EC5 are implemented to avoid damage of non-structural elements without further information. It seems to be a matter of communication/exchange of design parameters and expectations of the users or the providers of such non-structural elements e.g. claddings. Nevertheless, a principle within EC5 requires compatibility of displacements at every time without damage.

This issue could be simplified, if the assessment would be more related to building practice and supported with structured and representative information in the standard. As a benefit, the spectrum of different limits related to this issue could be dropped and will no longer remain as barrier to potential and future building developments of non-load bearing members. This approach is much more flexible and easily expandable to meet the demand of the building market.

5.4 Simplified verification of criteria

According to Colling (2017) the requirement on the verification of elastic deflection is questionable, because to avoid damage to adjacent components, the creep must be considered anyway. Therefore, two verifications are recommended:

1. The verification of final deflections (w_{fin}) calculated from the instantaneous deflection for the characteristic combination ($w_{inst,char}$) and the creep deformation using the quasi-permanent combination ($k_{def}w_{qs}$). This is associated to the irreversible limit states ($w_{limit,1}$).

2. The verification of net final deflections ($w_{net,fin}$) calculated from the instantaneous deflection for the quasi-permanent combination ($w_{inst,qs}$) and the creep deformation using the quasi-permanent combination ($k_{def}w_{qs}$). This is associated to the reversible limit states ($w_{limit,2}$).

Furthermore, absolute limits for the deflections are recommended based on a proposal from BDF Merkblatt 02-04 (2015). This concept and the actual values can perhaps be further discussed. However, according to the authors, such initiatives from practicing engineers should be considered when updating the Eurocodes. Furthermore, it is important that the formulation of the code should be unambiguous and based on sound scientific principles.

6 Conclusions

The verification of timber deflections can be a relatively complex issue due to a number of reasons, such as time and moisture dependent material behaviour, system effects, unclear definition of appropriate load levels, uncertainty concerning deflection limits, subjective perceptions of serviceability performance. Specific deflection limits is difficult to set, thus a performance based approach might be more appropriate than using prescriptive design rules. Clear guidance is, however, essential to support the designer and the client in evaluating adequate serviceability performance. Such guidance must rest on a rational basis and thus be developed involving probabilistic assessments of serviceability.

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Vibrations in timber buildings

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Abstract

The contribution reviews the state of the art of structural timber design regarding human induced vibrations with a focus on the relevant European structural codes, their background and recent research results. The purpose of the study is to provide useful information for further developments of Eurocodes and facilitate the penetration of new scientific knowledge into standardisation.

1 Introduction

If application and/or release of loads acting on a structure is fast, i.e. their duration is small compared to the natural period of the structure, the load and the resulting deflections are dynamic. Thus, the dimension of time needs to be considered to when analysing structural performance. Dynamic loads can be transient or cyclic. Transient loads are short in duration i.e. less than the natural period of structure. Cyclic loads last longer but change constantly causing a periodic steady-state (Galambos et al., 1973). Either way, dynamic loads may cause dynamic deflections, i.e. vibrations. Vibrations of structural members then might lead to unwanted consequences associated with human discomfort, malfunction of equipment and machinery, and damage to structural and non-structural elements (e.g. due to fatigue). Traditionally dynamic problems were often reduced to static problems by the application of equivalent static loads. This is often reflected in standards, when limitations on static deflections are prescribed to ensure that the effect of vibrations is reduced. Vibration leading to discomfort of building occupants can be divided into whole-body vibrations or vibrations which influence only a part of the body. Extensive research has been carried out in the last decades concerning vibrations of structures e.g. (e.g. Crist and Shaver, 1976; Ohlsson, 1982; Ellingwood and Tallin, 1984; Bachmann and Ammann, 1987). However, there are no clear limits for acceptable magnitudes of these vibrations. General guidance on vibrations is given in ISO 2631-2 (2013); ISO 10137 (2007). However, the response of humans to vibration is highly subjective and dependent on many factors. Inter-subject differences relate to the fact that people react differently to the same vibrations, whereas intra-subject differences refer to the difference in the response of the same person to the same vibration under different circumstances (Pavic and Reynolds, 2002).

The dynamic behaviour of a timber floor is determined by several factors, such as its mass, stiffness, damping and geometrical and structural characteristics, namely the existence of struts, the thickness of the floorboard, the type of connection between beams and walls, etc. In most cases, the floor stiffness ensures a satisfactory dynamic behaviour. However, it also happens that floors designed to meet the criterion of deformation exhibit vibration problems. The traditional deflection criterion does not always guarantee satisfactory vibration behaviour (Hu et al., 2001) and therefore it is important to limit the lowest values of the frequencies or other criteria, in order to fulfil comfort and safety requirements.

In an occupied building, with high permanent loads, the increased mass may decrease the floor natural frequencies to critical levels, since timber floors themselves have low mass (50-100 kg/m^2). This reinforces the idea that the application of heavy materials on timber floors, such as concrete slabs, can result in unsuitable dynamic behaviours. This can be a problem when rehabilitating a residential building with several apartments in the different floors, in which mass would be important to solve the acoustic insulation – while solving the acoustic behaviour, the mass might be a problem for the structural behaviour.

In order to understand the real in-situ dynamic behaviour of existent timber floors, some non-destructive tests can be performed. The most effective one is the dynamic testing using ambient vibration, which allows the identification of the mechanical characteristics of structures, namely timber floors. The existence of highly sensitive sensors allows testing without imposing a forced excitement on the structure and considering only environmental dynamic actions, such as wind, traffic, movement of persons etc.

2 Eurocode recommendations

2.1 General recommendations

2.1.1 Eurocode 0 recommendations

In Eurocode 0, EN 1990 Basis of structural design (EN 1990, 2002), the following clauses regarding floor vibrations can be found (A1.4.4):

- To achieve satisfactory vibration behaviour of buildings the comfort of the user and the functioning of the structure should be considered. Other aspects may be considered for each project and agreed with the client.
- The natural frequency of vibrations of the structure should be kept above an appropriate value which depends upon the function of the building. This may be agreed with the client and/or the relevant authority.
- If the natural frequency of vibrations of the structure is lower than the appropriate value (*which is not given here*), a more refined analysis of the dynamic response of the structure, including the consideration of damping, should be performed. A note of guidance is referred to standards (EN 1991-1-1, 2004; EN 1991-1-4, 2005; ISO 10137, 2007).
- Possible sources of vibration that should be considered include walking, synchronised movements of people, machinery, ground borne vibrations from traffic, and wind actions. These, and other sources, should be specified for each project and agreed with the client.

The exact requirements for floor vibration design, which could be compared to design values, is not at present given in EN 1990 (2002). It has been proposed that EN 1990 (2002) should state the exact performance requirements for floor vibrations and that the material based Eurocodes should give the design methods for floor vibrations.

2.1.2 Timber specific recommendations

Based on Ohlsson (1988) human sensitivity and perception to structural vibrations is:

- Related to floor vibration acceleration for frequencies which are lower than 8 Hz;
- Related to vibration velocity for frequencies which are higher than 8 Hz;
- Increased by the duration of vibration;
- Decreased by proximity to or awareness of the vibration source;
- Decreased by physical activities of the observer.

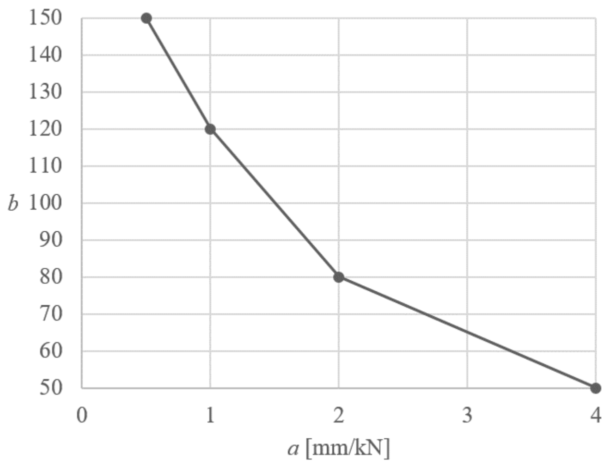

Figure 1: Relation of parameters a and b, which may be chosen as NDPs (EN 1995-1-1, 2004).

Considering the above as the basic reference, EC5 (EN 1995-1-1, 2004) gives a design method for residential floor vibrations, which is a mix of performance evaluation and performance requirements. Three separate requirements are set: a limit on the floor natural frequency, a point load deflection limit and a velocity limit caused by a unit impulse.

Firstly Eq. (1), it states that the fundamental frequency, applied with self-weight only, should be at least 8 Hz, and gives an equation to estimate this for one-way spanning floors. It is conservative to use it also for two-way spanning floors. Only self-weight is considered which could lead to an unsatisfactory floor design. If this condition does not apply, a special investigation should be done (which is not defined).

$$f_1 = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_l}{m}} > 8[\text{Hz}] \tag{1}$$

where m is the mass per unit area [kg/m^2], L is the floor span [m] and $(EI)_l$ is the plate bending stiffness of the floor along the span direction [Nm^2/m].

Two nationally determined parameters prescribe the required performance level. One is simply the allowable deflection Eq. (2) for a 1 kN static concentrated load a [mm/kN]. The other is a factor b used to calculate the allowed 'unit impulse velocity response', v [m/(Ns^2)]. A higher value of b means that higher velocities are allowed. EC5 does not provide equations to calculate the static deflection w, however equations to estimate v are given, which include the effect of the transverse stiffness of the floor and damping.

$$w \leq a\left[\frac{\text{mm}}{\text{kN}}\right] \tag{2}$$

The allowable value of the unit impulse max. velocity response v in Eq. (3) depends on the factor b, the fundamental frequency f_1 and the 'modal damping' ς, the latter two are dependent on the structure. If no damping values are available, EC5 recommends to use a damping value of $\varsigma = 0.01$. According to the British guideline by Harris et al. (2007) it should be at least 0.02, which significantly relaxes this criterion for normal timber floors. Other studies give even higher values, up to 0.06 (Feldmann et al., 2009). The Eurocode also gives a rather simple method to calculate the unit impulse velocity that is needed when compared to criterion in Eq. (3). This is based on a simplified expression considering all the frequency modes below 40 Hz.

$$v \leq b^{(f_1\varsigma-1)} \left[\frac{\mathrm{m}}{\mathrm{Ns}^2}\right] \tag{3}$$

where b is taken from Figure 1 and it is a NDP, f_1 is the natural frequency and ς is the damping value.

EC5 recommends that for a rectangular floor with overall dimensions $l \times b$, simply supported along all four edges, the value v may be approximated as:

$$v = \frac{4(0.4+0.6n_{40})}{m \cdot b \cdot l + 200} \tag{4}$$

where b is the floor width [m], n_{40} is the number of first order modes with frequencies below 40 Hz. This can be approximated by:

$$n_{40} = \left\{ \left[\left(\frac{40}{f_1}\right)^2 - 1 \right] \left(\frac{b}{l}\right)^4 \frac{(EI)_l}{(EI)_b} \right\}^{0.25} \tag{5}$$

where $(EI)_b$ is the equivalent plate bending stiffness [Nm^2/m] of the floor in the direction perpendicular to the span, where $(EI)_b < (EI)_l$. EC5 does not specify the equation or guidance to calculate the $(EI)_b$.

2.2 National application rules

2.2.1 Overview

In Zhang et al. (2013) an overview is given about how the EC5 related national application rules in 13 countries. Based on CEN (2017), the following differing approaches may be summarised on the national application of floor vibration design (based on a survey involving 28 countries), see also in Figure 2:

- 36 % (10) of the European countries apply the EC5 method as such.
- 25 % (7) have nationally recommended values for a and b.
- 11 % (3) have only a recommended a value and do not use the velocity criterion.
- 29 % (8) have other design methods applied.

Values for the parameters a and b for the method given in EC5 are given in most countries as nationally determined parameters. Often this is supplemented by

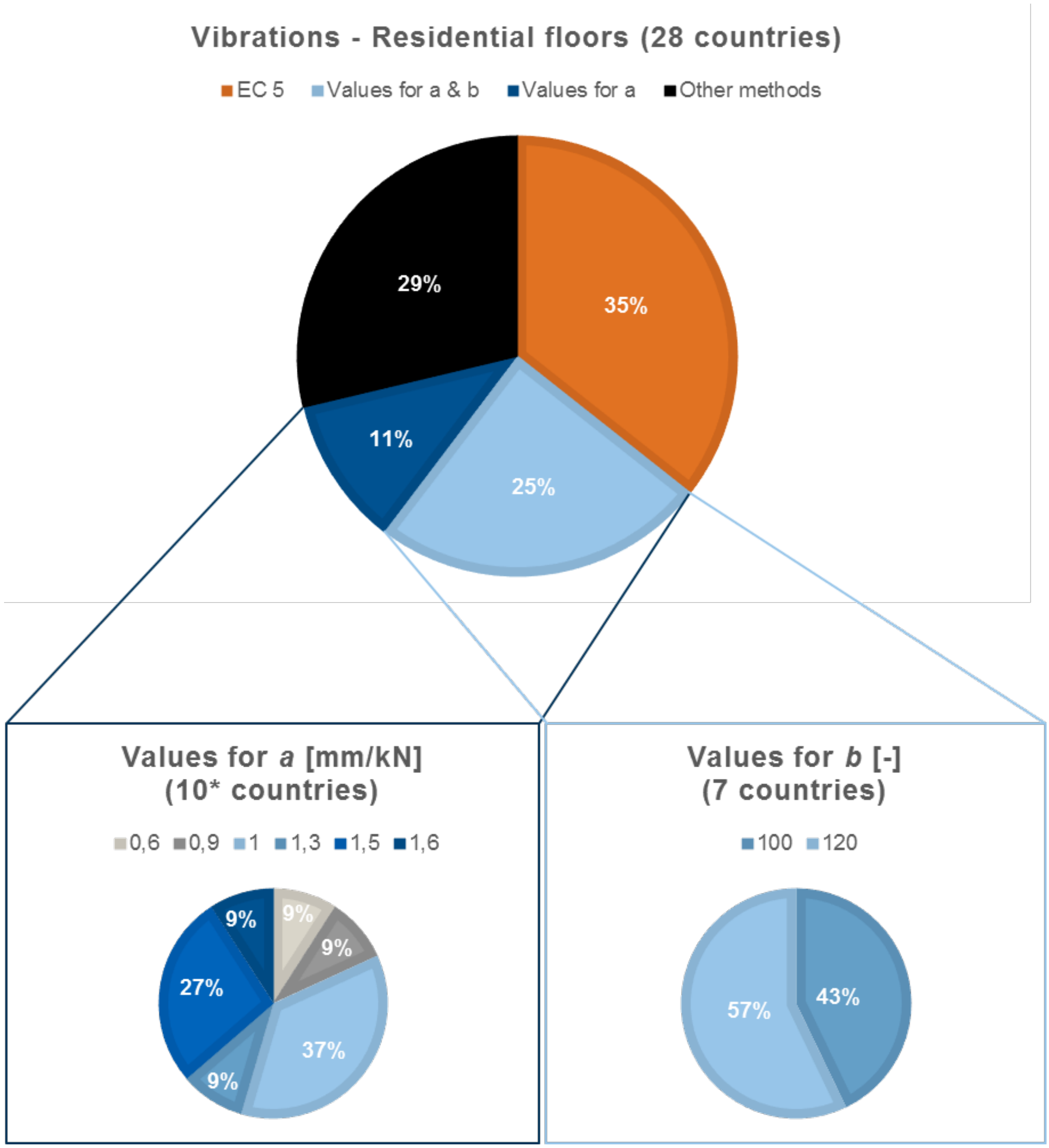

Figure 2: Overview of national choices for limiting values for vibrations of residential floors in EC5 (*) Norway recommends two values for a (CEN, 2017).

a differing approach. In a Nordic comparison Norway and Finland have much more onerous point load deflection requirements than Denmark and Sweden. Below is a brief overview of floor design approaches in some countries

In EN 1995-1-1 (2004) no recommendation of values for a and b are specified. The recommended range of limiting values of a and b and the recommended relationship between a and b is given in Figure 2. Information on the National choice may be found in the National annex.

2.2.2 Denmark

The EC5 method can be used with $a \leq 2$ mm/kN and with the recommended relation between a and b, but it is presumably not used in practice. A simplified method for span up to 6 m requires that the simple beam deflection for a point load is less than 1.7 mm/kN, which by experience ensures satisfying performance.

The Danish NA to EN 1990 (2002) gives common recommendations based on the natural frequency and the acceleration, which in principle supersedes the method in EN 1995-1-1 (2004). The recommendation for residential buildings is that the natural frequency should be larger than 8 Hz and the acceleration less than 0.1% of gravity (≈ 0.01 m/s^2).

2.2.3 Sweden

The Eurocode method is used with $a = 1.5$ mm/kN and $b = 100$ m/Ns^2.

2.2.4 Norway

sIt is recommended for span up to 4.5 m to use $a = 0.9$ mm/kN and $a = 0.6$ mm/kN if high stiffness is required. For longer spans, the natural frequency must be evaluated.

SINTEF (2011) propose to use the recommendation from a Canadian study (Hu and Chui, 2004) summarized later in this document with the limits $a < 1,3$ mm/kN and $f_1 > 10$ Hz.

2.2.5 Finland

The Finnish NA is strictly divided into performance and requirements, and gives equations to determine the performances for both one-way and two-way spanning floors. The section on vibrations is completely replaced, defining performance and requirements in terms of the simple deflection and natural frequency.

If the natural frequency is larger than 9 Hz it is sufficient to show that the simple deflection of the floor beam for a concentrated load is smaller than 0.5 mm/kN. If the longest side length of the room is smaller than 6 m larger deflections are accepted, for e.g. 4 m a deflection of 0.65 mm/kN is permitted.

The fundamental frequency for a two-way spanning floor may be calculated as:

$$f_1 = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_l}{m}}\sqrt{1+\left[2\left(\frac{l}{b}\right)^2+\left(\frac{l}{b}\right)^4\right]\frac{(EI)_b}{(EI)_l}} \tag{6}$$

where l is the floor span, in m; b is the width of room, in m; $(EI)_l$ is the equivalent plate bending stiffness of the floor in the main direction [Nm^2/m], $(EI)_b$ is the equivalent plate bending stiffness of the floor in the perpendicular direction [Nm^2/m], m is the mass per unit area calculated as the sum of the self-weight of the floor and the quasi-permanent value of the imposed load $(\psi_2 q_k)$, in kg/m^2.

The maximum deflection a caused by a point load may be calculated as:

$$a_1 = \min\begin{cases}\dfrac{l^2}{42k_\delta(EI)_l}\\ \dfrac{l^3}{48s(EI)_l}\end{cases} \quad \text{with} \quad k_\delta = \min\begin{cases}\sqrt[4]{(EI)_b/(EI)_l}\\ b/l\end{cases} \tag{7}$$

where s is the spacing of floor beams, in m. The term k_δ expresses the ability of the floor to distribute load between the beams. The other term is the deflection of a single beam, $s(EI)_l$ is the stiffness of one beam.

2.2.6 Germany

The German NA does not provide any equations, but it refers to the research project summarized in Hamm et al. (2010). The report is written in German and can be downloaded (Winter et al., 2010). Figure 3 shows the rules for design and construction, Table 1 the limit values.

For a single span beam the natural frequency can be calculated with help of Eq. (8).

$$f_{e,1} = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_l}{m}} = f_{beam} \tag{8}$$

If there are bearings on 4 sides, the frequency of a plate can be calculated by Eq. (9).

$$f_{plate} = f_{beam}\cdot\sqrt{1+1/\alpha^4} \quad \text{with} \quad \alpha = \frac{b}{l}\cdot\sqrt[4]{\frac{EI_l}{EI_b}} \tag{9}$$

m is the mass of the floor regarding only the self-weight of the floor b is the width of the floor. EI_l is the effective stiffness in longitudinal direction (stiffness of construction and stiffness of the screed). EI_b is the effective stiffness in transverse direction (stiffness of construction and stiffness of the screed), where $(EI)_l > (EI)_b$.

Table 1: Limit values for the frequency, deflection, acceleration and construction depending on the demands regarding the vibrations.

	demands regarding the vibrations		
	floors with higher demands	floors with lower demands	floors without demands
evaluation	1.0 to 1.5	1.5 to 2.5	2.5 to 4.0
installation position	floors between different units of use	floors between one unit of use	
during the research project examined type of use	e.g. corridors with low spans, floors between different users, floors in apartment buildings or floors in office buildings	e.g. floors within a single-family house, floor in existing buildings or with agreement of the owner	e.g. floors under not used rooms or in not developed attic storeys
description of perception of vibrations	Vibrations are not perceptible or only perceptible when concentrating on them. Vibrations are not annoying.	Vibrations are perceptible but not annoying.	Vibrations are clearly perceptible and sometimes annoying.
frequency criterion $f_e \geq f_{limit}$	$f_{limit} = 8$ Hz	$f_{limit} = 6$ Hz	-
stiffness criterion / deflection due to single load $w(2kN) \leq w_{limit}$	$w_{limit} = 0.5$ mm	$w_{limit} = 1.0$ mm	-
	demands on construction depending on type of construction		
Timber concrete composite systems	no more demands	no more demands	-
Massive timber floors, e.g. cross laminated timber or nail laminated timber floors	- heavy floating screed on a light or heavy fill - light floating screed on a heavy fill	- heavy floating screed (fill not necessary) - light floating screed on a heavy fill	-
timber beam floors	- heavy floating screed on a heavy fill - probably not possible	- heavy floating screed (fill not necessary) - light floating screed on a heavy fill	-

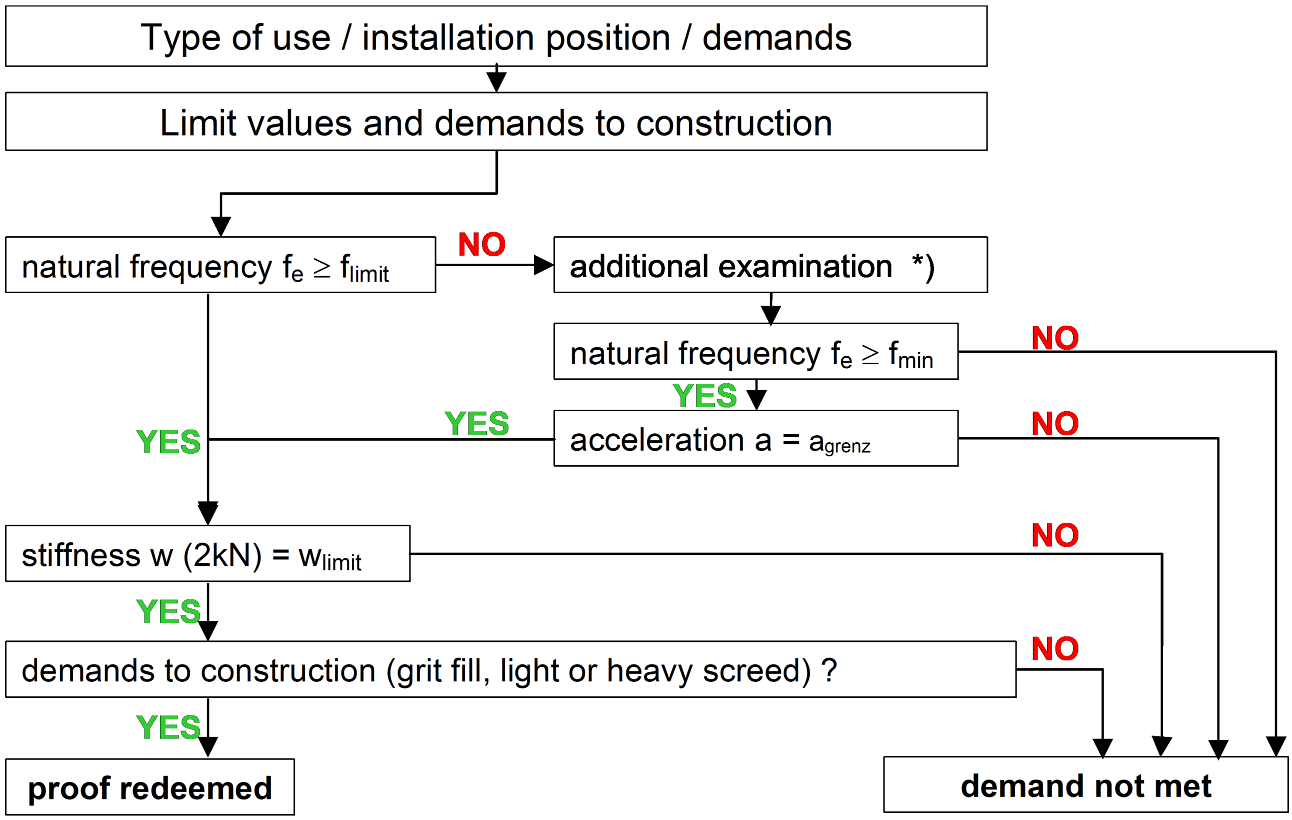

Figure 3: Chart of rules for design and construction.

The deflection due to a single static load of 2 kN should be determined by respect to the following:

$$w(2kN) = \frac{2l^3}{48EI \cdot b_{w(2kN)}} \leq w_{limit} \tag{10}$$

with

$$b_{w(2kN)} = \min \begin{cases} b_{ef} \\ b \end{cases} \qquad b_{ef} = \frac{l}{1.1}\sqrt[4]{\frac{EI_b}{EI_l}} = \frac{b}{1.1\alpha} \tag{11}$$

Independent of the original system this deflection should be calculated based on a substitute system of a single span beam with pin ended supports on both sides. The span of the substitute system should be taken as the greatest span of the original system.

If the construction is supported by elastic bearings (e.g. a beam below) the deflection of the elastic bearing must be regarded.

The stiffness of screed may be regarded and added to the stiffness of the construction (same as for calculating the frequency). If the natural frequency is lower than the limit value, resonance can occur. In this case the acceleration has to be calculated and compared to limit value (same as Austria), see Table 3.

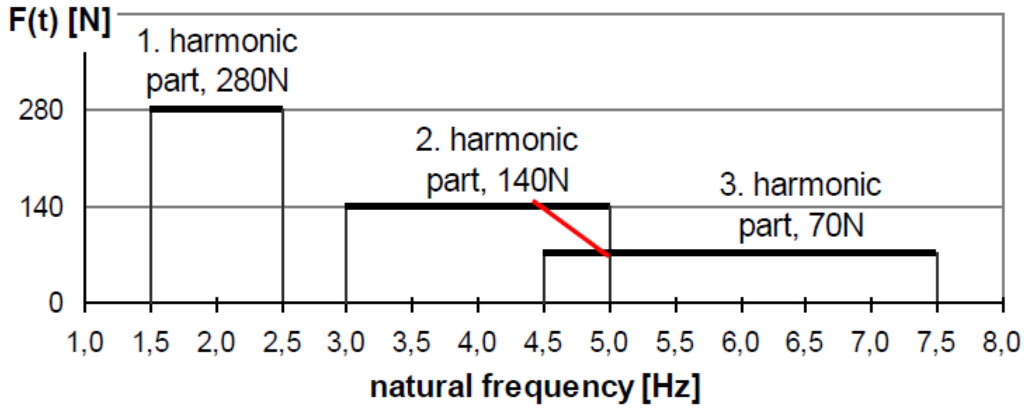

Figure 4: Force $F(t)$ depending on the natural frequency of the floor (Hamm et al., 2010).

Table 2: Values of modal damping ratios, taken from Winter et al. (2010)

Type of floor	Damping D [ ]
timber floors without any floor finish	0,01
plain glued laminated timber floors with floating screed	0,02
girder floors and nail laminated timber floors with floating screed	0,03
CLT floors without any or with a light floor finish, supported on two sides	0,025
CLT floors with heavy floating screed, supported on steel	0,025
CLT floors with floating screed, supported on four sides	0,035
CLT floors with floating screed, supported on timber walls on four sides	0,04
Timber concrete composite floors without any floor finish	0,025
Timber concrete composite floors with floating screed	0,035

The proof of acceleration is successful only in case of a heavy floor, such as timber concrete composite systems, or systems with wide spans.

$$a = \frac{F_{dyn}}{M^* \cdot 2D} = \frac{0.4 \cdot F(t)}{m \cdot 0.5l \cdot 0.5b \cdot 2D} \tag{12}$$

M^* is the modal mass of the floor. b is the width of the floor, but in the calculation b should be less than $b \leq 1{,}5l$. D is the damping of the structure. See Table 2. F_{dyn} is the total dynamic force. $F(t)$ are the harmonic parts of the force on the floor. They depend on the natural frequency and can be taken from Figure 4.

2.2.7 Austria

The method described in the Austrian NA is based on the German research project (Hamm et al., 2010). Some modifications are added, e.g. the link to the utilisation categories A, B, C1, C3.1 and D of EN 1991-1-1 (2004). The minimal floor weight of 50 kg/m^2 is assumed. Different floor classes ranging from I to III are introduced.

Table 3: Vibrational design limits of the ÖN B EN 1995-1-1 (2004)

Type of floor	Floor class I	Floor class II
Frequency	$f_1 \geq 8$ Hz	$f_1 \geq 6$ Hz
Stiffness	$w_{1kN} \leq 0,25$ mm	$w_{1kN} \leq 0,50$ mm
Acceleration	$a_{rms} \leq 0,05 m/s^2$	$a_{rms} \leq 0,10 m/s^2$

Class I includes floors with the highest vibration requirements and applies to floors between apartments, floors in office buildings, etc. Floors of class II applies to cases within the same apartment or in single family houses where the requirements are not as high. Floor class III is for floors with no requirements on vibrations such as floors without occupancy. Some prescriptive structural requirements for the floor types and floating floor screeds are also given.

The natural bending frequency and the point load deflection are to be evaluated and compared to the design limits listed in the Table 3. It is allowed to include the bending stiffness of the screed, also in the case without any composite action to the load bearing floor.

In the case of rigid supports and two-way spanning floors, the natural frequency is calculated as follows:

$$f_1 = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_l}{m}}\sqrt{1+\left(\frac{l}{b}\right)^4\frac{(EI)_b}{(EI)_l}} \tag{13}$$

$$b_F = \min\begin{cases}\dfrac{l}{1.1}\sqrt[4]{\dfrac{(EI)_b}{(EI)_l}}\\ b\end{cases} \tag{14}$$

where f_1 is the natural bending frequency [Hz], l is the floor span [m], b is the width of the floor [m], m is the mass of the floor (equivalent mass of the permanent load) [kg/m^2], $(EI)_l$ is the bending stiffness along floor span [Nm^2/m], $(EI)_b$ is the bending stiffness rectangular to floor span while $(EI)_b < (EI)_l$ [Nm^2/m], and b_F is the contributing width of the floor [m] in the deflection calculation.

For the deflection under a concentrated 1 kN load, the floor is assumed as a simply supported single span beam with the load placed at midspan. Multi-span beams are calculated as single span beams as well, in which case the highest span is used. The contributing width b_F of the floor is calculated with Eq. (14) and considers the transverse distribution of the 1 kN load.

Note that this is equivalent to the German stiffness requirement, where 2 kN is used (instead of 1kN), but has twice the limit values. If needed, it is allowed to go below the frequency limit of Table 3 but not lower than $f_{1,min} = 4,5$ Hz. When frequencies below the recommended design limits are requested the additional proof of the acceleration a_{rms} must be made (design limits as in Table 3). The acceleration can be calculated with the following.

$$a = \frac{0.4\alpha F_0}{2\zeta M^*} \tag{15}$$

where a_{rms} is the acceleration [m/s^2], α is the Fourier coefficient depending on the natural frequency $e^{-0,4f_1}$; F_0 is the weight of a person walking on the floor (estimated to $F_0 = 700$ N) [N]; ζ is modal damping, M^* is the modal mass.

The modal damping ratios ζ are given from 1% to 4% for a range from a bare floor to a fully built floor with a floating screed.

2.3 Background to the Eurocode criteria for vibrations of timber structures

The method described in EC 5 on vibrational serviceability design is based on Ohlsson (1988). The method described is applicable to high frequency residential floors where human footfall induced vibrations are considered. It is mentioned that human discomfort may also be caused by machine-induced vibrations, for this case the ISO 10137 (2007) vibration perception curve is referenced and no further design methods are given.

The Eurocode method is a combination of a static deflection and an impulse response method. It was originally developed only for timber floors (Ohlsson, 1988) and it is only applicable if the fundamental frequency is above 8 Hz. The deflection of a 1 kN point load is limited to a recommended value of 1.5 mm and the contentious unit impulse load is used to obtain a floor maximum velocity.

This method has been applied in numerous studies in many countries. From these studies, it has widely been found that the criteria do not perform satisfactorily, the method being too generous. The method approves floors that are rated as unacceptable by occupants. To correct this, it has been proposed to restrict the deflection criterion to a lower value. This is one reason for the numerous deviations from the method given in additional national application rules. The velocity criterion is generous as well and seldom becomes the dimensioning requirement, especially if realistic damping values are applied. The velocity criterion could also be further restricted, but this has not been proposed. An interesting feature of the velocity calculation is that it considers other natural frequencies than just the fundamental frequency. There is a simple procedure to estimate the number of modes below 40 Hz, which is considered as a limit for walking induced frequencies.

2.4 Limitations of current rules

Eurocode does not give any guidance as how to take into account the load sharing between floor beams when calculating the deflection or the fundamental frequency of two-way spanning floors. This may be due to that the method was primarily tuned for joist floors with relatively small stiffness perpendicular to the span. In such cases simple one directional floor bearing models may be sufficient. However, even this could be doubted. The current practice in the UK is that transverse stiffness is often provided solely by 22mm P5 particleboard. Even in such a case broadly 50 % of the point load will be distributed to adjacent joists to the one over which the point load is located.

Ideally, EC5 should enable the determination of the fundamental frequency and the deflection for a static concentrated load in a way that accounts for two-way spanning floors which is a more realistic model for modern timber floors.

3 Main difficulties for designers

Some difficulties concerning vibrations, when designing timber floors are discussed here.

Regarding the calculation of natural frequencies, Eq. (9) indicated by EC5 to estimate the theoretic frequency of a timber floor can lead to conservative results, resulting in lower theoretic frequency values compared to the real ones. Although the equation indicates the "stiffness of the plate equivalent to the floor in the direction of the beams $(EI)_{long}$", as this condition is difficult to simulate (due to the boundary conditions and to the contribution of the secondary elements, such as the floorboards and the struts), most of the designers tend to calculate the stiffness of an isolated simply supported beam, $(EI)_{beam}$.

Due to this fact, guidance on the calculation of a factor that would simulate the boundary conditions of the beams (embedment in the stone masonry walls) and the contribution for the overall stiffness of the floorboards, struts and ceilings (and their connection to the beams) would be very useful. In this case, $(EI)_{long}$ would be calculated using $(EI)_{beam}$ and the referred factor, as in Eq. (16).

$$f_{e,1} = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_{beam}(Sf)}{m}} \tag{16}$$

In this point one must note that there are authors, some involving experimental campaigns, that conclude that the real value of the frequency is usually quite higher than the one calculated with the equation considering the $(EI)_{beam}$. TRADA (2009) states that the frequencies obtained in situ are usually up to 50% higher than the frequencies estimated using the equation Eq. (9), suggesting its multiplication by a factor up to 1.5 (equivalent to increase the stiffness of a beam, $(EI)_{beam}$, with a factor (Sf) of 2.25).

According to Blaß (1995), in a common timber floor, the load distribution factor is 1.15, close to the one defined by the EC5 (1.1). Experimental dynamic tests were performed by NCREP in 16 different buildings (90 timber floors, see Figure 5 and Figure 6) showed a good approximation between the results of numerical and experimental frequencies when multiplying Eq. (9) by a value between 1.2 and 1.4

Figure 5: Dynamic identification of timber floors (with floorboards)

Figure 6: Dynamic identification of timber floors (without floorboards)

(equivalent to increase the stiffness of the beam with a factor (Sf) ranging between 1.5 and 2.0).

Most of the tested floors had floorboards, struts and ceiling; some had only one of those elements. The comparison between different timber floors (with and without floorboards, struts and ceilings) also showed:1) Floorboards: the stiffness is much more relevant than the mass for all modes; 2) Struts: the mass is more relevant than the stiffness for the 1st mode; the stiffness is much more relevant than the mass for the upper modes; 3) Plaster ceilings: the mass is more relevant than the stiffness for all the modes.

This condition is naturally also applicable to the calculation of the instantaneous deflections of the timber floors, as usually this calculation is performed for a single beam (usually, by simplification, for a simply supported beam). Future research in this field, including experimental campaigns and numerical modelling, is fundamental to help to define that factor. Finnish and Austrian NA already have an approach taking into account this subject, namely considering an equation for the fundamental frequency of a two-way spanning floor. German research and Austrian NA suggest to consider bending stiffness of beams and structural layers such as screed.

Another interesting question is related to the limitation of the natural frequencies. The 8Hz limit for the vibration frequency indicated in the EC5 is supposed to be for the calculation of the floor only with its own weight and permanent loads. Engineering intuition suggests that it could be more suitable to consider the effects of live loads as well to better understand the behaviour of the floors with different occupations (even if the method indicated in the EC5 is only for residential buildings). However, past experience from suggest that consideration of live loads will make the situation more complex. If, say, 30% of the live load, as it was in a former version of the German code, needs to be regarded, several problems arise on how to take into account the live load properly. Furthermore, live load decreases the calculated frequency and the difference between the calculated and real frequency might grow if the load is overestimated. On the other hand, the first issue could perhaps be treated with clear guidance and contribute to a better estimation of realistic response of the floor when loaded. In overall, this issue is not easy to resolve as e.g. the common areas of a building have different live loads from the apartments. However, it is important that the situation should be clearer in EC5 so as not to lead to different analyses by designers.

Furthermore, EC5 states that the "vibration frequency should be higher than 8Hz in residential buildings", otherwise a more specific analysis should be made. What analysis should this be? And if the use of the building is different from residential, such as in the cases of commercial buildings, offices, etc.? What should be the limits to consider? Although there is specialized literature on dynamic behaviour of timber floors, it would be important for designers to have some kind of guidance in the codes (namely EC5) on how to proceed in these cases.

4 Recent research on timber floor vibrations

In the followings some recent studies are listed which could be the basis of a proposal to be included in the new version of EC5.

4.1 JRC-report

General calculation methods and requirements valid for all materials are proposed in Feldmann et al. (2009), which is intended as background for future Eurocodes. It is based on the fundamental frequency, modal mass and damping. Several modes may be considered simultaneously. From these values, the floor can be assigned to a class indicating the performance, independent of the primary construction material. The method is based on a dynamic analysis of a representative human step load and comparing the weighted rms-velocity value with acceptable values based on recognized ISO standards. This representative human step is a 90% fractile value where different person weights, walking speeds and shoe properties are considered.

Damping values for floors of different material are given and the damping for a timber floor is in the JRC-report given as 6%.

The performance is rated to six classes. The requirement is given as a recommended class depending on the use of the building. The report contains both detailed equations for estimating the various parameters and very simple approximate equations. Further simplifications of the general approach can be made for each material. The analysis is rather simple for the user, being based on diagrams, from which knowing the natural frequencies respective modal masses and damping values the performance can be determined easily.

4.2 Swedish study

In Jarnerö (2014), new and more realistic vibration performance criteria for Sweden are suggested. Here classes A-D are introduced, with the aim to ensure the same level of user satisfaction as the similar classes already applied for sound insulation. A study was carried out on user perception for buildings fulfilling the requirements in EC5 applying the Swedish NA ($f_1 > 8$ Hz, $a \leq 1.5$ mm/kN, $b \geq 100$ m/Ns^2). Based on this, Jarnerö (2014) suggests a requirement of $a \leq 0.7$ mm/kN for class C, independent of f_1 (except that it should be higher than 8 Hz).

4.3 Finnish study

Floor vibration measurements and subjective ratings carried out over a ten-year period in Finland consisting of timber, steel and concrete as well as composite floors are summarised in Toratti and Talja (2006). Some of these experiments were carried out in laboratory conditions and some were done in the building. A floor classification was proposed for different end uses. It was proposed to limit the fundamental frequency to 10 Hz and the point load deflection of floors for residential floors to 0.5 mm (Figure 7). This was a basis for the Finnish NAD presented earlier. The rather high 10 Hz requirement on the fundamental frequency was based on having some margin not to fall to the low frequency range.

4.4 Criteria based on natural frequency dependent deflection — Canadian approach

4.4.1 Comparison to measured floors

As mentioned before Hu and Chui (2004) proposed, based on studies of user perception in Canada, the following requirement between the fundamental frequency f_1 and the point-load deflection a [mm] caused by a load of 1 kN.

$$\frac{f_1}{a^{0.44}} \geq 18.7 \tag{17}$$

This is represented by the black curve in Figure 8. The paper gives equations for determining the parameters taking also the transverse stiffness into account.

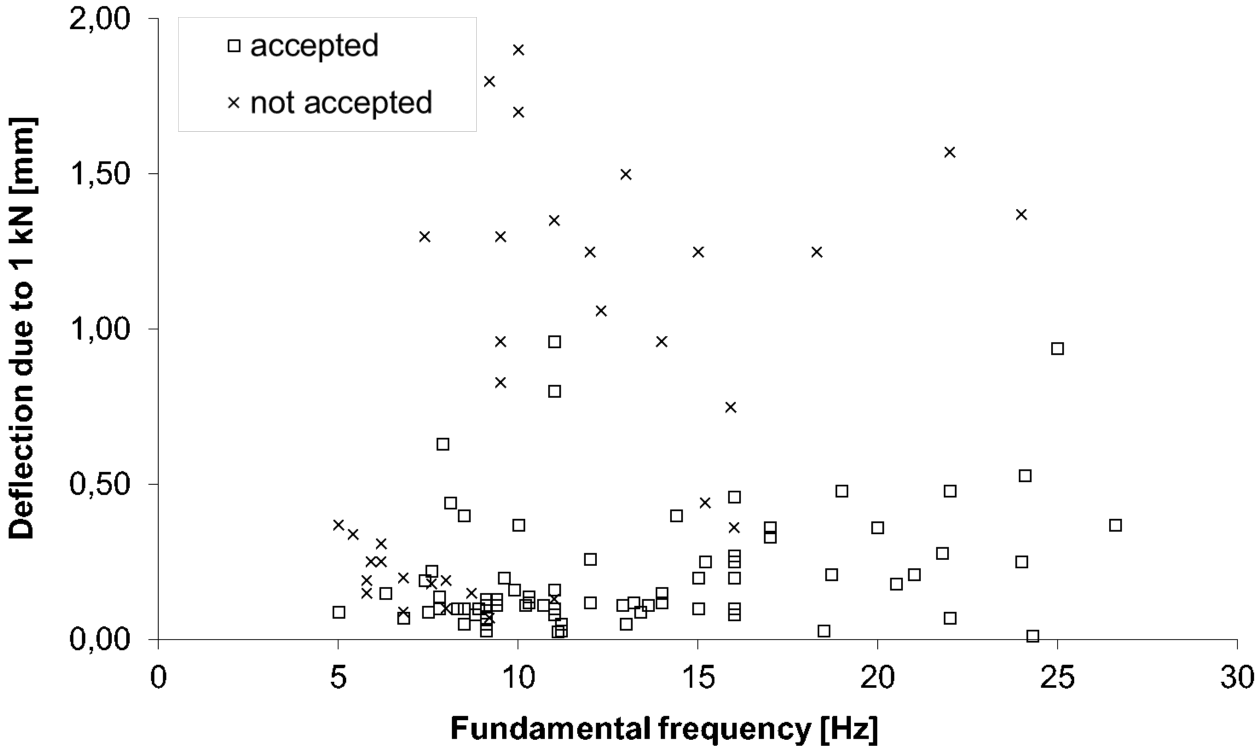

Figure 7: The combined result from Finland and the proposed point-load deflection requirement of 0.5 mm (Toratti and Talja, 2006).

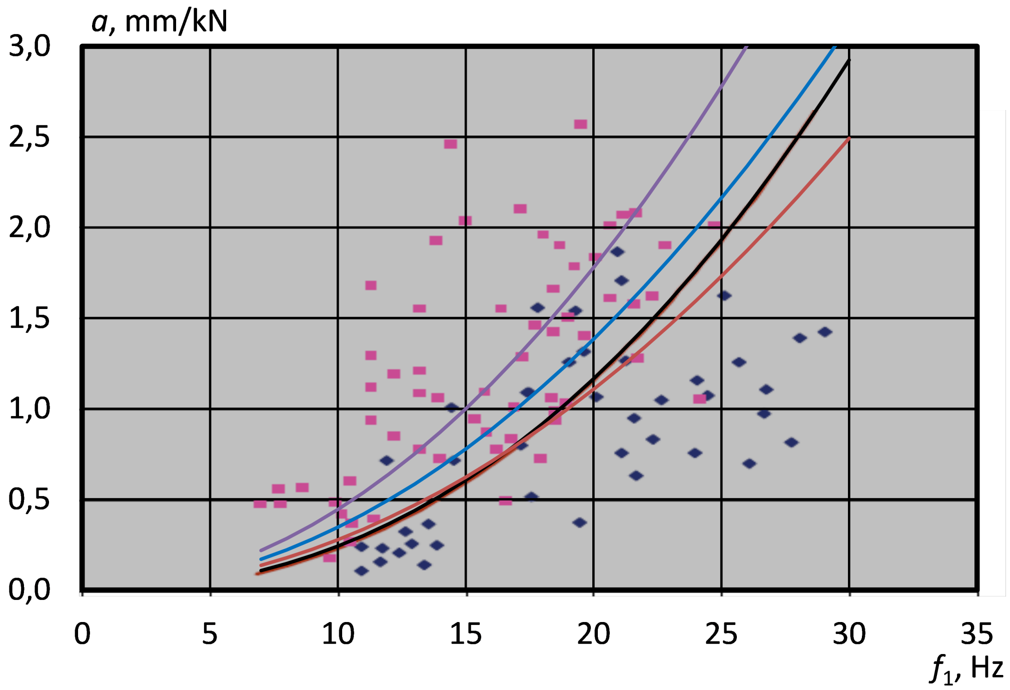

Figure 8: User perception for vibration of timber floors in Canadian studies. Purple marks indicate dissatisfied users, blue marks satisfied users, f_1 *is calculated using self-weight only, after Homb (2007). The black curve is the criteria proposed by Hu and Chui (2004). The red line is the simplification in Eq.* (17). *The blue and purple line represents constants 17 and 15 instead of 18.7 in Eq.* (17).

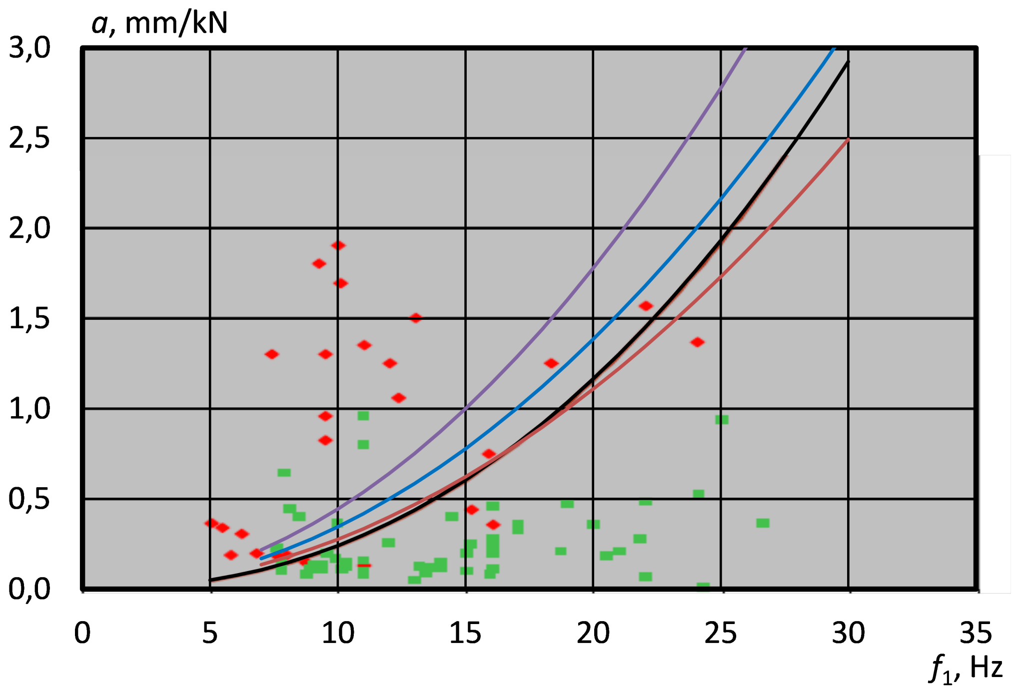

Figure 9: User perception for vibration of timber floors in Finland. Red marks indicate dissatisfied, green marks satisfied users. f_1 is measured. After Homb (2007).

In Homb (2007) the Canadian results (Hu and Chui, 2004) are compared with measurements of studies from Finland (Toratti and Talja, 2006) and from Norway Homb et al. (1988), see Figure 9 and Figure 10, where the black line represents Eq. (17). The Finnish study has structures with very low natural frequencies represented and the Norwegian study has an intermediate group for user perception, which can assist in defining classes. In all cases the Canadian equation seems to predict the user perception very well. Below 10 Hz it is most difficult to judge the necessary value for the deflection a from these results, but already for $f_1 = 12$ Hz, the deflection limit seems to be about $a \leq 0.4$ mm.

The consequence of the fundamental frequency f_1 being a measured value in Figure 9 and Figure 10, whereas it is calculated based on the self-weight in Figure 8 is not straightforward to estimate. But assuming that the measurements are taken in inhabited apartments the measured f_1 could be somewhat lower than if it was calculated based on the self-weight, presumably by about 2 Hz. This equals to shift the curves 2 Hz to the left. In Figure 8 the black curve will then be better in dividing good and bad floors, whereas in Figure 9 and Figure 10 it will mean that more dissatisfying floors will be ranked wrongly. It is obvious that other relations than Eq. (17) will represent the data with quite similar accuracy. The red line in the three figures is determined from the simpler form where the constants are rounded as follows:

$$\frac{f_1}{a^{0.5}} \geq 19 \tag{18}$$

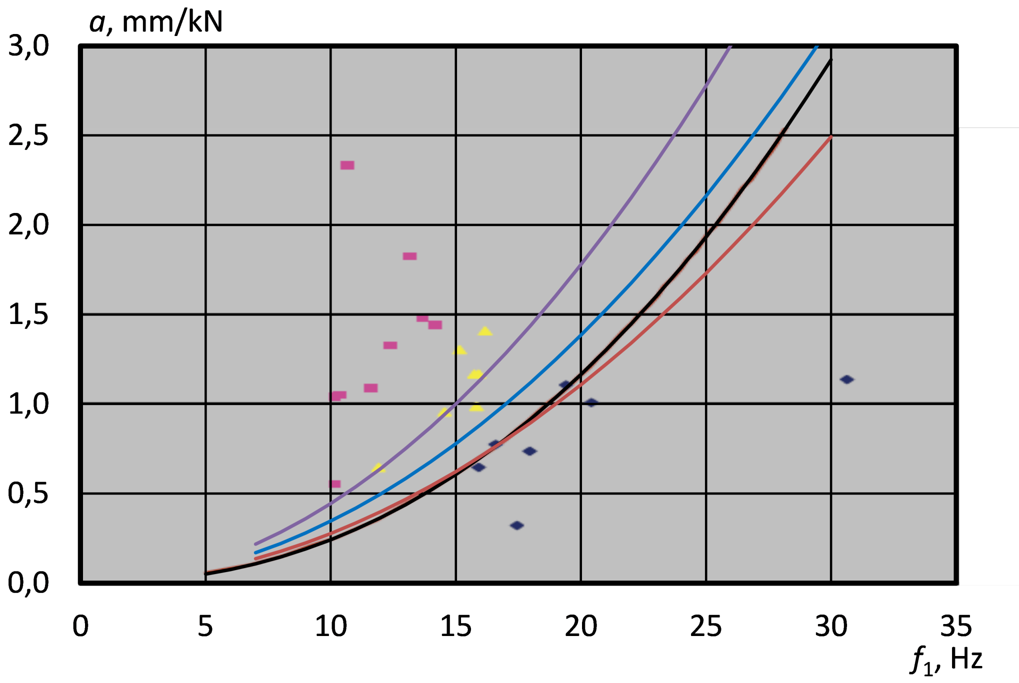

Figure 10: User perception for vibration of timber floors in Norway Purple marks indicate dissatisfied users, blue marks satisfied users and yellow marks border cases. f_1 is measured. After Homb (2007).

This deviates slightly from the black line for high frequencies, in an area with very few observations. The blue and purple lines represent constants 17 and 15 instead of 19 in Eq. (18). The number represents the necessary minimum fundamental frequency if the static deflection is $a = 1$ mm/kN. Floor vibration classes could perhaps be defined by this number.

It should be mentioned, that the measured values are usually higher than the calculated ones presumably due to the "hidden stiffness", e.g. in the non-bearing construction members or in the supports, which are assumed to be flexible in rotation.

4.4.2 Comparison of the Canadian proposal

The various requirements prescribe a minimum natural frequency between 8 and 10 Hz, but some criteria prescribe that the natural frequency should be determined with only the self-weight (as in EC5), in other cases the quasi-permanent part of the imposed load should be included. This decreases the natural frequency by about 2 Hz, which should be considered when comparing different requirements. Consequently, a requirement of minimum 8 Hz when the mass includes the quasi-permanent imposed load might be a harsher requirement than a 10 Hz requirement when the mass used is only the self-weight.

Various equations take the transverse stiffness into account. Based on Eq. (6) and (16), the influence of the transverse stiffness on f_1 is very limited in the case when only claddings or panels on upper and lower side of the beams are considered. If there are perpendicular stiffeners between the beam (e.g. noggins) this will increase the effect, and a simple equation for that ought to be established.

The allowable deflection for a point load is much smaller in Austria, Norway and Finland than in Denmark and Sweden (and most other countries).

The secondary stiffness $(EI)_b$ has a significant influence on the deflection, even if the only contribution comes from the floor deck as assumed in the Finnish Eq. (7). If there are perpendicular stiffeners between the beams, which force adjacent beams to share loads and deform together, the effective stiffness for a point load will be much higher. The deflection might decrease significantly, but no simple equation is available to estimate this. It is quite essential for timber floors to obtain such an equation, see Eq. (19) and ÖN B EN 1995-1-1 (2004):

$$b_{ef} = \frac{l}{1.1}\sqrt[4]{\frac{EI_b}{EI_l}} = \frac{b}{1.1\alpha} \tag{19}$$

The perception studies show that user satisfaction increases with $f_1/a^{0.5}$. From Eq. (6) and (7) it is seen that this ratio is largely proportional to the primary bending stiffness $(EI)_l$.

It seems likely that a requirement $f_1/a^{0.5} \geq 19$ (with f_1 [Hz] and a [mm/kN]) will ensure a very high degree of user satisfaction, or if f_1 is determined including quasi-permanent live load $f_1/a^{0.5} \geq 17$.

There is some discrepancy between the three perception studies. It may appear as if the Norwegians are more tolerant than the Canadians, whereas the Finnish are more critical. It could be due to different expectations or different methods used to obtain the user perception.

If the fundamental frequency is determined for self-weight only, the requirement $f_1/a^{0.5} \geq 19$ could perhaps represent a high class floor. In lower classes, the requirement could be reduced. The requirement, depending on the constant, can be fulfilled by e.g. $a = 0.6 - 1.0$ mm/kN and $f_1 = 15$ Hz and by $a = 0.25 - 0.45$ mm if $f_1 = 10$ Hz. This is not too far from the stricter of the Nordic and Austrian rules. It might be most attractive to aim for a fairly high natural frequency as the evidence for the criteria is somewhat doubtful at low frequencies.

5 Proposal for changes

5.1 Regarding the scope

In terms of the scope of Section 7.3 of EC5 it should be extended as follows:

1. Beyond residential floors into other end uses such as office floors.
2. With more heavy floor finishes (e.g. screeds) being used, it is important that procedures are also given for floors whose fundamental frequency is < 8 Hz.

3. For separating floors it is desirable that the vibrational performance can be determined not just for the structural deck but also for the top surface of the floating floor which of course is the walking surface.

4. For more than one-way spanning single-span rectangular rooms to cover both multiple-span floors and two-way spanning floors.

5. To cover attic truss floors.

Although it is logical for the new procedure to continue to be predominantly based on the parameters 'fundamental frequency' and 'unit point load deflection', there are benefits in also calculating floor acceleration. This is because it enables comparisons between calculated and on-site measured vibrational performance as floor acceleration is the parameter most commonly measured on site (e.g. when the floor performance is subject to a legal dispute).

With regard to calculation of fundamental frequency, the associated equations should not solely be based on joist bending properties as, in the case of engineered wood-based joists, the component of shear (or joint slip) deflection, depending on joist type, can easily be 25% of the total deflection.

With regard to calculation of unit point load deflection, the associated procedure must accurately be able to determine the proportion of the unit point load distributed to adjacent joists. Even where transverse stiffness is provided only by 22mm P5 Particleboard, broadly 50% of the point load will be distributed to adjacent joists to the one over which the point load is located.

5.2 Loading model

It has been shown that humans normally walk at frequencies between 1.5 and 2.5 Hz. A human walk can be idealised as a periodic load, with harmonic components at whole integer multiples of the walking frequency. Harmonics at frequencies higher than 4 times the walking frequency do not cause resonant response, as their energy is too low. For a more rigorous explanation of footfall induced vibration of Structures the reader should refer to the CCIP Guide (Willford and Young, 2006) and other literature (e.g. Kerr, 1998; Galbraith and Barton, 1970; Ohlsson, 1982; Wheeler, 1982).

Resonance can only occur if the first mode of the floor is less than 10 Hz (i.e. 4 x 2.5 Hz). When the first natural frequency of a floor is greater than 10 Hz (i.e. greater than the 4th harmonic of a walk) there is no resonant build-up of vibration and so the floor response is instead transient with the response dying away in-between footfalls. In this instance, the response of the floor to a person walking is similar to the effect of a series of impulses.

In this instance the same walking time histories that were used to derive the dynamic load factors (DLF – dynamic force expressed as a fraction of the walker's static weight) have been used to calculate the impulses from footfall loads. The effective impulse can be thought of as the equivalent impulse of infinitesimally short duration that induces the same peak floor response as the direct application of that footfall time history.

The mean and design impulsive load (with a 25% chance of exceedance) due to a person walking has been found to be empirically equivalent to Eq. (20) and (21) respectively (where f_n is the natural frequency of the floor and f_w is the walking frequency in Hz and the effective impulse is provided in Ns):

$$l_{eff,mean} = \frac{42 f_w^{1.43}}{f_n^{1.3}} \tag{20}$$

$$l_{eff,d} = \frac{54 f_w^{1.43}}{f_n^{1.3}} \tag{21}$$

The CCIP Guide provides a procedure for the calculation for the impulsive response. The procedure calculates the velocity time history due to a single footfall at a particular location, from which either peak or RMS velocities can be calculated. A similar approach could be adopted in EC5.

6 Conclusions

During the last 30 years, many research studies have been carried out in Europe and in Canada on the vibration performance of timber floors. Many of these studies attempt to correlate subjective assessments to actual vibration values which are either measured or calculated. Experiments have been done in laboratory and in onsite conditions. These generally give different results and it should be recommended to rely mostly in onsite testing as boundary conditions and damping properties are more realistic. In addition, experimental procedures should be further developed and unified.

The developed methods are mostly simple descriptions of the vibration problem, as realistic dynamic loads and true boundary conditions are difficult to estimate. In several studies, point load deflections have been found to correlate with the performance, however the relation is not perfect as this involves subjective testing and the method is indirect and empirical.

Modern floor structures are more complex than before, mostly due to acoustic and fire requirements set on floors, especially in multi-storey construction. Additional floor layers are needed such as floating floors and resilient ceilings. New floor types as massive CLT and timber concrete floors have developed. The vibration design methods have to develop as well, for example the damping of such floors has been found to be much higher than currently stated in EC5.

There is a need to develop an engineering code and standard provisions which apply to vibration serviceability of modern structures. Such actions are underway in both technical committees of CEN and ISO standardisation bodies. In CEN

TC/250/SC5/WG3 floor vibration design methods are discussed and a code format for the next generation EC5 is sought. In Europe national provisions differ significantly and it seems clear that several floor vibration performance levels are needed so that appropriate criteria may be adapted by the designer or the national regulator. In a longer term, unified design and testing methods of floor vibrations between different material codes should be aimed for, but so far this is not well coordinated.

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Creep related issues in EC5

Summary

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1 Motivation

Timber – similar to other building materials – is sensitive to creep. Therefore the impact on the structural assessment should not be neglected. Moreover, creep strains in the context of timber is stress component sensitive, since e.g. creep due to shear is two times higher than creep due to bending and creep of semi rigid joints is two times higher than creep for the surrounding timber matrix. Nevertheless, short-term stiffness at time zero is preserved also at time infinity, even after formation of creep displacements. This is also a basic requirement from testing standards for long term behaviour of building materials.

Today, the common practice with structural modelling of creep is diverging and unsatisfying in order to reproduce the field of displacements from creep effects. The first approach is an increase of external permanent loadings, which might imply troubles with buckling respective second order analysis. The second approach is a single compilation of fields of displacements neglecting possibly induced constraints in terms of internal forces. The third approach – also favoured by EC5 – is a fictious decrease of stiffness by a factor in terms of a final value of stiffness, assuming the possibility of superposition for load cases and again neglecting possible redistribution of internal forces due to creep.

2 Flashback to basics of structural mechanics in the context of induced strains

Everybody is familiar with induced strains respective curvature from external loading like temperature, shrinkage or swelling. For the case of structurally undetermined systems, the consequence of constraint forces is well known. Nevertheless for e.g. bending-type systems, a field of displacements could also be reproduced by the use of the curvature, back-calculated from the bending moments from the load combination called quasi permanent loading. This thereby generated field of displacements can be scaled by the creep factor and introduced as quasi “new and additional” load case, to be optionally activated at time infinity.

3 Current consideration of creep in EC5

The current implementation and recommendations for a structural consideration of creep in EC5 is rather restrictive. Numerous use-case specific concepts for subsequent structural modelling have been implemented, being either related to systems with unique or different time dependent behaviour, or the level of loading (= SLS or ULS) or the order of analysis. Today, the effect of creep is – due the lack of linear superposition – explicitly excluded from second order analysis, but implicitly compensated by the concept of reduced design values for stiffness parameters.

Possibly for sake of simplification related to hand-calculation and user-friendliness, drawbacks of the current design concept according to EC5 are a unique value for k_{def}, independent from the type of triggering stress component without any coupling of k_{def} to the level of stress. No information about time dependency of creep for timber is given in EC5, which might be helpful and necessary for building stage specific applications.

With increasing popularity of composites and building structures consisting of several building materials like concrete, steel, glass and intelligent wood based products, the restrictive case sensitivity – as proposed in the present version of EC5 – has become a challenge for both engineers in practice and producers of commercial structural engineering software. Today, only a very small scope of building applications can be handled in a mechanical correct way by the concept of EC5.

4 New engineering approach to consider creep deformations in timber design

The assumptions for the new concept and therefore changes by the new concept to the procedure in structural modelling can by summarized as follows:

- The linear elastic displacements are related to short-term stiffness parameters and must be reproducible every time. Therefore, a constant set of short-term material stiffness values should be used for the whole structural analysis, only adopted to both environmental parameters like moisture or temperature and structural effects like weakening due to crack-formation, as it is typical for concrete. Therefore, the present procedure for calculation of load specific stiffness parameters will become obsolete.

- The extent / height of the generated individual or generalized creep strains is triggered by the elastic strains, which are related to long-term loading components and only have to be scaled by individual material and specific creep factors. This procedure opens the field for a more realistic implementation of creep within structural modelling. Those results from local, member or joint specific creep effects, only induced by the quasi-permanent load combination are again to be compiled into one, quasi external load case with a partial load safety factor $\gamma_Q = 1.0$, generally to be activated at time infinity and even, if the short-term loading components are not part of the addressed load combination.

The strength of the concept – compared to the existing one in EC5 – consists of better mechanical consistency and interoperability of design concepts for different building materials, less efforts with organisation of numerous load combinations and less computational costs. Nevertheless, the increased level of structural modelling – instead of acceptance of further model uncertainty induced by simplifications – should end up in smaller values of partial safety factors.

Concerning effects not considered by the new approach, the present approach is still an assessment at fixed instants with respect to the building stages, not taking into account transient processes, which usually are beyond the scope of commercial engineering software. The change of permanent loading by transient changing internal constraint forces still must be balanced by some type of more generalized values for the creep factors.

5 Complete implementation in structural engineering software

The reproduction of system displacements starts with the identification of the therefore responsible strains / curvatures and ends with their application to the structural system as quasi-separate load-case at time infinity. Nevertheless, the scope of strains / curvatures will depend on the theory for structural modelling (with or without shear displacements) as well and the type of the structural element (homogeneous and hybrid cross sections. . .) itself. This procedure has to enabled for all types of structural elements like beams, planes or volumes also including concentrated joints or linearly allocated semi rigid connection (= line hinges). The concept can only be applied to arbitrary structures in daily practice, if all aspects mentioned above have correctly been implemented in the software.

6 Consequences for the next EC5

Facing the potential of simplification by the new concept of improved structural modelling of creep, the corresponding clauses in the EC5 could significantly be consolidated in terms of less text and restrictions. Due to the drop of the case sensitivity, the more general-purpose approach is also in line with facilities of modern engineering software. The consequences will be mechanical transparency and an increase of confidence into the results.

7 Conclusions

By mechanical correct and software-efficient structural modelling of creep phenomena, both design standards and the efforts in daily engineering practice could significantly be reduced. Furthermore, more realistic stress distributions due to improved structural modelling should end up in smaller partial safety factors in the context of probabilistic assessments and code calibration.

Although some features for innovative structural modelling in commercial engineering software already support this new approach, only the whole package of faculties for members and joints ensures a consistent and reliable stress analysis independent from the complexity of the structural system.

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Stability related issues in EC5

Summary

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1 Motivation

In general, the content of design standards should be driven by the requirements of building practice. Especially the issue of stability is affected by numerous parameters and boundary conditions. The first domain of structural variety is to be fixed on the level of the type of the structural system itself (= beam and/or plane elements, 2D-plane or 3D-spatial arrangement), on the level of beams (= straight or curved beam axis, beam or wall like shape...) and on the level of cross sections (= arbitrary shaped, single or multiple, homogeneous or multi-layered, constant or variable height).

With respect to the material characteristic of timber and wood-based products, the following items could have an impact on local or overall stability of structures. The stiffness parameters due to growth irregularities are scattering even with consequences for the distribution over the cross section. The time dependent behaviour like creep increases the field of displacements with impact on the equilibrium of internal and external forces. The impact of moisture content becomes more complicated with large cross sections due to the only moisture affected boundary zone and homogenisation step for a representative value of e.g. beam bending stiffness. Also crack formation could change the characterisation on the level of the cross section like torsional stiffness respective shear stress distribution.

Further general-purpose aspects, which are not directly linked to timber structures, should also be taken into account: The stability of the whole process of assessment should minimize damage related to human error. The result of the design process should meet the demand for efficient use of resources like raw material and time. Flexibility during the design process with respect to composites and mixture of materials is necessary especially during the stage of optimization. Interoperability of design concepts among different building materials helps to reduce barriers for engineers, only being familiar with the design of steel or concrete structures.

Today, two strategies respective set of tools for handling issues of stability are available. Hand calculation (= use of hand-held calculator, Excel, Mathcad...) is still important at the stage of early design or plausibility check of complex FEM-simulations. Nevertheless, the use of structural engineering software, either on the level of component-based or general-purpose software packages is already and will remain the dominating use case in daily practice.

2 Current consideration of stability in EC5

By definition, the job of real and/or fictious imperfection is either to handle respective balance varying stiffness parameters or shift a structural system with perfect geometry and a reduced set of internal forces to an imperfect system with a full set of internal forces for the purpose of design. Stiffness itself has impact on both distribution of internal forces and system displacements. The wording of stability should better be substituted by the formulation *sensitivity* of displacements to the equilibrium of external and internal *forces*. Depending on the maximum amount of displacement components with respect to the size of the system or cross section itself, second or third order analysis should be applied with different linearization of kinematic relationships.

In EC5, issues of stability unfortunately are allocated in different chapters of the code and separated to different items. The equivalent member method (EMM), originally designed for systems only consisting of members without any connections, only provides results for buckling due to bending or lateral torsional effects. Neither the corresponding internal forces at locations with joints nor realistic displacements are available for the further design process. Alternatively, the design according to second order analysis (SOA) is capable to consistently handle system nonlinearities and both types of structural system like beams and plane elements. However, the assessment for buckling of planar elements by too simplified geometrical constraints is too inflexible for reliable and economic design. This is also true for the calculation of design loads for the verification of the bracing system itself on the basis of second order analysis. It should be a matter of design strategy, if the minimum stiffness of bracing structures is below or above the calculated value. Finally in the section related to quality management during production and assembly, some limits for geometrical tolerances during the stage of assembly on site are given, which should be completed by only material specific values for imperfections to be taken into account as special type of external loading in the context of second order analysis.

3 Approved engineering approach to consider stability in timber design

Due to the character of SOA as general purpose method, each structural assessment should be consequently performed according to this approach without any costly preliminary check for the option of first order analysis. Areas with displacement sensitive equilibrium conditions are varying with load combinations. A working structural system is already a guarantee (= cost-free quick check) for a design solution below the first (= lowest) buckling load. The shapes at different buckling levels reveal weak points of a structure, being a valuable guideline for provisions of balancing the probability of failure among all members and joints of a structure. It could probably be desirable (= design principle), that the lowest buckling mode should rather be related to global (= system mode) than to a single member, which in addition could possibly be classified as key member for the whole structure. The structural assessment should be performed with only one set of short-term stiffness parameters on the level of mean values for both members and possibly nonlinear slip curves for joints, but adopted to environmental parameters. The variability of displacements due to material specific variation of stiffness parameters and different levels of execution classes should only be handled by customized setting of imperfections. The course of imperfections should qualitatively be based on the course of the lower buckling shapes with only intermediate adjustment by sign to the dominating field of displacements, specific for each load combination. The already, according to SOA calculated field of creep related displacements from the quasi permanent load combination should be converted into induced generalized strains respective curvature, compiled within a "new and additional" load-case and activated for structural assessments at time infinity. At all times, a consistent set of internal forces in line with realistic global system or relative joint displacements is available for a correct design of connections.

4 Consequences for the next EC5

The focus in preparatory discussions and finally in EC5 should be set on general purpose methods in line with the capabilities of modern engineering software. A set of principles respective criteria should guarantee a minimal level of consistent and therefore reliable structural modelling. For simple structural systems, hand calculation might still be reasonable. Nevertheless, most structural systems require the use of multipurpose engineering software, which rather should be based on consistent mechanical concepts than on an accumulation of restrictive and case sensitive analytical formula. Furthermore, the demand for harmonisation of procedures and boundary conditions for the design of composites or structures with different building materials will help to reduce barriers for engineers, who would like to become familiar with all relevant building materials.

Stability issues should be consolidated into one chapter, only containing regulations for second and alternatively third order analysis. The regulations for EMM - as alternative and simplified method - for a restricted set of applications should therefore be shifted to the appendix and no longer be expanded by still more complicated formula for further special cases of applications, which could better and simpler be handled by SOA.

5 Conclusions

Simplifications should only be accepted, if no further uncertainties from too simplified structural modelling are introduced again and the field of applications is representative not only for a small set of academic structures. By the use of already available and still further to be implemented features within commercial engineering software, the assessment of structures with at least partially sensibility to displacements for the equilibrium of forces, becomes more accurate and well structured.

Nevertheless, code calibration in the future for such structures should use imperfections as calibration parameters instead of stiffness characteristics and design standards have to provide at least the material specific contribution to the calculation value of imperfections.

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Evaluation of the failure behaviour and the reliability of timber connections with multiple dowel-type fasteners in a row

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Abstract

Connections are amongst the most important details in timber structures in order to build large structures. Connections with metal dowel-type fasteners are commonly used for the connections of single timber members. The load-carrying capacity of such connections is limited by the different failure modes that occur in the fastener itself and in the surrounding timber. The failure behaviour can be described as brittle or ductile, depending on the failure mode and the associated deformation capacity of the connection. In case of brittle failure modes with only small deformation capacity premature failure can occur before the entire connection with multiple fastener has reached its ultimate capacity. Within this contribution the failure behaviour and load-carrying capacity of timber connections with multiple fasteners in a row are evaluated. The implementation of design approaches in Eurocode 5 is critically assessed. It can be concluded that only ductile failure modes with sufficient plastic deformation of the fasteners allow for an optimal load-carrying capacity. A high reliability can be achieved if failure modes with a low variability occur, i.e. failure modes with plastic failure of the steel. In other cases with brittle failure modes the reduced load-carrying capacity due to premature failure should be accounted for.

1 Introduction

Connections enable to build larger timber structures from individual timber elements. For the estimation of the structural behaviour of timber structures the behaviour of the timber elements as well as the structural performance of the connections is of importance. The standardisation of the design of connections in the serviceability and ultimate limit states shows considerable deficits compared to the design regulations of timber structural components. A possible reason for this lack in regulation of the performance of connections can be found in the large variety of connection types and configurations. This large variety precludes the reliable quantification and verification of statistical and mechanical models by means of testing samples with large numbers specimens.

Glued-connections, dowelled, bolted, nailed or stapled connections, connections with screws or glued-in rods are amongst the connections types most commonly used in modern timber structures.

Connections with dowel type fasteners typically consist of steel fasteners and the surrounding timber. If the fasteners govern the failure of the connection the lower variability of the steel can contribute to a much smaller variability of the load-carrying capacity and, hence, lead to lower safety factors when evaluating the reliability (Kohler, 2005). This fact is currently considered in the European design code for timber structures EN 1995 (Eurocode 5, EC5 (EN 1995-1-1, 2004)) by a increase of the load-carrying capacity of 15% (Jockwer et al., 2017). Not only the reduced variability but also ductile failure behaviour can offer a high potential for further enhancement of the currently applied design procedures. The ductility in the connection allows for a potential redistribution of loads in the structures, which can create robustness (Dietsch, 2011). Different geometrical requirements (sufficient timber thickness, spacing, end- and edge-distances) have to be met in order to achieve the desired level of ductility in the connection. Especially for connections with multiple fasteners these requirements for achieving ductility can be much more challenging compared to single fastener connections.

2 Load-carrying capacity of connections

The load-carrying capacity of laterally loaded connections with dowels type fasteners can be described by the so called European Yield model (EYM). This model distinguishes different failure modes with either embedment of the fasteners, plastic hinges of the fastener or a combination of both. The model was described by Johansen (1949) and further developed by Meyer (1957), who included the plastic section modulus of the fastener. The embedment strength $f_{\mathrm{h},i}$ of the timber members and the yield moment M_{y} of the fastener are the material parameters used in the EYM. The model can be used to estimate the load-carrying capacity of a single fastener based on the thickness t_i of the timber members and the diameter d of the fastener. In connection with multiple fasteners additional effects have to be accounted for. An effective number of fasteners $n_{\mathrm{ef}} \leq n$ is used in EC5 to account for effects such as unequal distribution of load between the fasteners or the accumulation of splitting forces.

Besides the fastener failure modes described by the EYM also the surrounding timber may cause failure of the connection. The timber failure is often characterised by brittle failure in shear or tension perpendicular to grain offering hardly any deformation capacity of the connection. Different timber failure modes in connections are discussed by Cabrero and Yurrita (2018), in Appendix A of EC5 a block shear failure mode for steel-timber connections is described. Timber failure modes can be described in dependency of the spacing between fasteners a_1, end-grain distance a_3, edge distances a_4, member thickness t and in dependency of the shear strength f_{v}, tension perpendicular to grain strength $f_{\mathrm{t,90}}$, stiffness properties (E_0 and G_{v}) and fracture energies in tension perpendicular to grain $G_{\mathrm{f,I}}$ and shear $G_{\mathrm{f,II}}$. Models for describing timber failure modes were proposed e.g. by Jorissen (1998) or Schmid et al. (2002).

Werner (1993) studied the impact of varying material properties on the load-carrying capacity of connections. Köhler (2007) gives information on the distribution characteristics of the material properties related to the load-carrying capacity of connections.

The load-carrying capacity determined in experiments is approximately 20% higher compared to the estimated load-carrying capacity according to EYM as discussed e.g. by Larsen (1974). This differences was explained by Meyer (1957) by additional friction acting between the timber elements. Svensson and Munch-Andersen (2016) proposed a model accounting for friction between the fastener and the timber in dependency of the deformation of the fastener. The model uncertainty of different mechanical and empirical models compared to experimental results determined by Jorissen (1998) was discussed by Köhler (2007). When accounting for a model uncertainty with a certain bias and variation Köhler (2007) was able to minimize the missmatch of experimental and estimated results with a resulting the coefficient of variation of $CoV \approx 15\%$.

3 Failure behaviour and failure consequences of connections

In order to allow for a potential redistribution of loads within a connection and in a structure it is commonly aimed at achieving sufficient deformation capacity of the connection. Different design codes, such as the Swiss SIA 265 (2012) or the former German standard DIN 1052 (2008), set the failure mode with plastic deformation of the fasteners as the basis for the design of connections. This failure mode with plastic deformation of the fasteners sets the upper limit of the load-carrying capacity of a connection with dowel type fasteners. According to SIA 265 (2012) a ductility ratio $D_s > 3$ is associated to this failure mode. In EC5 all different failure modes with different deformation capacity are treated equally. The consequences of failure of connections showing a brittle failure mode should be evaluated more in detail.

A connection with multiple fasteners can be modelled as a system of resistance elements in a serial or parallel assembly. The brittle failure behaviour of the fastners/the connection can be represented by a serial assembly of these elements. The failure of the weakest of these elements sets the lower limit of the load-carrying capacity of the entire system. With increasing number of fasteners in the connection, i.e. with increasing number of brittle elements, the relative load-carrying capacity decreases. This behaviour was described by Weibull (1939) as the weakest link theory. If the fasteners show a ductile failure behaviour, a parallel assembly of elements can be used. The load-carrying capacity can be described as the sum of all resistances of the single elements. This allows for the activation of the full number of fasteners in the connection.

In experiments on connections with multiple fasteners it can be observed that the load-carrying capacity of the entire connection is smaller than the sum of the load-carrying capacity determined in tests on connections with single fasteners. The reduced load-carrying capacity in connection with multiple fastener results amongst others from the unequal distribution of forces between the fasteners in the connection. Studies on the distribution of forces between different fasteners in a connections were performed e.g. by Volkersen (1938). Blaß (1991) evaluated the distribution of forces in connections with large numbers of nails in an experimental campaign.

The effective number of fasteners, that is smaller or equal to the total number of fasteners, as it is accounted for in EC5 is based on the studies by Jorissen (1998). Based on a large number of tests on dowelled connections with various configuration and different numbers of fasteners in a row, Jorissen determined the reduction factor in dependency of the fastener spacing. For large spacing premature splitting can be avoided and a perfect load redistribution between the fasteners is achieved.

Theoretical studies performed by Jockwer et al. (2018) show that the maximum load-carrying capacity of the entire connection requires that all fasteners are able to reach a ductile failure mode. Brittle failure modes result in premature failure at lower load levels. Further it was observed that even for small spacings, brittle failures occur also for large side member thickness.

4 Discussion and Conclusions

The following conclusions can be drawn from the evaluation of the failure behaviour of connections with multiple fasteners:

- The larger deformation capacity of connections experiencing ductile failure modes allow for a redistribution of forces within the connection and within the structure.
- In connection with ductile failure modes the ultimate failure is caused by excessive deformations, which can be associated to lower failure consequences.
- In connections experiencing a brittle failure, no redistribution of forces is possible and weakest link effect can be observed.
- Brittle failure modes should be avoided whenever possible.
- Ductile failure modes are essential to achieve high load-carrying capacity, low variability, high reliability and sufficient robustness especially in connections with multiple fasteners.

- Sufficient spacing and end- and edge-distances as well as sufficiently large member thickness in order to achieve a failure mode with plastic hinges in the fastener are prerequisites to reach ductile failure modes.
- Reinforcement of connections by means of self-tapping screws can mitigate the risk of brittle timber failure modes due to splitting.
- Also for other connections types, such as glued-in rods, axially loaded screws or finger joint connections, the impact of the failure behaviour on the variability of the load-carrying capacity should be accounted for when evaluating the reliability in design as discussed by Jockwer et al. (2017).

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Basis of design principles – application to CLT

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1 Introduction

In the current version of Eurocode 5 (EN 1995-1-1, 2004) the design of cross laminated timber (CLT) structures is not regulated. Key elements that are missing for the implementation into the Eurocode are:

- Standardized production procedures: to guarantee standardized and reliable products
- Testing and evaluation standards for CLT: to achieve reliable characteristics of the product properties
- Modification factor k_{mod}: to consider the strength reduction due to duration of load effects
- Partial safety factor γ_{m}: to ensure an acceptable level of structural safety
- Regulations for the design and execution: to address product specific aspects

Due to the growing importance of CLT in the timber sector, it is one of the major goals of COST Action FP1402 as well as for the second generation of the Eurocodes to implement regulations for the design and execution of CLT in structures. This chapter presents a comprehensive summary of the research performed on this topic during the COST Action FP1402. For more information it is referred to Fink et al. (2018) and Köhler et al. (2016), where all necessary results are presented.

When modelling the material properties of structural timber several issues related to strength grading, size effects and duration of load effects have to be taken into account (see e.g. Köhler, 2006). For engineered timber products, such as CLT, the modelling is even more complex. For such products also the joint behavior of the assembled timber boards and the production process (e.g. layup, finger joint quality) have to be represented. Accordingly, the variability of the product properties for the individual failure scenarios as well as the corresponding strength reducing effects (size effect, duration of load effects, etc.) have to be identified under consideration of e.g. the existing regulations for production procedures, quality control and testing standards.

2 Variability of the product properties

In order to calibrate the partial safety factors γ_m the variability of the product has to be known, here, it has to be considered that the variability of individual batches are not representative for the basic population, in particular for CLT where the production has not been standardized so far.

In Fink et al. (2018) the variation between and within batches is investigated for the out-of-plane bending strength and the out-of-plane rolling shear strength. Both investigations are performed on six batches.

The variation of the out-of-plane bending strength of the individual batches is identified to be between $COV = 0.046$ and $COV = 0.152$. Between the batches a large variability is observed; e.g. the variation of the mean values of the batches $COV \approx 0.20$. Accordingly, the variability of all samples is significantly larger compared to those from individual batches: $COV = 0.208$.

The variability of the out-of-plane rolling shear strength of the individual batches seems to be slightly smaller. However, the variability between the batches are similar.

For the implementation in the Eurocode the target reliability has to be met for all individual failure scenarios (tension, compression, perpendicular and parallel to grain, shear, rolling shear, etc.). It has to be considered that the variabilities of the product properties of the different failure scenarios are different (see e.g. JCSS, 2006; Köhler, 2006). Examples for structural timber and the influence on the structural safety and thus the choice of the partial safety factors γ_m was already investigated by Kohler and Fink (2015a,b). There a significant over- and underestimation is indicated for different failure scenarios, due to regulations of constant γ_m values for all failure scenarios.

3 Modification factor k_{mod}

The strength of timber and thus of CLT is influenced by the intensity and the duration of the applied stresses. This effect is referred to as the duration of load (DOL or static fatigue) effect. For CLT experimental investigations of the DOL effect are limited, however, it seems appropriate to assume a rather similar behavior as for glued laminated timber, at least for bending and tension. For other failure modes where also the long-term stress-strain behavior of the glue line is relevant this assumption necessitates corresponding investigations.

4 Testing and evaluation standards

For CLT a test standard is missing, however, it is common to perform experimental investigations for CLT in accordance to EN 408 (2003). The investigations described in Fink et al. (2018) show that a significant amount of the specimens differed from the target ones. Accordingly, the characteristics of the product properties that are estimated from such experimental investigations are associated to larger uncertainties; this uncertainties have to be considered in the design procedure. In order to use the full potential of CLT it is important that standardized test and evaluation procedures will be developed.

5 Recommendation

In Fink et al. (2018) a reliability based code calibration was performed. As an outcome, for CLT the same partial safety factor as recommended for glued laminated timber $\gamma_\mathrm{m} = 1.25$ seems to be appropriate. The result is significantly influenced by the large variability of the material properties between the batches combined with missing test standards. Accordingly, a smaller partial safety factor (e.g. $\gamma_\mathrm{m} = 1.20$) seems to be realistic in case that appropriate test and evaluation standard and more standardized productions procedures of CLT will be developed.

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Guideline for data analysis

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Summary

The experimental investigation of uncertain phenomena is addressed in this subsection. In timber engineering due to the large natural variability of the material the use of experimental investigations and the corresponding consistent treatment of uncertainties is particularly important. In order to use the results for further investigations and discussion it is essential that the data analysis is performed in an appropriate way. The main target of the presented chapter is to summarize most common tools and methods as well as relevant aspects for efficient data analysis. Selected methods are illustrated on small examples at the end of the chapter. This chapter only provides a compilation of selected methods. For more detailed information about the ones presented it is referred to standard literature concerning applied statistics and uncertainty representation in engineering (e.g. Benjamin and Cornell, 1970; Thoft-Christensen and Baker, 1982; Melchers, 1999; Ditlevsen and Madsen, 1996; Madsen et al., 2006; Faber, 2009).

1 Introduction

Evidence gained from experimental data is a strong argument in scientific reasoning as it allows for insight about the regularity and variability of a studied phenomena. In structural engineering the analysis of experimental data serves the establishment of the representation of phenomena relevant to structural engineering design, i.e. the corresponding experiments and observations relate to the strength and stiffness related properties of structural materials and material assemblies, load phenomena, deterioration phenomena, structural dimensions, etc. The phenomena are in general associated with significant uncertainties which implies that they are best represented by random processes and random variables.

Timber is by nature a very inhomogeneous material. The load bearing performance of timber material depend on the specific wood species, the geographical location where the wood has been grown and furthermore on the local growing conditions of every single part of a tree. Timber is an orthotropic material, i.e. it consists of “high strength” fibres/grains which are predominantly orientated along the longitudinal axis of a timber log/ tree and packed together within a “low strength” matrix. After a log is sawn into pieces of structural timber, irregularities in regard to e.g. grain direction, knots, fissures, become (in addition to the orthotropic characteristics mentioned above) highly decisive for the load bearing capacity. As a natural material structural timber cannot be designed or produced by means of some recipe but may be ensured to fulfill given requirements only by quality control procedures implemented during the production process.

When performing experimental investigations in timber engineering usually mechanical properties, such as the load bearing capacity of the tested structural component are measured. Material properties are thus defined on an element level, i.e. material properties are defined as the load bearing capacity of timber material specimens of defined size and conditioning and assessed in accordance with an agreed testing procedure (ISO 8375). E.g. the tensile strength of timber is not an ultimate stress property of the timber material; it is rather the tensile capacity of the test specimen, divided by the cross section. This is of particular relevance for structural timber where the failure usually occurs in areas with significant growth irregularities such as knots. This is also true for other loading modes, that are in general referred to as "material properties" (see Part II – Timber specific code calibration). Similar observations can be made for timber based engineered materials as e.g. glued laminated timber and cross laminated timber.

1.1 Population and Sample

The above sketched particularities of structural timber material necessitate that for planning of experiments and subsequent for the analysis of the results is of particular importance to define the population.

A formally planned experiment that has the objective to gain insight about a phenomenon of interest, defines prior a population as a set of similar items or events that carries information about that phenomenon. For statistical inference a sample from the population is chosen and in order to identify a sample that represents the population, random sampling is the most straightforward sampling method.

Example 1:
Consider the objective of characterizing the thermal conductivity of European spruce timber. The population is obviously all available European spruce timber and a representative sample would be established by the random collection of specimen throughout the European timber market. Although this sounds pretty straightforward, a strict random sampling from the entire population is hardly feasible. In a practical project sampling from a sub-population (representing e.g. a reduced geographical area) would be defended by the hypothesis that the geographic variability within the population is negligible. Note that such a hypothesis principally requires proof that is seldom provided in practical projects. Another very important aspect

for the definition of population and the identification of the corresponding representative sample is how the phenomena of interest is defined and measured. Thermal conductivity is a physical property, however, in the context of structural timber, the measured magnitude is expected to be sensitive to the test assembly. The definition of the population, thus, also includes the condition on the (ideally standardized) test procedure.

Example 2:
The aim is to characterize the lateral bending capacity of cross laminated timber (CLT) to be represented in the European design code. The population is correspondingly all the CLT that is available on the European market. It is easy to imagine that the gross supply of European CLT is rather in-homogeneous, i.e. different producers, different lamination dimensions and set ups, different production conditions (glue, gluing pressure, etc.) induce a pronounced separation of the population in sub-populations. The variability of the lateral bending capacity between these sub-populations might be in the same magnitude (or even higher) than the variability of the lateral bending capacity within a sub-population (see e.g. Fink et al., 2018a). A sample has to represent both these types of variability, which is in practice a quite challenging task. And as it is clear from the introductory text, the lateral bending capacity of CLT has to be defined by a standardized test set up, specifying dimension, load configuration and duration, moisture content and temperature.

It can be concluded that the representation of uncertain phenomena by random variables should always be related to a set of populations that is meaningful and consistent for the problem at hand. The description and modelling of these random variables, as described in the remainder of this chapter, should correspond to this set. A reliability analysis based on these random variables is only valid for the considered set of populations. The basis for the definition of a population is the physical background of the quantity. Factors which define a population are in general the nature and the origin of the random quantity (e.g. strength, modules of elasticity), the spatial characteristics (e.g. size of the specimen, geographical origin of the considered material, regional wind speed characteristics) and temporal conditions (e.g. duration of exposure). The choice of specifications which define a population may depend on the objective of the analysis, the nature of the available data and the amount of resources which can be afforded. A population with a unique set of specifications is referred to as elementary population; a population in which specification parameters vary is referred to as a composite population. The set of measurements associated with a certain population is referred to as an elementary or composite sample respectively. A sampling procedure may be representative or artificial. Representative samples or representative realizations of random variables are obtained through random sampling. Artificial means that no direct relation exist between the statistical properties of the sample and the statistical properties of the population. An artificial sample is e.g. when only weak specimen are selected for testing by engineering judgment or proof loading. Observations on a representative sample may be undertaken according to a standardized test procedure. Hereby the test standard specifies partly the population: If a sample of timber specimens is tested and all spatial and temporary conditions are specified, the statistical properties of the sample are assumed to be the same as the statistical properties of the population. The statistical properties of the sample are described by a suitable probability distribution function. The physical characteristic of the random variable determines the possible type of distribution function.

1.2 Purpose of the investigation

Different purpose for experimental investigations exist and the selected testing method as well as the appropriated methods for the analysis of the results varies, accordingly. In the following two different scenarios are introduced briefly in order to illustrate the differences:

Quality control:
The majority of the experimental investigations are performed within the framework of quality control, e.g. to guarantee the material (strength) properties of a production line. For quality control it is most important that the experimental investigations follow standardized procedures in order to identify changes (reduction) of the quality. Of course influencing parameter such as the dimensions of the specimens have to be documented for the analysis. The data analysis has to be performed according to a clear defined procedure as well. Here the purpose of the tests are important, namely to ensure a certain quality. In this respect statistical uncertainties due to a low number of tested samples, have to be considered in a way that quality is ensured with an acceptable probability.

Input for mechanical or numerical models:
Often experimental investigations are performed in order gain input parameters for mechanical or numerical models. Obviously, the experimental investigations have to be planned and performed in order to find the parameters of interest (e.g. the compressive strength of a short timber board section) and thus they might not follow the procedure defined in testing standards. For the data analysis it is important to find the most accurate results, and the corresponding probabilistic models are of predictive nature.

2 Data characterization

2.1 Classification of information

In order to perform data analysis it is essential to know the type of the information gain during the experimental investigation. In general it can be distinguished between *equality type information* and *inequality type information* as well as between *direct information* and *indirect information*. The differences between the different types of information are explained on the bending strength of a structural component:

- Loading until failure: the bending strength is recorded and known (equality type information, direct information)
- Loading to a specific stress level without failure (e.g. proof loading): lower limit of the bending strength is known (inequality type information, direct information)
- Measurement of the density: the bending strength can be estimated e.g. by a regression model (equality type information, indirect information)
- Visual inspection: a lower or upper limit of the bending strength can be estimated (inequality type information, indirect information)

For more information and examples regarding the classification of different types of information in timber engineering it is referred to e.g. Köhler (2006); Fink and Kohler (2015)

2.2 Basic variables of the resistance

An elegant method to describe data is by using distribution functions. In the following paragraphs, distribution functions that are most relevant for timber engineering are introduced (Table 1). For an overview of other distribution functions see e.g. Benjamin and Cornell (1970) and Hahn and Shapiro (1967).

Normal distribution:
The Normal distribution is widely used in many applications. The sum of many independent random values are Normal distributed (central limit theorem) (e.g. Benjamin and Cornell (1970)). The range of the Normal distribution is $-\infty < x < \infty$, which gives always a finite probability of negative values. This is contradictory when modelling the resistance of materials, which can never be negative.

Lognormal distribution:
A random variable is Lognormal distributed if the logarithm of its realizations is Normal distributed. The Lognormal distribution is often used for the representation of non-negative strength and stiffness related properties in structural engineering.

Exponential distribution:
The interval between two sequential events that follow a Poisson process is Exponential distributed. One example is the distance between two adjacent knot clusters within the trunk of a Norway spruce tree. The Exponential distribution forms also the basis for extremal analysis as tails of many distributions tend to exponential shape.

Gamma distribution:
The Gamma distribution can be seen as a generalized version of the Exponential distribution. It describes the interval to the n^{th} event of a Poisson process. The distribution is generalized when k is not an integer.

Weibull distribution:
The Weibull distribution can be used to describe extreme minima and maxima. In engineering applications it is common to use the Weibull distribution to describe the strength of a structural component, especially for brittle materials (weakest link theory).

2.3 Uncertainties in reliability assessment

Decision problems are in general subjected to uncertainties. It is generally to be distinguished between aleatory uncertainties (inherent natural variability) and epistemic uncertainties (model and statistical uncertainties), see e.g. Melchers (1999), Faber (2009).

Inherent natural variability:
Uncertainties according to the inherent natural variability results from the randomness of a phenomenon. An example is the future realization of the applied wind or snow load on a construction.

Model uncertainties:
Uncertainties that are associated with the inaccuracy of our physical or mathematical models.

Statistical uncertainties:
The statistical evaluation of test results is connected to statistical uncertainties. They can be reduced through an increased number of specimens.

In general all three types of uncertainties have to be consistently represented by the probabilistic model. An example is an empirical model to predict the tensile strength of timber boards (f_t) based on the tKAR-value: $f_t = \beta_0 + \beta_1 \cdot \text{tKAR}$. Here the model uncertainties are a result of the inappropriate model, they can be reduced through an improvement of the model; e.g. additional indicators. Furthermore, both empirical parameters β_i might be connected with statistical uncertainties since they are estimated based on a limited number of data. However, even with a model that is physically and mathematically 'perfect' (i.e. no model and statistical uncertainties), some uncertainties will remain according to the inherent natural variability of timber.

3 Methods for data analysis

3.1 Probability plots

In order to describe a set of data it is necessary to know the associated distribution function. One quantitative approach in order to find such distribution function is by using probability plots. Selected probability plots are illustrated in Example II (Chapter 5.2). Here it also has to be considered that the selected distribution function should not violate physical boundary conditions, such as the lower limit of strength properties $f \geq 0$. Accordingly a Normal distributions is not the most appropriate

Table 1: Compilation of selected distribution functions

	Density function / Distribution function	Range / Par. restriction	Mean value / Standard deviation
Normal distribution	$f(x)=\frac{1}{\sigma\sqrt{2\pi}}\exp\Big(-\frac{1}{2}\Big(\frac{x-\mu}{\sigma}\Big)^2\Big)$ $f(x)=\frac{1}{\sigma}\phi\Big(\frac{x-\mu}{\sigma}\Big)$	$-\infty<x<\infty$	μ
	$F(x)=\frac{1}{\sigma\sqrt{2\pi}}\int\limits_{-\infty}^{x}\exp\Big(-\frac{1}{2}\Big(\frac{x-\mu}{\sigma}\Big)^2\Big)$ $F(x)=\Phi\Big(\frac{x-\mu}{\sigma}\Big)$	$\sigma>0$	σ
Lognormal distribution (shifted)	$f(x)=\frac{1}{(x-\varepsilon)\zeta}\phi\Big(\frac{ln(x-\varepsilon)-\lambda}{\zeta}\Big)$	$\varepsilon<x<\infty$	$\mu=\varepsilon+\exp\Big(\lambda+\frac{\zeta^2}{2}\Big)$
	$F(x)=\Phi\Big(\frac{ln(x-\varepsilon)-\lambda}{\zeta}\Big)$	$\zeta>0$	$\sigma=\exp\Big(\lambda+\frac{\zeta^2}{2}\Big)\cdot\sqrt{\exp(\zeta^2)-1}$
Exponential distribution (shifted)	$f(x)=\lambda\exp\big(-\lambda(x-\varepsilon)\big)$	$\varepsilon\leq x<\infty$	$\mu=\varepsilon+\frac{1}{\lambda}$
	$F(x)=1-\exp\big(-\lambda(x-\varepsilon)\big)$	$\lambda>0$	$\sigma=\frac{1}{\lambda}$
Gamma distribution	$f(x)=\frac{\nu(\nu x)^{k-1}}{\Gamma(k)}\exp(-\nu x)$ $F(x)=\frac{\Gamma(\nu x,k)}{\Gamma(k)}$	$0\leq x<\infty$	$\mu=\frac{k}{\nu}$
	$\Gamma(\nu x,k)=\int\limits_{0}^{\nu x}t^{k-1}\exp(-t)dt$ $\Gamma(k)=\int\limits_{0}^{\infty}t^{k-1}\exp(-t)dt$	$\nu>0$ $k>0$	$\sigma=\frac{\sqrt{k}}{\nu}$
2p - Weibull distribution	$f(x)=\frac{p}{b}\Big(\frac{x}{b}\Big)^{p-1}\exp\Big(-\Big(\frac{x}{b}\Big)^p\Big)$	$0<x<\infty$	$\mu=b\Gamma\Big(1+\frac{1}{p}\Big)$
	$F(x)=1-\exp\Big(-\Big(\frac{x}{b}\Big)^p\Big)$	$p>0$ $b>0$	$\sigma=b\cdot\sqrt{\Gamma\Big(1+\frac{2}{p}\Big)-\Gamma^2\Big(1+\frac{1}{p}\Big)}$

distribution function for properties that can only attain positive values. However, for most applications used in timber engineering the distribution functions are already identified and can be selected according to the associated literature e.g. JCSS (2006).

3.2 Maximum Likelihood Method (MLM)

The basic principle of the Maximum Likelihood Method (MLM) is to find the parameters of the chosen distribution function in order to represent the data sample most likely. The parameters of the distribution function are estimated by solving the optimization problem given in Eq. (1), where $L(\boldsymbol{\theta}|\hat{\mathbf{x}})$ is the likelihood function, $L_i(\boldsymbol{\theta}|\hat{x}_i)$ is the likelihood of an individual realization i and $\boldsymbol{\theta}$ are the parameter of the distribution function.

$$L(\boldsymbol{\theta}|\hat{\mathbf{x}}) = \prod_{i=0}^{n} L_i(\boldsymbol{\theta}|\hat{x}_i) \qquad \min_{\boldsymbol{\theta}}\left(-L(\boldsymbol{\theta}|\hat{\mathbf{x}})\right) \tag{1}$$

The likelihood $L_i(\boldsymbol{\theta}|\hat{x}_i)$ of an individual realization i can be calculated for equality type information and inequality type information (ofter denotes as censored data) by using Eq. (2). Here f_X is the density function of the random variable X, and $\hat{x}$ are the measured values of the data sample.

$$L_i(\boldsymbol{\theta}|\hat{x}_i) = \begin{cases} f_X(\hat{x}_i|\boldsymbol{\theta}) & \text{for non censored data} \\ 1 - F_X(\hat{x}_i|\boldsymbol{\theta}) & \text{for censored data} \end{cases} \tag{2}$$

The uncertainties of the MLM estimators can be expressed with covariance matrix $\mathbf{C}_{\boldsymbol{\Theta\Theta}}$, where the diagonals are the variances of the estimated distribution parameters and the other elements are the covariances between the parameters. The covariance matrix $\mathbf{C}_{\boldsymbol{\Theta\Theta}}$ is defined as the inverse of the Fisher information matrix $\mathbf{H}$. The components of $\mathbf{H}$ are determined by the second order partial derivatives of the log-likelihood function; see e.g. Faber (2012).

$$\mathbf{C}_{\boldsymbol{\Theta\Theta}} = \mathbf{H}^{-1} \qquad H_{ij} = -\frac{\partial^2 l(\boldsymbol{\theta}|\hat{\boldsymbol{x}})}{\partial\theta_i \partial\theta_j}\Big|_{\boldsymbol{\theta}=\boldsymbol{\theta}^*} \tag{3}$$

3.3 Regression analysis

Regression analysis is a wide spread tool to identify the (semi-) empirical relation between different parameter. In engineering linear regression models are most common and accordingly this chapter is limited to them. However, other regression models are possible as well. The general form of a linear regression model is presented in Eq. (4), where Y is the parameter we want to predict (e.g. the tensile strength), X_i are the input variables (e.g. the density), β_i are the regression parameter and ε is the standard Normal distributed error term with σ_ε.

$$Y = \beta_0 1 + \beta_1 X_1 + \beta_2 X_2 + ... + \varepsilon \qquad \text{or} \qquad \boldsymbol{y} = \boldsymbol{X\beta} + \boldsymbol{\varepsilon} \tag{4}$$

The parameter of the regression model are usually estimated using the least square approach (or MLM), using Eq. (5), where n is the number of data and k is the number of regression coefficients.

$$\boldsymbol{\beta} = (\hat{\boldsymbol{X}}^T \hat{\boldsymbol{X}})^{-1} \hat{\boldsymbol{X}}^T \hat{\boldsymbol{y}} \qquad \sigma_\varepsilon^2 = \sqrt{\frac{\sum_{i=1}^n \varepsilon_i^2}{n-k}} \tag{5}$$

3.4 Bayesian data analysis

When performing data analysis often prior knowledge is already available, such as information from an previous investigation. One widespread tool to update the prior information with results from new studies is the Bayes updating. The method can be applied among others to update the distribution function but also to update the regression model. However, the method is outside the scope of this publication and it is therefore only referred to the standard literature mentioned before.

4 Methods to estimate characteristic values (5% values)

In this chapter different methods to estimate the characteristic value of the strength properties (5% values) are introduced: Empirical approach, parametric and non-parametric approach according to EN 14358 (2016), based on distribution functions and a predictive approach. A comparison between the approaches is illustrated in Example I (Chapter 5.1).

It has to be distinguished between methods used for data description and methods used for the estimation of predictive values of the 5%-fractile. The latter ones also have to consider the statistical uncertainties related to the limited number of observations.

4.1 Empirical

The most straight forward approach to estimate the characteristic value of a population is by using a empirical approach. In most textbooks the empirical distribution function is defined according to Eq. (6), where n is the sample size and I is the indication of an event x_i.

$$\hat{F}_n(t) = \frac{1}{n} \sum_{i=1}^{n} I(x_i) \quad \text{with} \quad I(x_i \leq t) = 1 \quad \text{and} \quad I(x_i > t) = 0 \tag{6}$$

This approach is also used e.g. in EN 14358 (2016) combined with a linear interpolation between data points, if needed. The approach is very useful for large data samples. However, for small samples it leads to a bias (systematic underestimation of the characteristic value). At this point it has to be mentioned that some text books define the empirical distribution:

$$\hat{F}_n(t) = \frac{1}{n+1}\sum_{i=1}^{n} I(x_i) \quad \text{with} \quad I(x_i \leq t) = 1 \quad \text{and} \quad I(x_i > t) = 0 \tag{7}$$

4.2 Based on distribution function

If an appropriate distribution function is selected and the parameter are estimated, each fractile value can be estimated easily using inverse function. Compared to the other approaches presented in this chapter it also possible to handle different types of information (e.g. censored data).

Using these methods the statistical uncertainties related to the limited number of observations can also be considered for the estimation of the predictive values of the 5%-fractile.

4.3 EN 14358

Often the characteristic value is estimated using EN 14358 (2016). There two different approaches are presented:

- Parametric calculation
- Non parametric calculation

For the first one the procedure for Normal and for Lognormal distributed data is presented. In this chapter it is only refereed to the Lognormal distributed one. The procedure presented in EN 14358 (2016) is used to estimate the characteristic value on confidence level of 75%, meaning that the probability that the characteristic value is greater than the estimated value is 75%.

4.3.1 Parametric calculation

If it is assumed that the n test observations m_i are independent and can be represented with a Lognormal distribution, the characteristic value m_k can be calculated according to Eq. (8)-(10). Here $\bar{y}$ is the mean value, s_y is the standard deviation, $k_{0.75}(n)$ is the 75-percentile in a non-central t-distribution with $n-1$ degrees of freedom and the non-centrally parameter λ. u_{1-p} is the $(1-p)$-percentile of the standard Normal distribution function. For the calculation of the 5% quartile $p = 0.05$.

$$m_\mathrm{k} = exp(\bar{y} - k_\mathrm{s}(n)s_\mathrm{y}) \tag{8}$$

$$\bar{y} = \frac{1}{n}\sum_{i=1}^{n} \ln m_\mathrm{i} \qquad\qquad s_\mathrm{y} = max\begin{cases}\sqrt{\frac{1}{n-1}\sum_{i=1}^{n}(\ln m_\mathrm{i} - \bar{y})^2}\\ 0.05\end{cases} \tag{9}$$

$$k_{\mathrm{s}}(n) = \frac{k_{0.75}(n)}{\sqrt{n}} \qquad \lambda = u_{1-p} \cdot \sqrt{n} \tag{10}$$

4.3.2 Non parametric calculation

If the sample size is $n \geq 40$ also an non parametric approach is presented. The percentiles of the test data are calculated according to Eq. (6). Using the 5-percentile from the test data the 75% confidence interval can be calculated according to:

$$m_{\mathrm{k}} = y_{0.05}\left(1 - \frac{k_{0.05,0.75}V}{\sqrt{n}}\right) \qquad k_{0.05,0.75} = \frac{0.49n + 176}{0.28n + 7.1} \tag{11}$$

4.4 Predictive 5%-fractile values

According to modern design codes as the Eurocodes (EN 1990, 2002) characteristic values for strength related material properties have to be introduced as predictive values of the 5%-fractile. The general form of the predictive p%-fractile value can be given as:

$$x_{p,pred} = F^{-1}_{X,pred}(p) \quad \text{with} \quad f_{X,pred}(x) = \int f(x|\boldsymbol{\theta}) f''_{\boldsymbol{\Theta}}(\boldsymbol{\theta}|\hat{\mathbf{x}}) d\boldsymbol{\theta} \tag{12}$$

where $\hat{\mathbf{x}}$ are the sample observations, $\boldsymbol{\theta}$ are the parameters of the distribution function. The parameters $\boldsymbol{\theta}$ are realizations of the random vector $\boldsymbol{\theta}$ with the posterior joint probability density function $f''_{\boldsymbol{\Theta}}(\boldsymbol{\theta}|\hat{\mathbf{x}})$. Eq. (12) can be generally solved by reliability methods as FORM/SORM or numerical integration, however, analytical solutions exists, e.g. for the case where X is normally distributed or Lognormal distributed.

If no prior information is available the predictive value of the p%-fractile of a lognormal distributed random variable $x_{p,pred}$ is given as:

$$x_{p,pred} = exp\left(m + t_p(\nu)s\sqrt{1 + 1/n}\right) \tag{13}$$

where m is the mean value of the logarithmic data, s the standard deviation of the logarithmic data, n is the sample size and ν is defined by $\nu = n - 1$. $t_p(\nu)$ is the p%-fractile value of the t-distribution with ν degrees of freedom. It should be noted that this method is fully consistent with the Bayesian updating scheme introduced in Chapter 3.4.

5 Examples

5.1 Example I – Characteristic value

In this example the characteristic value is calculated for a randomly selected set of data using different approaches: Empirical (according to Eq. (6)), parametric and non-parametric approach according to EN 14358 (2016), predictive and based on distribution functions (the distribution functions are estimated using MLM). In this example the MLM is used without considering statistical uncertainties (see Section 4).

Therefore batches with specific sample sizes n of GL28h are randomly generated. All batches are Lognomal distributed with COV$= 0.15$. From each batch the characteristic values are estimated using different approaches. Below a Matlab code to simulate the data set (approximation) and to calculate the characteristic values using different approaches is presented.

Matlab code 1: Generation of a Lognormal distributed random data set, base characteristic value and COV (approximation) and calculation of characteristic values using different approaches.

```
%% input
    char_value=28; % characteristic value of the bending strength
    COV=0.15; %Coefficient of variation
    n = 100; %sample size
    p = .05; % 5-%fractile
    alpha=0.75; % confidence interval
%% parameter of the lognormal distribution
    sigma=(log(COV^2+1))^0.5; %approximation
    mu=log(char_value)-norminv(p,0,1)*COV;
%% simulate data
    sim_data=lognrnd(mu,sigma,n,1);
%% empirical
    sim_data_sort=sort(sim_data); % sort simulated data
    y_05 = sim_data_sort(n*0.05)
%% EN 14358 (parametric calcualtion)
    freedom = n-1;
    u_1p=norminv(1-p);
    shift=u_1p*n^0.5;
    k_alpha = nctinv(alpha,freedom,shift);
    k_s=k_alpha/n^0.5;
    y_mean=sum(log(sim_data(:)))/n; % mean
    y_std=max((sum((log(sim_data(:))-y_mean).^2)/(n-1))^0.5, 0.05); % std
    y_c=exp(y_mean-k_s*y_std)
%% EN 14358 - non-parametric approach
    %y_05 from empiral calculation
    k_05075=(0.49*n+17)/(0.28*n+7.1);
    V=std(sim_data)/mean(sim_data);
    y_c = y_05*(1-(k_05075*V)/n^0.5)
%% based on distributino function (MLM)
    [par]=lognfit(sim_data(:));
    y_c = logninv(p,par(1),par(2))
%% Predictive approach
    t_p = nctinv(p,freedom,0);
    m=sum(log(sim_data(:)))/n;
    s=(sum((log(sim_data(:))-m).^2)/(n-1))^0.5;
    y_c=exp(m+t_p*s*((1+1/n)^0.5))
```

In Figure 1 the characteristic values of the 1000 batches with a sample size $n = 40$ are illustrated, by a Normal distributed probability density function. The underestimation using the empirical approach results from the low sample size as described in Section 4. It is clear that both approaches described in EN 14358 (2016) underestimate the characteristic value, as they are defined to be determined on a confidence

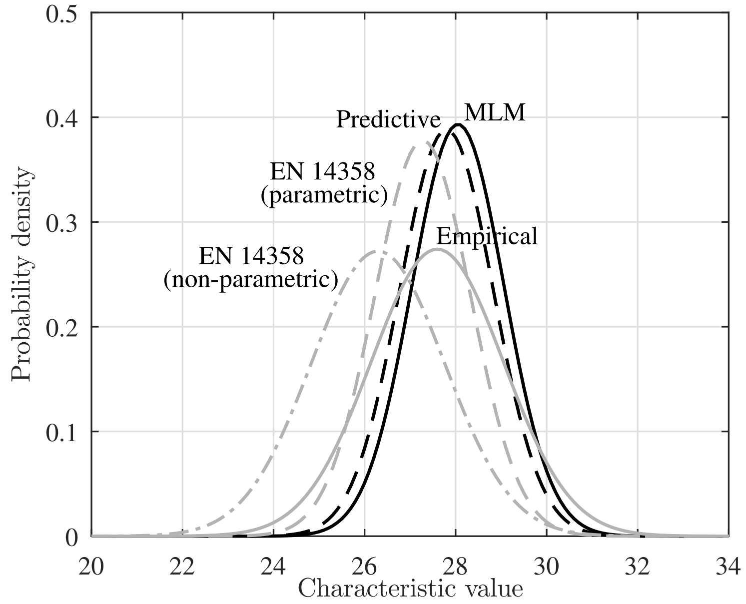

Figure 1: Characteristic value of 1000 randomly selected batches (GL28, sample size $n = 40$) using different approaches.

level of 75% (Section 4). Obviously the MLM and the predictive approach represents the best fit. Here it has to be mentioned that the selection of the distribution function is essential. In case that a Normal distribution would be assumed (even the data are Lognormal distributed) the characteristic value would be underestimated as well.

As mentioned the estimation of the characteristic value is more precise for larger sample sizes. The influence is illustrated in Figure 2 for batches with different sample sizes $n = 10, 20, 40, 100, 500, 1000$. The crosses and the corresponding line represents the 75% confidential level for the parametric approach presented in EN 14358 (2016).

5.2 Example II – Probability plot

Following the procedure described in Example I $n = 200$ test results are generated: Strength class GL28h, COV=0.15, Lognormal distributed. Two probability plots are illustrated in Figure 3: Weibull and Lognormal distribution. As expected the Lognormal distribution fits pretty good, meaning the points are all near the line. In contrast the Weibul distribution shows large deviations for small and large realizations.

Matlab code 2: Probability plot (line 1-12 can be copied from Matlab code 1).

```
13 %% plots for differnt distributions
14     probplot('Weibull',sim_data)
15     %probplot('Lognormal',sim_data)
```

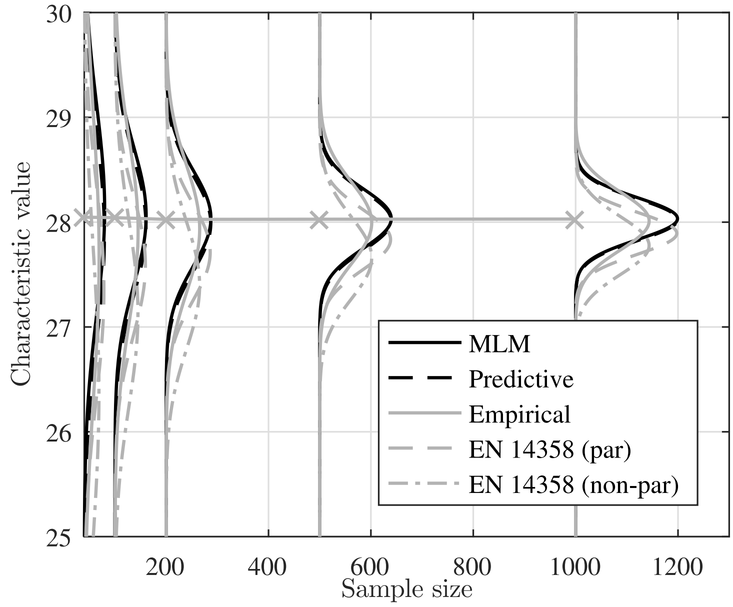

Figure 2: Characteristic value of 1000 randomly selected batches (GL28) with different sample sizes $n = 10, 20, 40, 100, 500, 1000$ *using different approaches.*

5.3 Example III – Type of information

When testing finger joint connections (FJ) in tension only a limited number of the test specimens fail in the FJ. Some of the specimens might fail in the clear wood or small knots next to the FJ. Others might fail in the area of the clamping jaws due to the increased lateral pressure and the associated local strength reduction. If the lateral pressure is to small the FJ might not fail at all. Often, a mixed failure containing a combination of this failure modes can be observed.

If the failure occurs outside the FJ the actual tensile strength of the FJ is not known. However, it is obvious that the tested FJ was able to resist the applied load, thus the tensile strength of the FJ must be equal or larger to the maximal stresses observed in the experimental investigation. Thus the obtained data set contains direct equality and in-equality information and such data is called censored data.

In Fink et al. (2018b), among others, 20 FJ fabricated from strength grade L40 were investigated and the influence of considering censored data in the analysis is discussed. The experimental investigations are performed different as proposed in EN 408 (2003). Also the sample selection is not representative for the basic population. In this respect absolute values of the strength properties should be considered carefully. However, for the purpose of this example, that is the investigation of the influence of considering censored data in the analysis the sample is appropriate, in particular due to the large variety of the corresponding results.

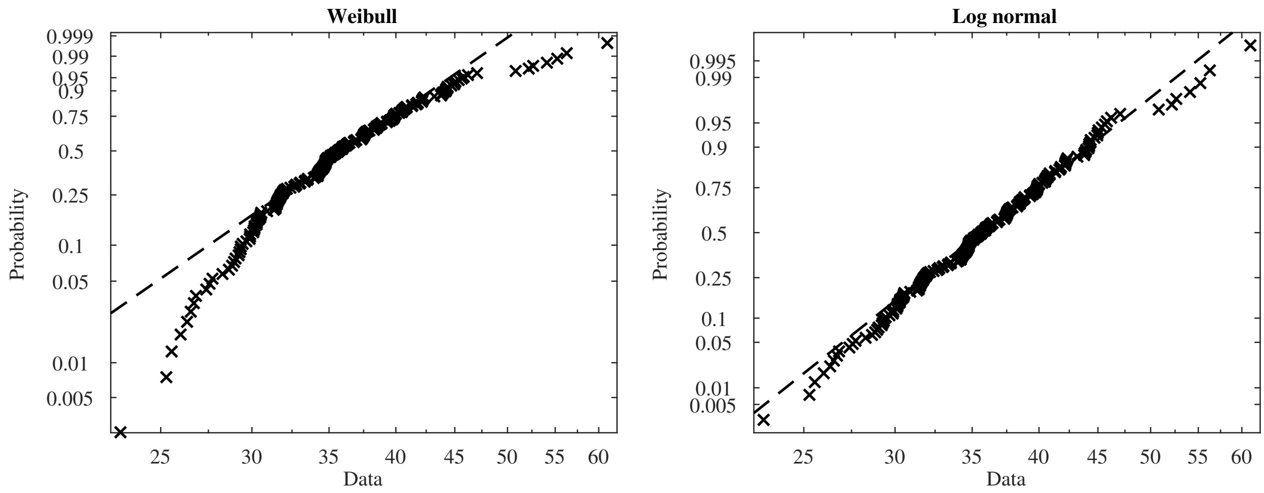

Figure 3: Probability plot for two different distribution functions.

Based on local fracture types, the FJ are classified into four types of failure:

- FJ failure (12 specimens)
- Wood failure (0 specimens)
- No failure (5 specimens)
- Mixed failure (3 specimens)

The different types of failure leads to different types of information. As mentioned only specimens with FJ failure can be categorized as non-censored information (equality type information); i.e. the identified value (here the maximal tensile stresses) is equal to the investigated parameter (here the tensile strength of the FJ). All other specimens have to be considered as inequality type information; meaning that the tensile strength of the FJ is equal or larger as the maximal tensile stresses applied during the test.

In the following the error due to wrong consideration of censored data is investigated and discussed. Therefore, different approaches for data analysis are applied and the corresponding error are presented. The tensile strength of the FJ is estimated using MLM assuming that the data can be represented by a Lognormal distribution. Tree different scenarios are investigated. The results are illustrated in Figure 4:

- Scenario I: FJ failure are considered as equality type information, all the others are inequality type information (correct approach)
- Scenario II: All tests are considered as equality type information
- Scenario III: Specimens with no failures or wood failures are neglected, all other specimens are considered as Scenario I.

If the censored information is not considered, meaning all test results as equality type information (Scenario II), the tensile strength is underestimated: The mean value is approx. 2 MPa (5%) smaller and the COV = 0.10 instead of COV = 0.15. In this particular case the underestimation of the variability even results in an overestimation of the characteristic value.

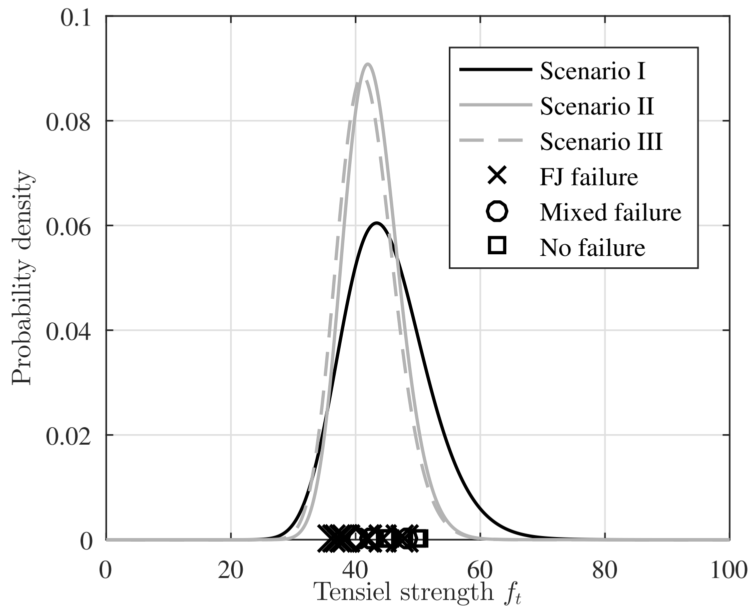

Figure 4: Probability distribution of the tensile strength of FJ for different approaches (Fink et al., 2018b).

Scenario III looks rather similar compared to the results from scenario II; meaning a significant underestimation of the strength properties and the variation; resulting in a slightl overestimation of the characteristic value.

As mentioned both type of errors lead to an underestimation of the basic population. However, the variability of the distribution might be underestimated as well. An underestimation of the variability might result in an overestimation of important characteristics like the 5%-value. This is of particular importance as neglecting of censored test results of censored information might mistakenly be assumed to be conservative.

The analysis of FJ might be a special case as many different failure types can occur. Performing experimental investigations according to test standards such as EN 408 (2003), the specimens are expected to fail accordingly and thus the number of censored data is limited. However, censored data have to be considered in many applications, anyway. One typical example in timber engineering is the investigation of slotted in steel plate connections under tensile exposure, if the specimens contains actually two connections, one on each end. There only the weaker connection fails, whereas the other connection has to be considered as censored information.

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