Engineering Structures and Technologies
ISSN 2029-882X / eISSN 2029-8838
2022 Volume 14 Issue 1: 7–23
https://doi.org/10.3846/est.2022.23007

MODIFICATION OF PARTIAL SAFETY FACTORS FOR A SEMI-PROBABILISTIC EVALUATION OF EXISTING TIMBER STRUCTURES

Maria LOEBJINSKI[1*], Zheng LI[2], Wolgang RUG[3], Hartmut PASTERNAK[4]

[1]Country Institute for Building Construction and Timber Engineering, Technische Universität Braunschweig, Braunschweig, Germany
[2]Institute of Civil Engineering, Technische Universität Berlin, Berlin, Germany
[3]Faculty of Timber Engineering, Eberswalde University for Sustainable Development, Eberswalde, Germany
[4]Institute of Civil and Structural Engineering, Brandenburg University of Technology, Cottbus, Germany

Received 2 June 2023; accepted 7 June 2024

Abstract. The evaluation of the load-bearing capacity of existing structures is a central and important part in the work of structural engineers. Currently, engineers are confronted with the challenge of applying design rules developed for new structures in the evaluation of existing ones as no specific recommendations exist on a European level. As a contribution to this, a first step of this study is the evaluation of the reliability level of timber elements subjected to common limit states. Based on these analyses, modifications of the target reliability and of partial safety factors (PSF) for existing structures on the resistance side are studied. Considering a modification of the target reliability, the PSF for the material strength could be proposed with $\gamma_{M,opt}$ = 1.20 for compressive and flexural strength in limit states, where variable actions are present. Additionally, options for incorporating updated material parameters from a survey on site supported by technical devices are discussed and further need for research is identified. Subsequently, this paper provides a stepwise evaluation procedure including modified PSF considering both, an update of the target reliability and update of the material parameters obtained by a survey on site and is thus adaptive for different individual cases and level of information.

Keywords: timber, existing buildings, semi-probabilistic evaluation, code calibration.

Introduction

Eurocodes form the basis of design and verification of structures. At state, rules for the design of new structures are applied for the evaluation of the load-bearing capacity of existing ones. The newly introduced Technical Specification CEN/TS 17440:2020-10 (European Committee for Standardization [CEN], 2020) as a first common specification for the evaluation of existing structures offers new paths: qualitative evaluation, quantitative evaluation or a combination of both. A qualitative evaluation is based on past performance of the structure. For a quantitative evaluation, the Partial Factor Method is recommended, reliability-based methods and risk-informed methods can be applied additionally. The assessment should verify that the structure has adequate reliability. Target values can be taken from EN 1990:2010-12 (CEN, 2010a) or may be defined in National Annexes. As the target reliability is defined based on an optimization of failure consequences and relative costs of safety measures (see ISO 2394:2015(E) (International Organization for Standardization [ISO], 2015) or SIA 269:2011 (Schweizerischer Ingenieur- und Architektenverein [SIA], 2011), an altered definition for existing structures is possible. Annex C of TS 17440:2019 (E) (CEN, 2020) formulates that "The target reliability level for existing structures can be lower than that for new structures as the relative cost of safety measures to increase the reliability of an existing structure is greater than that for a new structure". (C.3.(4) Note 2). Thus, CEN/TS 17440:2020-10 (CEN, 2020) opens new ways for the adjustment of the well-proven semi-probabilistic verification format of the Eurocodes for their application on the special demands in the evaluation of existing structures.

*Corresponding author. E-mail: *m.loebjinski@tu-braunschweig.de*

Copyright © 2022 The Author(s). Published by Vilnius Gediminas Technical University

This is an Open Access article distributed under the terms of the Creative Commons Attribution License (http://creativecommons.org/licenses/by/4.0/), which permits unrestricted use, distribution, and reproduction in any medium, provided the original author and source are credited.

Basis for an evaluation of the load-bearing capacity of an existing structure is a careful and detailed investigation including an assessment in situ. For existing structures, different national recommendations, on an international level ISO 13822:2010 (ISO, 2010) and with special focus on existing timber structures EN 17121:2019-12 (CEN, 2019) provide the basis for a qualified assessment.

Research in timber engineering has focused on the assessment of timber structures analyzing the potential of non-/semi-destructive devices. The list of authors is long and just some examples can be named here: Kotlínová et al. (2008), Palaia et al. (2008), Piazza and Riggio (2008), Hösl and Dietsch (2010), Kasal and Tannert (2011), Saporiti Machado et al. (2015), Linke et al. (2022). Updating techniques and statistical evaluation has also been a central part in research, examples are Köhler (2010) and Sousa (2013) who has been working in his dissertation with methods to update the load-bearing capacity of timber members by different statistical methods (Sousa, 2013; Sousa et al., 2015b). However, updated information from an assessment on site should be considered in the modified semi-probabilistic evaluation. Here, a lack of research and standardization is still present. Results are always specific for the structure at hand what complicates the latter. Thus, a structured process including options to modify safety elements is needed to systematize the use of updated information in the evaluation.

While an assessment procedure is already part of EN 17121:2019-12 (CEN, 2019) mentioned above, current codes for design and verification of timber structures on a European level do not consider the special challenges when the evaluation of the load-bearing capacity of an existing timber structure becomes necessary. This can be due to changes of the structural system, load changes or doubts concerning the capacity of the structural material which can be in the case of timber due to biological, chemical or mechanical influences. Proposals for modifications of the semi-probabilistic design concept for existing structures can be found for structures made from concrete in the literature, see Fédération internationale du betón [FIB] (2016), but are still missing for timber.

In the context of the development of a sustainable building sector providing better options to preserve built infrastructure and thus energy and resources, the need for standardized approaches to evaluate the load-bearing capacity of existing structures increases, what is also true for timber. This is why this research aims to contribute to this crucial task by providing modifications of the semi-probabilistic design format for existing timber structures. These options are structured into a stepwise evaluation procedure considering different levels of information, as every project, especially when dealing with existing timber structures, is a special task needing individual solutions.

1. Methodology

Loads and material parameters are subjected to a natural variability. Besides, model uncertainties have to be considered. Thus, the resulting reliability of different limit states varies depending on the input variables. Hence, a standard cannot provide a reliability level that is most optimal for all kinds of limit states Turkstra (1970), safety elements are defined by optimization. What is more, Melchers (1999) emphasizes that reliability indices can hardly serve as indicators without direct link to the model parameters used to calculate them. The calculation of the implicit safety level of current design is hence a good and goal-oriented option to produce reliable values. This approach presupposes that built structures satisfy public safety requirements (Baravalle & Köhler, 2017). The reliability index β is thus a comparative value of the actual analysis and should not be used separated from the underlying model assumptions.

Based on these considerations, the following path is chosen: Selected timber members in common limit states are designed for a 100% utilization of the semi-probabilistic design-check equations with current partial safety factors (PSF) from EC 0 (EN 1990:2010-12 (CEN, 2010a)) and EC 5 (EN 1995-1-1:2010-12 (CEN, 2010b)) ($\gamma_G = 1.35$ for permanent loads, $\gamma_Q = 1.5$ for variable loads, $\gamma_M = 1.3$ for structural timber strength properties) and evaluated by reliability analyses. Here, the modelling of load and resistance variables is crucial (see explanations below). The resulting reliability level is determined for a set of limit states, load ratios and a range of the coefficient of variation (*cov*) of the material strength. Based on the obtained reliability level, target reliabilities for existing structures are discussed and PSF are calibrated by probabilistic optimization for chosen limit states aiming for an optimized verification of existing timber structures with dominating limit states clearly defined. Besides, options to update PSF based on new information are discussed. This information depends on the *Knowledge Level* of the qualified survey on site. Thus, two paths are available for an optimization of PSF: probabilistic optimization by calibration and optimization based on an updated resistance variable. Finally, a proposal for a stepwise evaluation procedure for practical application is presented. The methodology is presented in Figure 1.

Selected limit states for the analyses are given in Figure 2. The numbering is used in the presentation of results.

The general formulation of the limit state function (LSF) is given by Eq. (1):

$$g_i\left(R,G,Q_1,Q_2,\theta_R,\theta_G,\theta_{Q1},\theta_{Q2}\right)=z_d\theta_R R - LA_G\theta_G G-\left(1-a_G\right)\left(LA_{Q1}\theta_{Q1}E_1\cdot\left(1-LA_{Q1}\right)\theta_{Q2}E_2\right)\overset{!}{=}0, \tag{1}$$

with R the resistance variable and θ_G its model uncertainty, G the permanent action and θ_G the model uncertainty associated with it. Q_1 and Q_2 are two variable actions, θ_{Q1} and θ_{Q2} their model uncertainty variables. LA_G is the load ratio of the permanent action and LA_{Q1} is the load ratio of the first variable action in relation to the total variable load. Variable loads are combined using the *Ferry Borges* & *Castanheta* combination rule (Ferry Borges & Castanheta, 1971). What is more, z_d is the design parameter to ensure a one-hundred percent utilisation of the semi-probabilistic design equation, see Baravalle (2017). It can be calculated by Eq. (2) for one an Eq. (3) for two variable actions:

$$z_d \frac{k_{mod} \cdot f_k}{\gamma_M} - \gamma_G LA_G g_k - \gamma_Q \left(1 - LA_G\right) q_k \stackrel{!}{=} 0; \tag{2}$$

$$z_d \frac{k_{mod} \cdot f_k}{\gamma_M} - \gamma_G LA_G g_k - \gamma_Q \left(1 - LA_G\right)\left(LA_{Q1} q_{1,k} + \left(1 - LA_{Q1}\right) q_{2,k}\right) \stackrel{!}{=} 0, \tag{3}$$

with k_{mod} the modification factor for load duration and service class, f_k, g_k and q_k the characteristic values for the material strength, permanent and variable loads respectively. Limit states have been formulated according to EC 5-1 (CEN, 2010b). Probabilistic parameters are given in Table 1. The *cov's* of the material strength are based on (Joint Committee on Structural Safety [JCSS], 2006) and a broad literature study in (Loebjinski, 2021), see summary of results in Figure 3 to Figure 5 (the bars represent different studies). The cov of the material strength depends on numerous factors including the strength grading procedure, growth characteristics on a macroscopic and microscopic level and the actual strength property considered. As the variability of results comparing different studies is quite high, the recommendations in (JCSS, 2006) are used as a basis and the influence of alterations of the material *cov* are studied in the parameter study.

Cov's for live loads are based on own approximations applying parameters from CIB (1989), JCSS (2001b), see Loebjinski (2021). Parameters for snow and wind loads are based on JCSS (2001b) (wind) and Grünberg (2004) (snow). Load change rates have been oriented on Baravalle (2017) and adopted for German climatic conditions.

Please note that in the current version of the JRC documentation "*Reliability background in the Eurocodes*" (unpublished) a higher variability of snow and wind loads

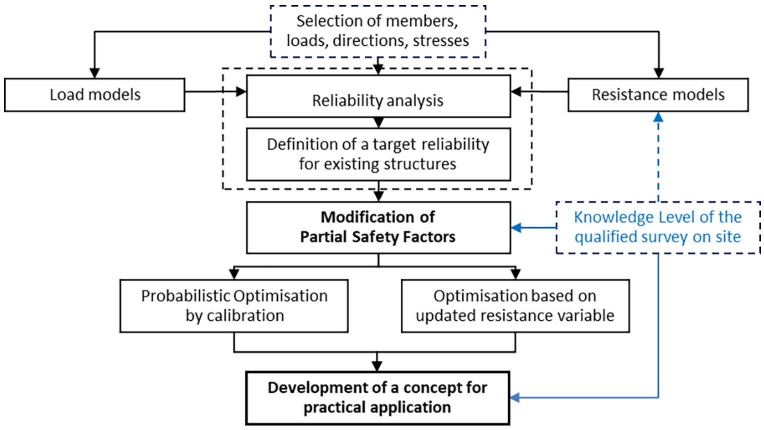

Figure 1. Illustration of methodology

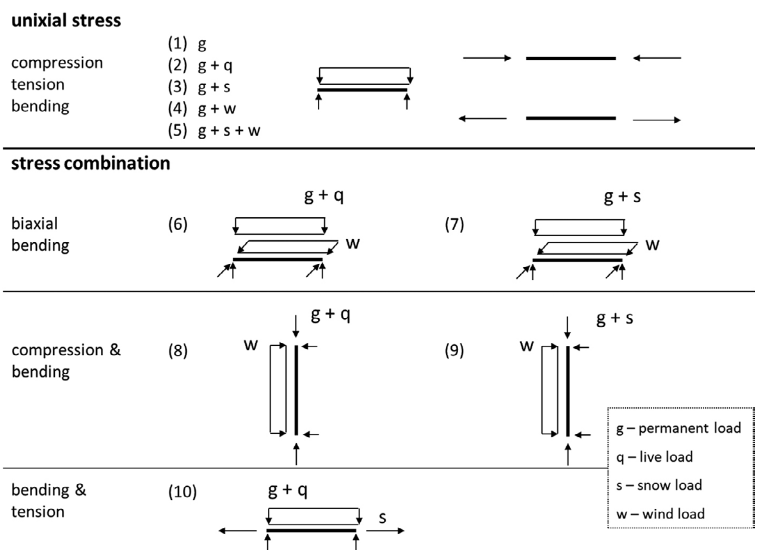

Figure 2. Loads, load directions and stresses considered in the study

has been documented. This especially affects the time-invariant part (taken here with $V_\theta = 0.10$). Besides, the variability of the bending strength is lower in the mentioned report ($V_{fm} = 0.20$) than in the present study ($V_{fm} = 0.25$). As the report was not published at the time of preparation of this contribution, values have been assumed based on the official background documentation of *DIN 1055-100* (Grünberg, 2004) and the *JCSS PMC* (JCSS, 2001b). New results on modelling variable actions need to be considered in further studies.

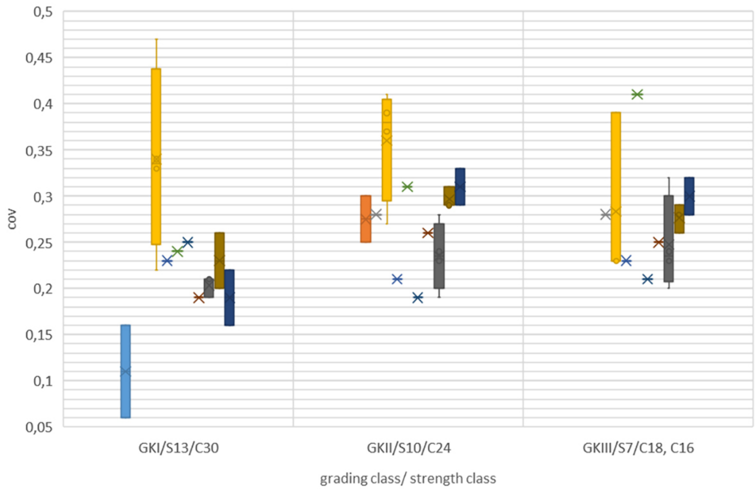

Figure 3. Coefficient of variation for strength properties of softwood, sawn wood, visual grading, **bending strength** (different colours of the bars stand for different investigations)

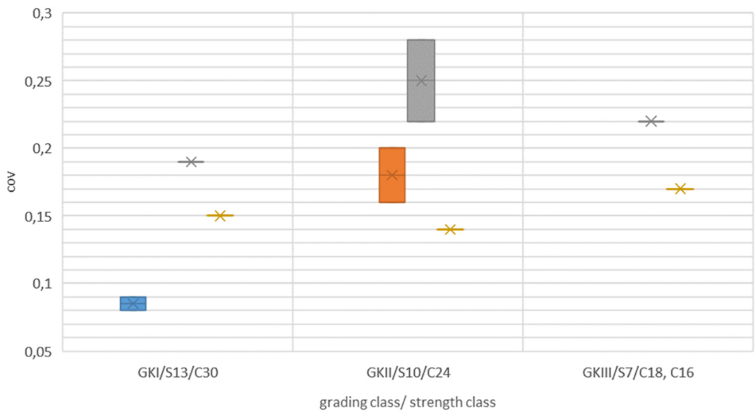

Figure 4. Coefficient of variation for strength properties of softwood, sawn wood, visual grading, **compression strength** (different colours of the bars stand for different investigations)

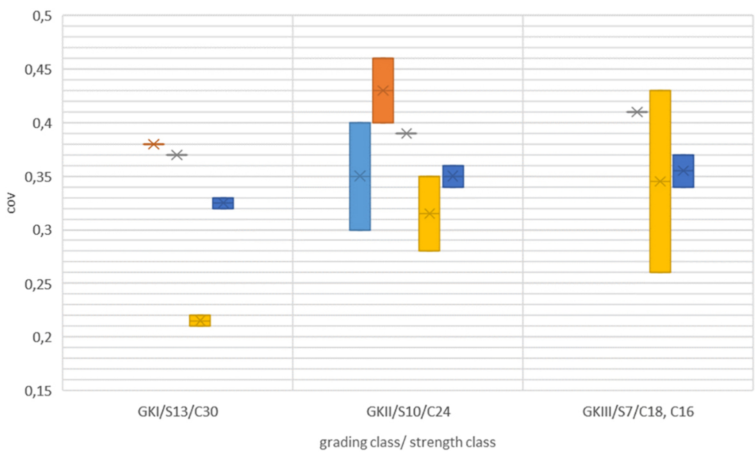

Figure 5. Coefficient of variation for strength properties of softwood, sawn wood, visual grading, **tension strength** (different colours of the bars stand for different investigations)

Table 1. Probabilistic parameters, T_{ref} = 50 years

Variable				Distr.	μ	V
R	Timber[1]	Bending strength	f_m	LN	1.00	0.25
		Comp. strength parallel to grain	$f_{c,0}$	LN	1.00	0.20
		Tension strength parallel to grain	$f_{t,0}$	LN	1.00	0.30
E	Permanent loads[2]		G	N	1.00	0.10
	Live Load[3]					
	Small room (A ≤ 20m²)		N	GUM	1.00	0.40
	Large room (A > 20m²)				1.00	0.25
	Snow load[4]		S	GUM	1.00	0.25
			n_p	det.	50·60	–
			n_r	det.	10	–
	Wind load[4]		W	GUM	1.00	0.16
			n_p	det.	50·365	–
			n_r	det.	50·365	–
Model[5]	Resistance		θ_f	N	1.00	0.07
	Load	Permanent load	θ_G	N	1.00	0.05
		Live load	θ_N	N	1.00	0.10
		Snow load	θ_S	N	1.00	0.10
		Wind load	θ_W	N	1.00	0.10

Notes: [1]Indicative from (JCSS, 2006), analyses for a range of values; [2]Based on (JCSS, 2006); [3]T_{ref} = 50a, based on own calculations; [4]T_{ref} = 50a, based on (Baravalle, 2017; Grünberg, 2004; JCSS, 2001b); [5]Multiplicative, attached to variable.

The reliability analyses are performed by *First Order Reliability Method* (*FORM*) (Hasofer & Lind, 1974) in MATLAB® (MathWorks, 2016). Calculations have been double-checked by exemplary hand calculations with the help of Spaethe (1992) and a selection of *Monte Carlo Simulations* (*MCS*) in MATLAB® (MathWorks, 2016). To study the influence of the variables on the calculated reliability, sensitivity factors for selected configurations are shown. All calculations have been performed for T_{ref} = 50 years.

2. Results

2.1. Reliability analyses

2.1.1. Uniaxial stresses

Figure 2 to Figure 4 show the reliability for uniaxial stress from different load combinations. Results are illustrated for different cov's of the material strength. Values exceeding the target β = 3.8 (CEN, 2010a) for consequence class 2 (T_{ref} = 50 years) are marked by red lines. Figure 6 shows the reliability index β depending on the cov of the material strength V_R for different cov's of permanent load; Figure 7 and Figure 8 illustrate the results for different load ratios of the permanent load to the total load.

Permanent loads (1): For $V_G \geq 0.10$ the calculated reliability is lower than the target from EC 0, Annex C. The sensitivity factors (Table 2) confirm the dominating influence of the structural resistance on the reliability. Please note that $\sum \alpha^2 = 1$; an α-value close to 1 shows a great influence of the considered value on the calculated reliability.

Table 2. Exemplary sensitivity factors (case 1)

V_R	f	G	θ_f	θ_G
V_G = 0.05				
0.20	–0.89	0.21	–0.35	0.21
0.25	–0.92	0.18	–0.28	0.18
0.30	–0.95	0.16	–0.24	0.16
V_G = 0.10				
0.20	–0.84	0.37	–0.33	0.21
0.25	–0.89	0.32	–0.27	0.18
0.30	–0.92	0.29	–0.23	0.15
V_G = 0.15				
0.20	–0.80	0.48	–0.30	0.19
0.25	–0.85	0.43	–0.26	0.17
0.30	–0.88	0.38	–0.22	0.15

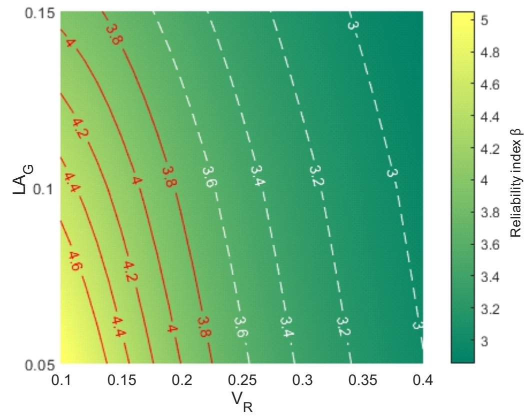

Figure 6. Reliability index β for permanent load dependent on the cov of the material strength V_R [case (1)]

Permanent and live load (2): Figure 7 depicts the results for permanent and live load. The reliability is lower especially for high load ratios of the variable load. However, even for large rooms with a moderate variability of the live load, the target value $\beta_t = 3.8$ is not reached. Table 3 shows selected sensitivity factors.

Permanent and snow load (3) or wind load (4): Figure 8 shows, that in a reference period $T_{ref} = 50a$ for permanent load and snow load a lower reliability index is calculated compared to wind load which is reasonable as for wind loads a lower *cov* has been set. Moreover, the influence of the choice of the distribution for the material strength is illustrated; a Normal distribution leads to lower values. Table 4 shows the sensitivity factors for a lower variability of the variable load (compared to Table 3).

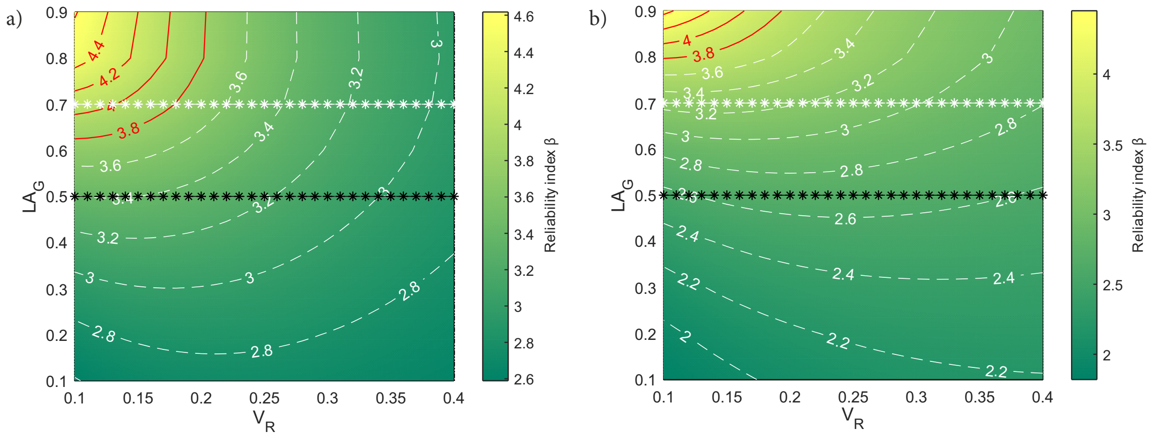

Figure 7. Reliability index β for uniaxial stress by permanent load + live load for a) $V_N = 0.25$ and b) $V_N = 0.40$ dependent on the cov of the material strength V_R [case (2)]

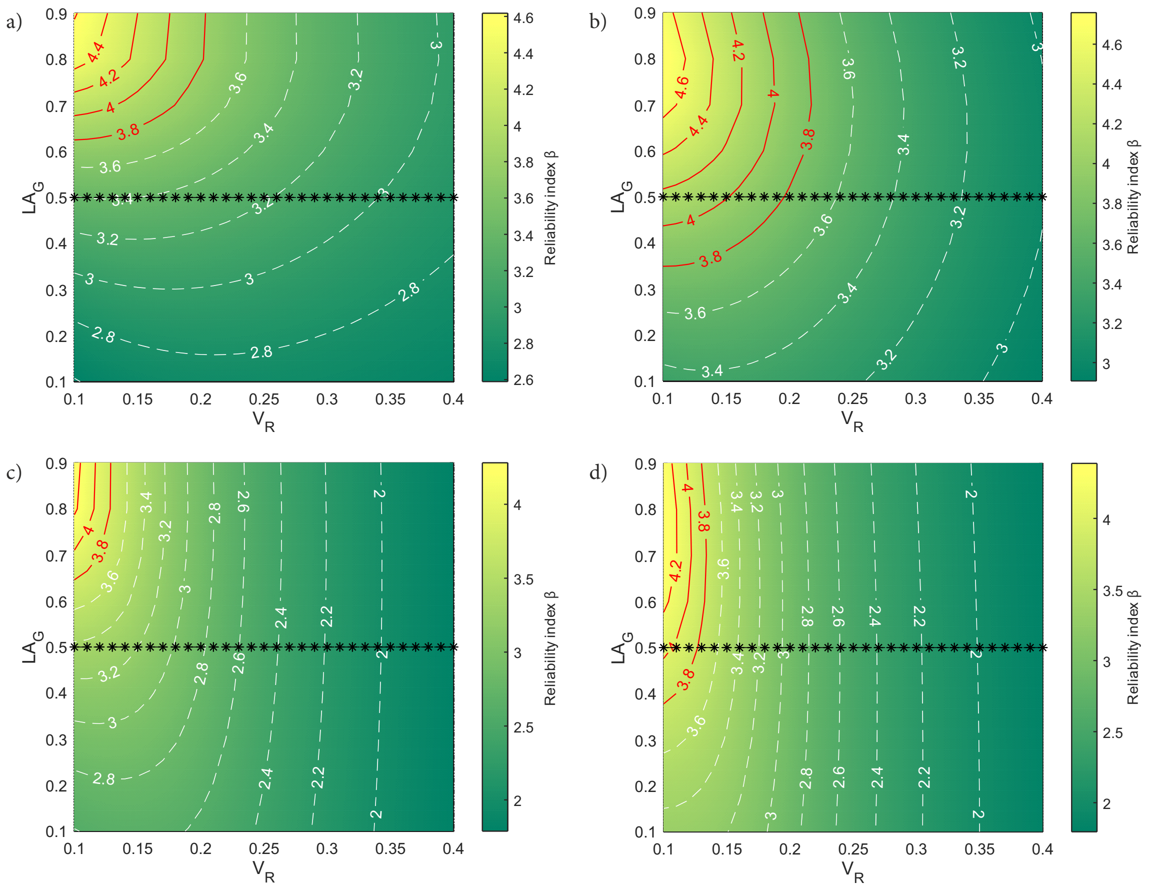

Figure 8. Reliability index β for permanent load and a) snow load, Lognormal distribution for material strength; b) wind load, Lognormal distribution for material strength; c) snow load, Normal distribution for material strength; d) wind load, Normal distribution for material strength dependent on the cov of the material strength V_R [case (3) & (3)]

Table 4. Exemplary sensitivity factors, $V_Q = 0.16$

V_R	f	G	W	θ_f	θ_G	θ_W
$LA_G = 0.3$						
0.20	–0.86	0.19	0.26	–0.33	0.14	0.14
0.25	–0.91	0.17	0.20	–0.28	0.12	0.12
0.30	–0.93	0.15	0.17	–0.23	0.11	0.10
$LA_G = 0.5$						
0.20	–0.79	0.11	0.47	–0.30	0.08	0.22
0.25	–0.86	0.10	0.37	–0.26	0.07	0.19
0.30	–0.90	0.09	0.30	–0.23	0.07	0.16
$LA_G = 0.7$						
0.20	–0.71	0.05	0.60	–0.27	0.04	0.26
0.25	–0.80	0.05	0.50	–0.24	0.04	0.23
0.30	–0.85	0.05	0.42	–0.21	0.03	0.21

Load and stress combinations: In Figure 9 to Figure 13 the reliability index β is given depending on the load ratio of one of two variable loads declared as "load no. 1" LA_{Q1} and different load ratios of the permanent load LA_G. Figure 9, Figure 10 and Figure 11 illustrate the results for uniaxial stress by permanent load, snow and wind load, two-axial flexural stress by permanent and wind load and two-axial flexural stress by permanent load, snow and wind load respectively. Table 5 contains exemplary values for sensitivity factors for the results shown in Figure 10. Figure 12 and Figure 13 depict the results for bending and compression and bending and tension respectively with exemplary sensitivity factors given in Table 6 for Figure 12a, Table 7 for Figure 12b and Table 8 for Figure 13. Especially the latter show low reliability indices which is justified by the high *cov* of the tension strength as well as the variability of the variable loads. Exemplary sensitivity factors are given.

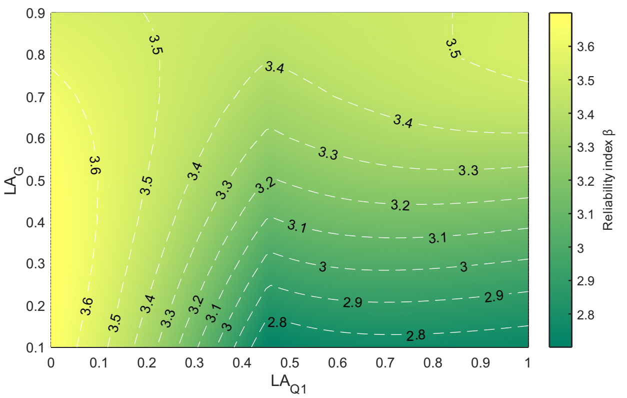

Figure 9. Reliability index β for uniaxial stress by permanent load + snow load + wind load dependent on the ratio of variable load No. 1 (snow load) [case (5)]

2.1.2. Two-axial flexural stress

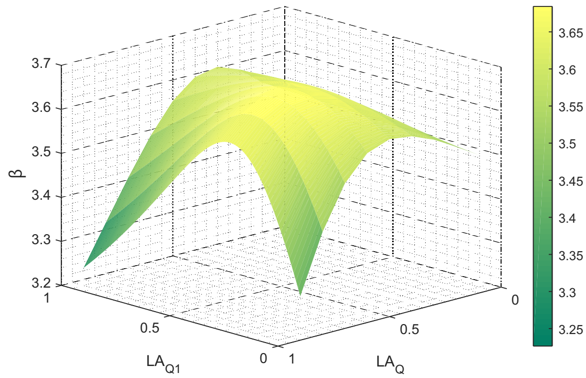

Figure 10. Reliability index β for two-axial bending by permanent load + wind load dependent on the ratio of variable load No. 1 (wind load vertical) [case (4)]

Table 5. Exemplary sensitivity factors, $V_R = 0.25$, $LA_G = 0.5$

V_R	f	G	S	W	θ_f	θ_G	θ_S	θ_W
$LA_{Q1} = 0.3$								
0.20	–0.88	0.21	0.18	0.00	–0.34	0.11	0.07	0.14
0.25	–0.92	0.18	0.14	0.00	–0.28	0.09	0.06	0.12
$LA_{Q1} = 0.5$								
0.20	–0.84	0.19	0.34	0.00	–0.32	0.10	0.12	0.09
0.25	–0.90	0.17	0.26	0.00	–0.27	0.09	0.10	0.08
$LA_{Q1} = 0.7$								
0.20	–0.78	0.16	0.50	0.00	–0.30	0.08	0.17	0.04
0.25	–0.85	0.15	0.39	0.00	–0.26	0.08	0.14	0.04

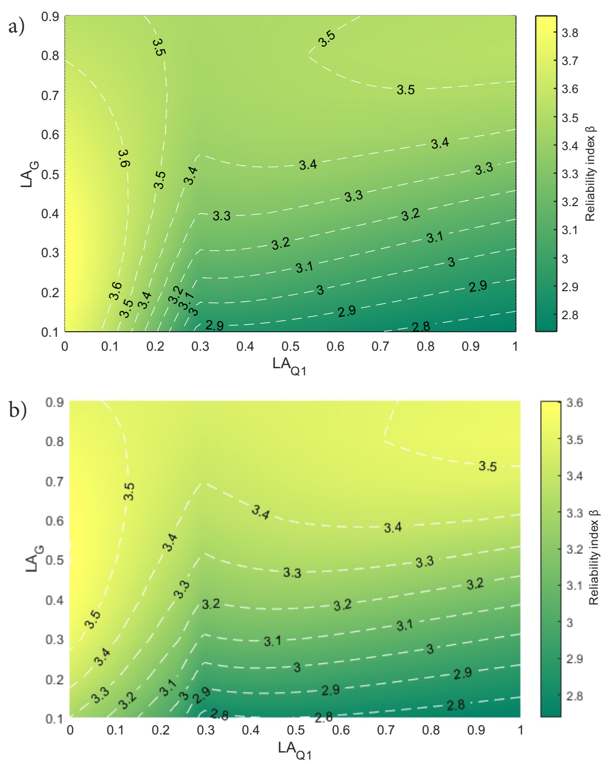

Figure 11. Reliability index β for two-axial bending a) permanent load + snow load + wind load, b) permanent load + live load + wind load dependent on the ratio of variable load No. 1 (snow load) (resistance variables correlated by ρ = 0.8, h/b = 1/2) [case (6) & (7)]

2.1.3. Stress combination flexural and compression stress

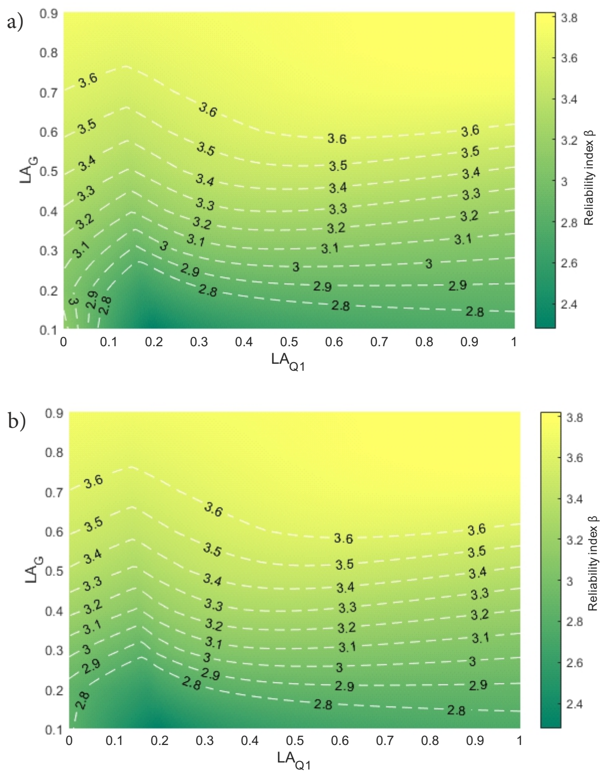

Figure 12. Reliability index β for compression by a) permanent load + a) snow load; b) live load; bending by wind load dependent on the ratio of variable load No. 1 (snow load), (resistance variables correlated by ρ = 0.8, h/b = 1/2) [case (8) & (9)]

Table 6. Exemplary sensitivity factors, $V_{R,m} = 0.25$, $V_{R,c} = 0.20$, $LA_G = 0.5$ (8)

f_m	f_c	G	S	W	θ_f	θ_G	θ_S	θ_W
$LA_{Q1} = 0.3$								
–0.74	–0.44	0.26	0.23	0.00	–0.33	0.14	0.09	0.06
$LA_{Q1} = 0.5$								
–0.68	–0.44	0.21	0.41	0.00	–0.31	0.11	0.14	0.04
$LA_{Q1} = 0.7$								
–0.61	–0.43	0.16	0.54	0.00	–0.28	0.09	0.18	0.02

Table 7. Exemplary sensitivity factors, $V_{R,m} = 0.25$, $V_{R,c} = 0.20$, $LA_G = 0.5$ (9)

f_m	f_c	G	S	W	θ_f	θ_G	θ_S	θ_W
$LA_{Q1} = 0.3$								
–0.74	–0.44	0.26	0.23	0.00	–0.33	0.14	0.09	0.06
$LA_{Q1} = 0.5$								
–0.68	–0.44	0.21	0.41	0.00	–0.31	0.11	0.14	0.04
$LA_{Q1} = 0.7$								
–0.61	–0.43	0.16	0.54	0.00	–0.28	0.09	0.18	0.02

2.1.4. Stress combination bending and tension stress

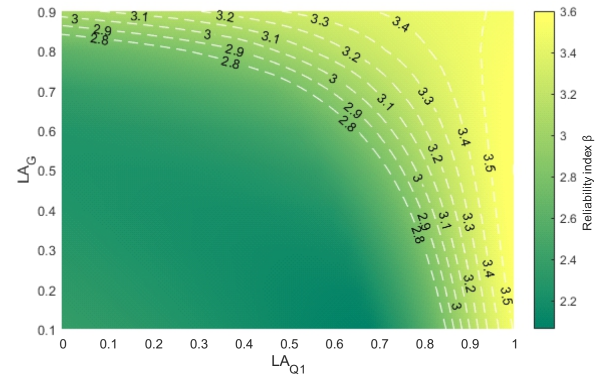

Figure 13. Reliability index β for bending by permanent load + live load and tension by snow load dependent on the ratio of variable load No. 1 (live load), a) 2D; b) 3D for better illustration [case (10)]

Table 8. Exemplary sensitivity factors, $V_{R,m} = 0.25$, $V_{R,t} = 0.30$ (10)

f_m	f_t	G	N	S	θ_f	θ_G	θ_N	θ_S
$LA_{Q1} = 0.3$								
–0.62	–0.34	0.06	0.01	0.65	–0.19	0.03	0.02	0.19
$LA_{Q1} = 0.5$								
–0.68	–0.32	0.07	0.01	0.60	–0.21	0.04	0.03	0.17
$LA_{Q1} = 0.7$								
–0.77	–0.26	0.10	0.03	0.50	–0.23	0.05	0.07	0.14

2.2. Summary and discussion of reliability analyses

The analyses of these rather simple limit states result in significant lower reliability indices than the target from *EC 0 Annex C* (CEN, 2010a) for consequence class CC2 ($\beta = 3.8$, $T_{ref} = 50$ years). The aim of the present study is to work out a proposal for an adjusted semi-probabilistic verification format for existing timber structures. As a basis it is assumed that current design fulfils public safety requirements and can thus serve as a basis to define target values. This assumption is justified as no increased number of failure events has been documented for these common limit states. Thus, it is suggested to take the average reliability level from the presented analyses as a target value. The combination of tension and bending can be excluded as these members mostly do not show a utilization of 100%; verification is often governed by connections which is not in the focus of this work. Based on the presented analyses $\beta_{t,exis} = 3.2$ ($T_{ref} = 50$ years) is suggested.

According to Diamantidis et al. (2016a), two levels are needed for the evaluation of existing structures: a minimum level and a target level. For the evaluation of existing members in structures not affected by structural changes and supposed to clearly defined dominating limit states it is suggested to apply a target reliability, that considers aspects of economic optimization. Diamantidis et al. (2017) suggest to apply $\Delta\beta = -0.5$ for a target reliability of existing structures and $\Delta\beta = -1.5$ for a minimum reliability. (Vrouwenvelder, 2002) suggests to move one line higher for the relative costs of safety measures from *ISO 2394:2015* (ISO, 2015) what results in $\Delta\beta = -0.2\ldots-0.9$. Based on the results of the analyses, $\beta_{t,eval} = 2.9$ is suggested as a target for members in service (T_{ref} = 50 years) that are not affected by damages or major changes which is $\Delta\beta = -0.3$ based on the calculated implicit level. During calibration of PSF, a scatter around the target will occur. During the calibration for the defined target, the reliability should not fall below the minimum level $\beta_0 = 2.5$, which is the minimum reliability level under economic optimization in Joint Research Centre (2015). Alterations of structural members should be verified using the same safety elements as for new structures.

Based on the results, a proposal for the target reliability is worked out. γ_M is then calibrated for a set of limit states by *FORM*. As permanent loads can be updated on site, an update of permanent actions as suggested in *SIA 269/2011* (SIA, 2011) is considered in the modification of γ_G and an adjustment of the stochastic properties in the calibration. To avoid systematic programming errors, the verification is done by *Monte Carlo Simulation* (*MCS*). All calculations have been performed on single structural components. Calibrated PSF and a suggestion for the update of γ_M based on a reference property are structured in an evaluation procedure for practical application.

3. Proposal for a modified semi-probabilistic evaluation

3.1. Proposed framework

The evaluation of the load-bearing capacity of an existing structure differs fundamentally from the design of a new structure. An existing structure already exists in tangible form, so that load and material parameters can be updated by a qualified investigation in situ. What is more, the relative costs of a comparatively small increase of safety reached by technical measures are a lot higher compared to a change in the design process of a new structures (see e.g., Diamantidis et al., 2016b, 2017; Diamantidis & Bazzurro, 2007; SIA, 2011; Steenbergen et al., 2015; Vrouwenvelder, 2002). Thus, the optimization problem that in its general nature characterizes all tasks of design, has to be analyzed in another context. Furthermore, every project is different. Thus, a certain flexibility of the design and verification procedure is needed to be applicable for different kinds of structural tasks. So as manifold as challenges are in practice, as flexible should a framework for the verification be.

Eurocode 0 (DIN EN 1995-1-1:2010-12 (Deutsches Institut für Normung [DIN], 2010)) systemizes reliability methods into deterministic methods (method a), full-probabilistic methods (method b) and semi-probabilistic methods (method c). Full-probabilistic methods are also called "level 3 methods". For the calibration of PSF to be used in a semi-probabilistic design ("level 1 methods") first (or second) order reliability methods (FORM/SORM, "level 2 methods") should be applied. Thus, all methods that are needed to work out an optimal procedure for the evaluation of existing structures, are already provided and classified.

To begin with, an evaluation procedure should match the current design rules that have been approved well in practice and then provide options for its enrichment. The more information is available, the more complex the evaluation format can get. Thus, the first step in the proposed framework is the evaluation in *Knowledge Level 1*, embracing current practice, i.e. a general investigation in situ to exclude major structural damages that may lead to serious stability concerns, and a semi-probabilistic evaluation applying safety elements from current design codes.

Various concerns may lead to the need for a more detailed evaluation of a structure including updated load and material parameters. Reasons might be changes of loads, increased consequences of failure, the (heritage) value of a structures and therefore the interest in its preservation and of course economic constraints that have to be evaluated in the special case. What is more, an update of information that reduces uncertainties concerning the modelling of random variables should be used in the semi-probabilistic evaluation format, that is widely applied in practice. For this purpose, a *Knowledge Level 2* comprising of a subdivision into three sub-levels *KL* 2*a*, *KL* 2*b* and *KL* 2*c* is suggested.

The application of level three formats, i.e. probabilistic methods, requires an experienced engineer and reliable information on probabilistic models. For structures with high consequences of failure and/ or a great interest in their preservation, a probabilistic evaluation would be the format to choose. This could be integrated in the *Knowledge Level 3* evaluation format. The proposal is illustrated in Figure 14.

The choice of the level of detail of the assessment in situ coming along with the extent of parameter update is in fact an optimisation problem. Focus of this paper is the provision of modified PSF for a semi-probabilistic evaluation in *Knowledge Level 2* for a given set of reference situations. For a deeper insight in "level 3" full-probabilistic methods to be applied in *Knowledge Level 3*, the interested reader is referred to the literature (e.g., Benjamin & Cornell, 1970; Cornell, 1969; Ellingwood, 1992; Spanos & Wu, 1993; JCSS, 2001a; Raiffa & Schlaifer, 2000; Spaethe, 1992).

3.2. Calibration of partial safety factors

3.2.1. Method

The general code calibration procedure can be found in Arnbjerg-Nielsen et al. (1996) and has been applied e.g. in Glowienka (2007), Fischer (2010), Stauder (2015) and Baravalle (2017), just to name a few.

According to this procedure, the optimization of PSF is performed based on a set of reference structures designed to a one hundred percent utilization of the semi-probabilistic design check equation. For these structures,

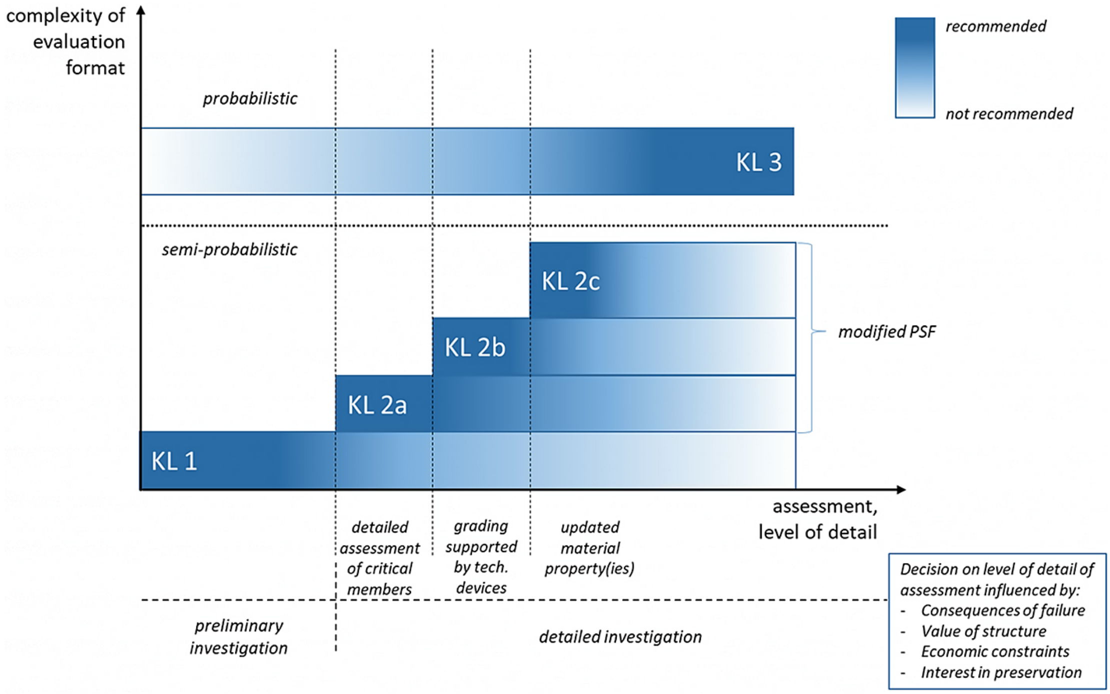

Figure 14. Proposal of evaluation format for the assessment of existing structures

reliability analyses are performed and based on the results a target reliability is set. The optimization is then performed aiming for a minimization of the deviation of single design situation in the set from this target level. This procedure is called *Probabilistic Optimization*, see also ISO (2015), Sørensen (2001) and Vrouwenvelder (2001).

For this work, the procedure is adopted as illustrated in Figure 15. In a first step, a set of reference scenarios has to be chosen. These categories have been selected for this work as given in Figure 2. For this given set of reference scenarios, reliability analyses have been performed.

For the limit states (1)–(9) (Figure 2) and the target reliability levels proposed above the PSF $\gamma_{M,opt}$ for the material strength have been calibrated. The general optimization procedure is given in Eq. (4):

$$\min D(\gamma_M) = \sum_i w_i w_{pen} (\beta_i - \beta_t)^2, \tag{4}$$

with w_i the weighting factor for the LSF and w_{pen} a penalty factor, penalizing a reliability lower than the target more than a higher one (Baravalle & Köhler, 2017). Different penalty functions have been investigated in Baravalle (2017). For this work, penalty factors from Eq. (5) have been applied.

$$w_{pen} = \begin{cases} 1 & \text{for} \quad \beta_i \geq \beta_t \\ 1 + (\beta_i - \beta_t) & \text{for} \quad \beta_i < \beta_t \end{cases}. \tag{5}$$

All considered load combinations of snow and wind load have been weighted equally, as studies indicated that they vary a lot depending on location and roof angle and no clear weighting could be identified (Loebjinski, 2021). For permanent and live load, the weighting factors in Table 9 have been considered and compared to results as-

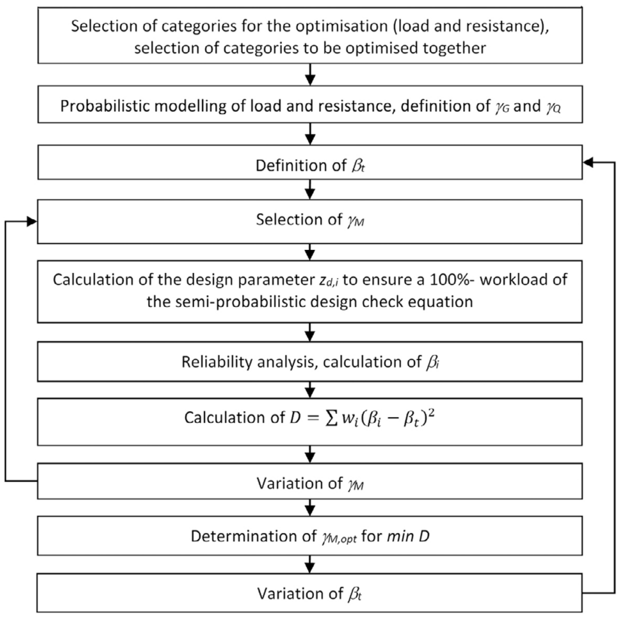

Figure 15. Adopted code calibration procedure

suming $LA_G = 0.5$ and $V_N = 0.25$ only. Results of the three options have turned out to lead to similar results.

During calibration the PSF for permanent and variable loads have been fixed to γ_G = 1.20 and γ_Q = 1.50, respectively. The value γ_G = 1.20 takes into account a reduced variability of permanent loads obtained in a survey on site and is chosen based on SIA 269:2011 (SIA, 2011). A permanent action can commonly be represented by a normal distributed variable. Applying the formulae from EN 1990:2010-12 (CEN, 2010a), Annex C for the characteristic and the design value for normal distributed variables and

Table 9. Weighting factor w_i for permanent and live loads

	V_N	LA_Q	w_i		Remarks
			a	b	
1)	0.25 (A > 20 m²)	0.3	0.50	0.35	*Live load categories office, lobby and living room with A > 20 m², and hotel and class rooms.*
2)	0.25 (A > 20 m²)	0.5	0.40	0.50	*Live load categories office, lobby and living room with A > 20 m², and hotel and class rooms.*
3)	0.40 (A ≤ 20 m²)	0.5	0.10	0.15	*Live load categories office, lobby and living room with A ≤ 20 m², and hospital rooms.*
			Σ = 1	Σ = 1	

$$V_E = \frac{\sigma_E}{m_E} \tag{6}$$

with V_E the coefficient of variation, σ_E the standard deviation and m_E the mean value, the PSF γ_g can be calculated

$$\gamma_g = \frac{E_d}{E_k} = \frac{x_d}{x_k} = \frac{m_E \cdot (1-\alpha_E \beta V_E)}{m_E \cdot \left(1+V_E \cdot \Phi^{-1}(q)\right)} = \frac{1-\alpha_E \beta V_E}{1+V_E \cdot \Phi^{-1}(q)}. \tag{7}$$

For permanent actions, the 50%-fractal is used for the characteristic value in common cases. Thus Eq. (7) becomes

$$\gamma_g = 1-\alpha_E \beta V_E. \tag{8}$$

EN 1990:2010-12 (CEN, 2010a) provides the sensitivity factor with $\alpha_E = -0.7$ for dominating actions and the target reliability with $\beta = 3.8$ for a reference period of 50 years and a consequence class CC2, which is the normal case applied for the majority of standard projects. A quite common assumption for the coefficient of variation for unfavorable permanent actions is $V_E = 0.10$. Please note that also a model uncertainty factor γ_{Sd} has to be considered. For this case, it can also be calculated assuming a normal distribution and the sensitivity factor for accompanying actions given by EC 1990:2010-12 Annex C $\alpha = -0.4$. Thus, the total PSF becomes $\gamma_{G,sup} = 1.36 \approx 1.35$, the value also given in EN 1990.

SIA 269:2011 (SIA, 2011) for existing structures allows to reduce γ_G if permanent actions are updated by a qualified survey on site. What is more, Annex B of SIA 269 offers options to reduce the target reliability under economic considerations. A realistic option could be to apply $\Delta â = -0.5$, see also Diamantidis et al. (2007). So, if it is assumed that the cov can be reduced to e.g. 7% by a detailed survey, which is a good value for e.g. historic timber floor structures as investigated in Loebjinski (2021) and a target reliability of $\beta = 3.2$, the PSF becomes $\gamma_{G,sup} = 1.23 \approx 1.20$ as given in SIA 269:2011 (SIA, 2011). This is considered in this study, as a reduction of the PSF on the load side has an impact on the resistance side in the probabilistic optimization.

The results of the reliability analyses are based on a load ratio of permanent loads of LAG = 0.5 and further statistical parameters given in Table 1.

3.2.2. Results

Table 10 and Table 11 show the results for the suggested reliability index for the evaluation level ($\beta_{t,eval}$ = 2.9, T_{ref} = 50 years). In Loebjinski (2021) results for further reliability indices can be found.

Table 10. Calibration results $\gamma_{M,opt}$ for the proposed evaluation level and uniaxial stresses

	permanent load (1)	live load (2)	snow load (3)	wind load (4)	snow + wind load (5)
V_R	$\gamma_{M,opt}$	$\gamma_{M,opt}$	$\gamma_{M,opt}$	$\gamma_{M,opt}$	$\gamma_{M,opt}$
0.18	1.14	1.19	1.19	1.11	1.15
0.20	1.16	1.19	1.19	1.13	1.17
0.22	1.18	1.21	1.21	1.15	1.19
0.25	1.22	1.23	1.23	1.19	1.22

Table 11. Calibration results $\gamma_{M,opt}$ for the proposed evaluation level and simple stress combinations

	two-axial bending (6)	two-axial bending (7)	comp. + bending (8)	comp. + bending (9)
$V_{R,m}$	$\gamma_{M,opt}$	$\gamma_{M,opt}$	$\gamma_{M,opt}$	$\gamma_{M,opt}$
0.18	1.15	1.16	1.19	1.18
0.20	1.17	1.17	1.20	1.19
0.22	1.19	1.19	1.21	1.20
0.25	1.22	1.22	1.23	1.22

Note: $V_{R,m}$ – coefficient of variation (cov) of the bending strength. cov of compression strength correlated by ρ = 0.8.

3.2.3. Summary and proposal

The choice of an optimal set of PSF is influenced by the optimization potential and by practical applicability. A high number of PSF contributes to an optimized utilization of the material stress. However, in practical application a higher number of PSF increases calculation efforts (Figure 16).

Thus, keeping in mind practical applicability, the number of PSF should be limited. Thus, a simplified set of optimized PSF $\gamma_{M,opt}$ is proposed that considers the inhomogeneity and anisotropy of timber as structural material but keeps the intended ease of use of the semi-probabilistic design / evaluation format, (Table 12). As γ_Q = 1.5 for all variable loads, $\gamma_{M,opt}$ also depends on the considered loads. The application of different PSF for different variable loads to consider their statistical properties properly would help to reach a more uniform reliability level. Please note that requirements according to Table 13 have to be considered.

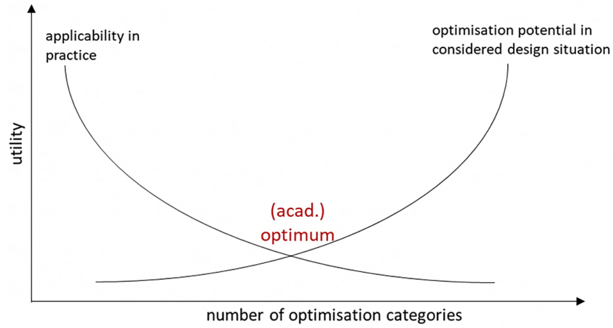

Figure 16. Principle Illustration of complementary targets in the calibration of optimised PSF

In order to apply the results (Table 12) for an updated material variability from material tests, a sufficient number of tests has to be considered. A proposal in (Fischer, 2010) introduces a conversion factor to account for a limited number of tests. For rehabilitation measures PSF as for new structures should be applied in accordance with e.g. (Fachkommission Bautechnik der Bauministerkonferenz [ARGEBAU], 2008) where it is stated that structural members directly affected by changes should be verified applying the same requirements as for new structural members.

3.2.4. Verification of PSF

The verification of results is an important part of scientific work. While the calibration of modified PSF has been performed with *FORM*, the verification has been performed by *MSC* (crude *Monte Carlo Sampling*) to avoid systematic programming errors. Results have been double checked with *FORM* analyses. Exemplary results are presented in Figure 17 to Figure 20(graphs produced applying *FORM* after double check with *MCS*). All analyses are performed for T_{ref} = 50 years. All figures are based on a 100% utilisation of the semi-probabilistic design-check equation. PSF are $\gamma_{G,up}$ = 1.20, γ_Q = 1.50, $\gamma_{M,opt}$ = 1.20.

Neglectable differences between results of FORM and MCS have been identified. Thus, for the investigated limit states a sufficient accuracy of the approximation by FORM is given and systematic errors are not present.

The results show that the calibration target $\beta_{t,eval}$ = 2.9 (indicated by blue horizontal line) has kept in the majority of analyses. The scatter results from the selection of a limit number of PSF. Thus, the calibration has been successful for the chosen target. However, the target reliability index is chosen based on the analyses presented and needs further discussion by the scientific community and relevant authorities. Results for further targets in the range $2.5 \leq \beta \leq 3.2$ can be found in (Loebjinski, 2021).

4. Update of partial safety factors based on an update of material properties

A great potential when analyzing the load-bearing capacity of an existing structure is that it exists in tangible form. Updated material models can be built combining on-site tests and analyses in the laboratory, see e.g. (Sousa et al., 2015a). Different options for the consideration of updated material properties in the semi-probabilistic safety format can be described. One is the application of an improved strength grading supported by technical means, see e.g. (Linke et al., 2022). This is, however, not in the focus of this work.

Table 12. Proposal for a set $\gamma_{M,opt}$ for the evaluation level

stress	loads	
	permanent	combinations with variable loads
compression parallel to grain	1.15	1.20
bending, two-axial bending, comp.& bending	1.20	1.20
tension parallel to grain	1.30	1.30
Remarks: Calibration for $\beta_{t,eval}$ = 2.9 (T_{ref} = 50 years), live load categories A and B, snow and wind loads, load ratio of permanent load $LA_G \geq 0.5$, $\gamma_{G,up}$ = 1.20, γ_Q = 1.50.		

Table 13. Requirements for the application of modified PSF in Knowledge Level (KL) 2a

Requirements for the application of modified PSF for the evaluation of existing timber structures in (KL) 2a
The structure has been designed for a minimum service life of 50 years
The structure has been used as intended for a minimum period of 5 years
The structure is free of damages
Permanent loads, residential/office live loads, snow and/ or wind loads have to be considered
The load ratio of permanent loads to total loads is $LA_G \geq 0.5$.
Geometric parameters and permanent loads are updated by a detailed investigation in situ
The minimum sample size to update material properties is $n_{min} = 5$
If an updated cov of the material strength is used to apply updated PSF other than given in Table 1 a conversion factor to consider a limited sample size for n < 30 has to be considered

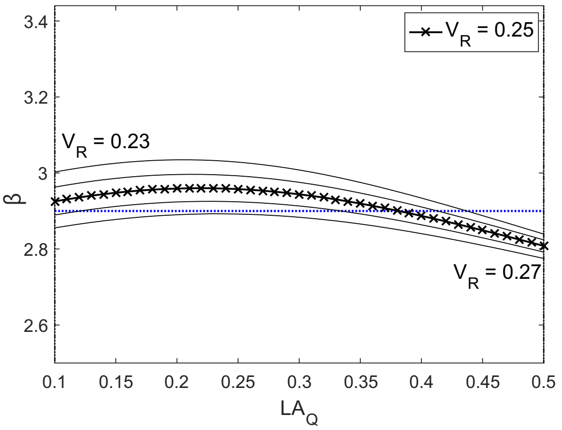

Figure 17. Reliability index for uniaxial stress, permanent and one variable load with $V_Q = 0.25$

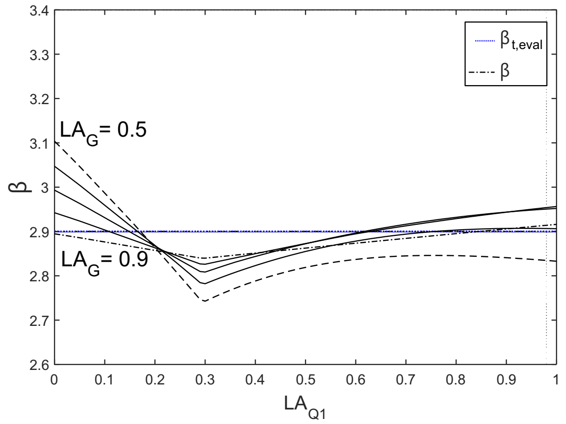

Figure 19. Reliability index for two-axial bending, permanent, snow and wind load, $V_R = 0.25$

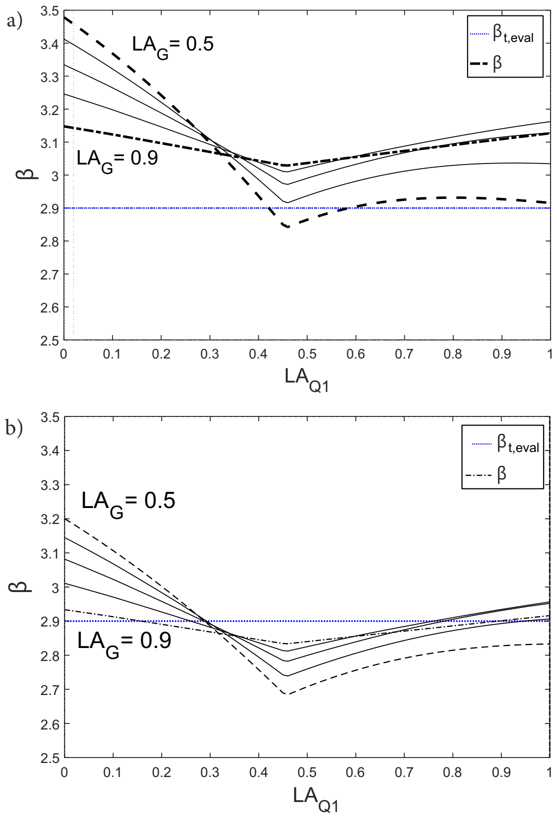

Figure 18. Reliability index for uniaxial stress, permanent, snow and wind load, a) $V_R = 0.20$, b) $V_R = 0.25$

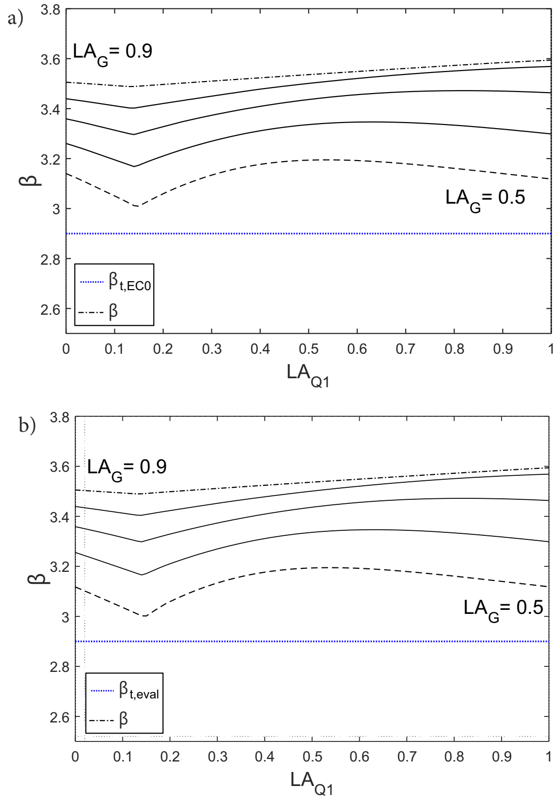

Figure 20. Reliability index for compression and bending, permanent, snow and wind load, a) $V_{R,c} = 0.20$, b) $V_{R,m} = 0.25$

Another option is the consideration of reduced uncertainties due to a detailed investigation in situ by adjusted safety elements. Here also two options can be discussed. The first one is the modification of PSF based on a reduced material variability by direct update of the variable. Adjusted PSF can be calculated applying the *Design Value Method* (DVM) / *Design Value Format* (DVF) or the *Adjusted Partial Safety Factor Method* (APFM) as described in (FIB, 2016). Additional to knowledge concerning the special material variability at hand, a target reliability that has been agreed upon and sensitivity factors are needed. Simplified sensitivity factors can be taken from EN 1990:2010-12 (CEN, 2010a). However, sensitivity factors are factors considering the influence of the change of a certain variable in the limit state on the reliability. Thus, the application of simplified sensitivity factors ignores the advantage of the availability of more detailed information concerning the actual load and material properties at hand and the governing limit state. Hence, updated sensitivity factors for certain limit states as outcome of the presented reliability analyses can be applied. Note that these sensitivity factors are only applicable if updated PSF are also applied for the loads to keep the intended reliability.

The second option is the update of the PSF on the material side using an updated reference variable. A proposal is described in Loebjinski et al. (2019a) and presented shortly hereinafter.

The mean value $\mu_{y|x_{meas}}$ and the standard deviation $\sigma_{y|x_{meas}}$ of a target property y dependent on a reference variable x are defined by Eq. (9) and (10), respectively, see Köhler (2011). $\mu_{y,code}$ and $V_{y,code}$ are the mean value and the *cov* of the target variable as defined in prior information (e.g. a code), respectively. $\mu_{x,code}$ and $V_{x,code}$ are the mean value and the *cov* of the reference variable as defined by prior information, $\rho_{x,y}$ is the correlation coefficient.

$$\mu_{y|x_{meas}} = \mu_{y,code} \cdot \left(1 + \rho_{x,y} \cdot V_{y,code} \cdot \frac{x_{meas} - \mu_{x,code}}{\mu_{x,code} \cdot V_{x,code}}\right); \tag{9}$$

$$\sigma_{y|x_{meas}} = V_{y,code} \cdot \mu_{y,code} \cdot \sqrt{1 - \rho_{x,y}^2} \ . \tag{10}$$

Eq. (11) is derived from Eq. (9) and (10).

$$V_{y|x_{meas}} = \frac{\sigma_{y|x_{meas}}}{\mu_{y|x_{meas}}};$$

$$V_{y|x_{meas}} = \frac{V_{y,code} \cdot \sqrt{1 - \rho_{x,y}^2}}{\left(1 + \rho_{x,y} \cdot V_{y,code} \cdot \frac{x_{meas} - \mu_{x,code}}{\mu_{x,code} \cdot V_{x,code}}\right)}, \tag{11}$$

with Eq. (12) for the PSF of a lognormal distributed resistance variable, the updated PSF $\gamma_{m,up}$ can be calculated according to Eq. (13). Here α_R is the sensitivity factor of the resistance variable, β is the target reliability and q is the quantile of the distributed used to define the characteristic value for semi-probabilistic design.

$$\gamma_m = exp\left(V_R \cdot \left(\alpha_R \cdot \beta + \Phi^{-1}(q)\right)\right); \tag{12}$$

$$\gamma_{m,up} = exp\left(\frac{V_{y,code} \cdot \sqrt{1 - \rho_{x,y}^2}}{\left(1 + \rho_{x,y} \cdot V_{y,code} \cdot \frac{x_{meas} - \mu_{x,code}}{\mu_{x,code} \cdot V_{x,code}}\right)} \cdot \left(\alpha_R \cdot \beta + \Phi^{-1}(q)\right)\right). \tag{13}$$

The model uncertainty factor γ_{Rd} is considered by Eq. (14):

$$\gamma_{M,up} = \gamma_{Rd} \cdot \gamma_{m,up}. \tag{14}$$

Application examples can be found in Köhler (2011), Loebjinski et al. (2019b). Please note that uncertainties determining the reference variable, e.g. measurement errors are not considered in this approach. An option is the consideration of an additional error term as described in Pöhlmann and Rackwitz (1981) and left to further work.

5. Summary of the stepwise evaluation procedure

An evaluation procedure for existing structures needs to be flexible in terms of applicability for the actual circumstances at hand. That includes availability of information, consequences of failure and economic considerations. Thus, a procedure is needed that embraces levels with lower degree of information and an evaluation more on the safe side and different options to include updated information that comes along with higher efforts in terms of time and costs. A proposal has been presented in Loebjinski et al. (2020). The stepwise evaluation procedure embraces three *Knowledge Levels* (terminology based on Joint Research Centre (2015)) including an evaluation without update (*KL 1*), a level including a modified semi-probabilistic evaluation divided into three sublevels (*KL 2*) and a level for advanced probabilistic methods (*KL 3*), see Table 14. With increasing level, the information becomes more detailed as well as the evaluation format does. These levels are connected to a proposal for strength grading levels (*SGL*), see Linke et al. (2022). An application example can e.g. be found in Loebjinski et al. (2019b).

Table 14. Proposal for a stepwise evaluation procedure

<table>
<tr><td colspan="3" align="center">semi-probabilistic <u>without</u> update – KL 1</td></tr>
<tr><td colspan="3" align="center">visual grading in situ</td></tr>
<tr><td colspan="3" align="center">partial safety factors from
EN 1990, EN 1995-1-1, EN 1995-1-1/NA</td></tr>
<tr><td colspan="3" align="center">semi-probabilistic <u>with</u> update – KL 2</td></tr>
<tr><td align="center">KL 2a)</td><td align="center">KL 2b)</td><td align="center">KL 2c)</td></tr>
<tr><td colspan="3" align="center">update of permanent loads: $\gamma_{G,up}$ = 1.20 (unfavourable)</td></tr>
<tr><td colspan="3" align="center">variable loads: γ_Q = 1.50</td></tr>
<tr><td>visual grading in situ</td><td>grading supported by tech. devices</td><td>update of material parameters</td></tr>
<tr><td>strength grade: visual grading</td><td>strength grade: grading supported by tech. devices</td><td>update of material properties by material tests</td></tr>
<tr><td colspan="2">PSF on resistance side, optimised $\gamma_{M,opt}$ for actual limit states</td><td>PSF on resistance side for updated property, $\gamma_{M,up}$</td></tr>
<tr><td colspan="3" align="center">probabilistic – KL 3</td></tr>
<tr><td colspan="3">grading supported by technical devices / update of material parameters</td></tr>
<tr><td colspan="3">probabilistic evaluation by approximation or simulation
update of material model based on material tests</td></tr>
</table>

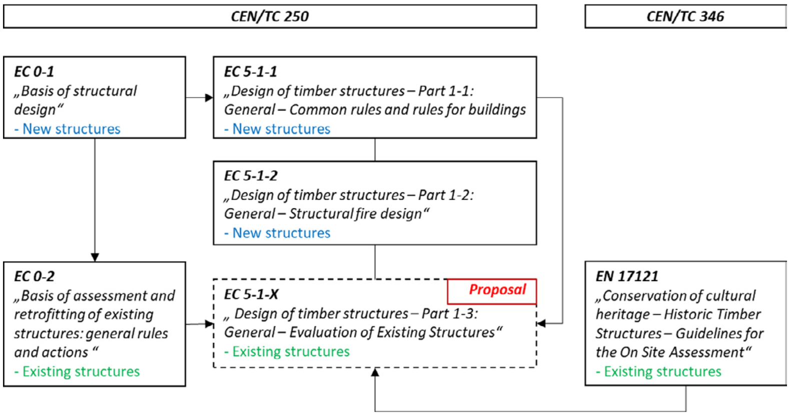

Figure 21. Proposal for the integration of the evaluation of existing timber structures in the concept of current Eurocodes

Please note that the calibration results for $\gamma_{M,opt}$ are only applicable under the given requirements and the intended limit states and load ratios. As results are given depending on the *cov* of the material strength, an updated of material properties leading to e.g. a reduction of the scatter can be considered by choosing the results for the adjusted value. Note that for the update of material properties a sufficient number of tests and strong correlation of target and reference variable are required. As a first orientation, correlation coefficients should be $\rho > 0.6$. For applicability in practice, the proposal needs to be integrated in the current system of Eurocodes. Options are: a new chapter in existing codes, an annex to an existing code or the development of a new code. Figure 21 illustrates a proposal.

Conclusions

A responsible use of energy and resources is an important part of the development of a sustainable economy. The preservation of existing structures plays a central role in this challenge that has never been more important than today. Thus, code calibration cannot only focus on the necessary further development of codes in the context of the design of structures and the application of innovative structural materials. The evaluation of existing structures needs to be integrated in the concept of current Eurocodes considering the special requirements that come along with the challenging tasks that are related to this interesting work. In this context, rules and requirements need to be flexible in a way so that they can be adopted for different structures. Thus, the evaluation of the load-bearing capacity of existing structures requires a modified concept to consider different levels of information. The decision on the level of information aspired (*Knowledge Level*) is influenced by numerous factors such as the consequences of failure, the value of the structure, economic constraints, the public interest in its preservation and may be very individual.

A proposal for a stepwise evaluation procedure considering different levels of information has been worked out for structural timber in existing structures and presented. Part of this evaluation procedure are modifications of the partial safety factors as the semi-probabilistic verification format is the one mainly used in practice. These modifications of PSF are on the one hand side based on a probabilistic optimization for chosen limit states and on the other hand side based on a parameter update on the resistance side. For the probabilistic optimization a reduction of parameter uncertainty for permanent actions has been considered and results are given for a range of cov of the material strength and target reliabilities. Recommendations for chosen values are given and can be summarized with $\gamma_{M,opt}$ = 1.20 for compression and flexural strength in limit states, where variable actions are present. Considering high statistical variability, the tension strength of structural timber is advised to be considered with a PSF $_{M,opt}$ = 1.30. Regarding modifications due to the updated material parameters, no definite values can be given, as structural properties are significantly influenced by the specific material used in the actual project at hand. Thus, this study presents methodologies for updating PSF using statistical methods based on updated material properties.

However, more studies considering further and more complex limit states and further detailing concerning the update of target properties based on in situ measured reference properties have to carried out to enlarge the concept for a wider range of practical cases. A reduction of parameter uncertainty can then be used within the update of PSF to be used for structural evaluation. Besides, further studies need to embrace the reliability of historic connections, also carpenters' connections. What is more, new information on the modelling of variable actions should be considered, see e.g., Holický and Sýkora (2016), McAllister et al. (2018) for wind loads. However, as the target value for the calibration has been defined based on this calculated implicit level, the evaluation of the influence on the calibration of PSF is left to further work. However, as a central task in the development of codes and standards, it has to be discussed how the target reliability has to be defined for existing structures.

Acknowledgements

A major part of the content has been developed in a funding period of The German Federal Environmental Foundation (DBU, project No. 20015/409, 2016–2018). The authors thank the institution for the generous support.

Author contributions

WR was responsible for the conception and first data collection, ML was responsible for detailed preparation of the analyses and wrote the first draft of the manuscript, ML and ZL where responsible for the data analysis and the illustrations, ML, WR and HP were responsible for the interpretation of results.

Disclosure statement

It is declared, that no competing financial, professional, or personal interests from other parties exist.

References

Arnbjerg-Nielsen, T., Karlsson, M., & Ditlevsen, O. (1996). Calibration of partial safety factors. In IABSE (Ed.), *Colloquium Delft. Basis of Design and Actions on Structures. Background and application of Eurocode 1* (pp. 243–250).

Baravalle, M. (2017). *Risk and reliability based calibration of structural design codes* [Dissertation]. Norwegian University of Science and Technology, Trondheim.

Baravalle, M., & Köhler, J. (2017). A framework for estimating the implicit safety level of existing design codes. In *12th International Conference on Structural Safety and Reliability* (*ICOSSAR*) (pp. 1037–1046). Vienna.

Benjamin, J. R., & Cornell, C. A. (1970). *Probability, statistics and decision for civil engineers*. McGraw-Hill, Inc.

CIB. (1989). *Actions on structures: Live loads in buildings* (CIB Report No. W81).

Cornell, C. A. (1969). A probability-based structural code. *ACI Journal*, *66*(12), 974–985. https://doi.org/10.14359/7446

Deutsches Institut für Normung. (2010, Dezember). *Eurocode 5: Bemessung und Konstruktion von Holzbauten – Teil 1-1: Allgemeines – Allgemeine Regeln und Regeln für den Hochbau* (Deutsche Fassung EN 1995-1-1:2004 + AC:2006 + A1:2008). Beuth Verlag.

Diamantidis, D., Holický, M., & Jung, K. (2007). Assessment of existing structures – On the applicability of the JCSS recommendations. In M. H. Faber, T. Vrouwenvelder, & K. Zilch (Eds.), *Aspects of structural reliability* (pp. 15–18). Herbert Utz Verlag.

Diamantidis, D., & Bazzurro, P. (2007). Safety acceptance criteria for existing structures. In *Conference: Workshop on Risk Acceptance and Risk Communication*, Stanford University, USA.

Diamantidis, D., Holický, M., & Sýkora, M. (2016a). Reliability and risk acceptance criteria for civil engineering structures. *Transactions of the VŠB – Technical University of Ostrava. Civil Engineering Series*, *16*(2), 1–10. https://doi.org/10.1515/tvsb-2016-0008

Diamantidis, D., Holický, M., & Sýkora, M. (2016b). Risk and reliability acceptance criteria for civil engineering structures. In *Conference: Structural Reliability and Modelling in Mechanics*. Ostrava, Czech Republic.

Diamantidis, D., Holický, M., & Sýkora, M. (2017). Target reliability levels based on societal, economic and environmental consequences of structural failure. In *12th International Conference on Structural Safety and Reliability* (*ICOSSAR*) (pp. 644–653). Vienna.

Ellingwood, B. R. (1992). Status of reliability-based design in North America: Impact for engineered wood constructions. In J. Bodig (Ed.), *NATO ASI series E, applied sciences: Vol. 215. Reliability-based design of engineered wood structures*. [*Proceedings of the NATO Advanced Research Workshop on Reliability-Based Design of Engineered Wood Structures, Florence, Italy, 2–5 June 1991*] (pp. 3–19). Kluwer. https://doi.org/10.1007/978-94-015-8044-1_2

European Committee for Standardization. (2010a, December). *Eurocode – Basis of structural design* (EN 1990:2010-12)

European Committee for Standardization. (2010b, December). *Eurocode 5: Design of timber structures – Part 1-1: General – Common rules and rules for buildings* (EN 1995-1-1:2010-12).

European Committee for Standardization. (2019). *Conservation of cultural heritage. Historic timber structures. Guidelines for the on-site assessment of load-bearing timber structures* (CEN Standard No. EN 17121:2019-12).

European Committee for Standardization. (2020, October). *Assessment and retrofitting of existing structures* (CEN Standard No. CEN/TS 17440:2020).

Fachkommission Bautechnik der Bauministerkonferenz (ARGEBAU). (2008). *Hinweise und Beispiele zum Vorgehen beim Nachweis der Standsicherheit beim Bauen im Bestand*. Retrieved January 15, 2016, from https://www.bauministerkonferenz.de/IndexSearch.aspx?method=get&File=bya892ba82y1b9bbba8a4a8yb9bb92b8y9ya8ayyb9y884b992a2a0a14948a4a1a14b80b8y0wfzrvwai1oioqdtn04mspnwt

Ferry Borges, J., & Castanheta, M. (1971). *Structural safety* (2 ed.). Lisbon.

Fédération internationale du betón. (2016). *Partial factor methods for existing concrete structures*. Recommendation (fib bulletin, vol. 80). Lausanne, Switzerland.

Fischer, A. M. (2010). *Bestimmung modifizierter Teilsicherheitsbeiwerte zur semiprobabilistischen Bemessung von Stahlbetonkonstruktionen im Bestand* [Dissertation]. Technische Universität Kaiserslautern. Retrieved March 29, 2016, from https://kluedo.ub.uni-kl.de/frontdoor/index/index/docId/2259

Glowienka, S. (2007). *Zuverlässigkeit von Mauerwerkswänden aus großformatigen Steinen: Probabilistische Analyse von großformatigem Mauerwerk aus Kalksandstein und Porenbeton mit Dünnbettvermörtelung* [Dissertation]. Technische Universität Darmstadt.

Grünberg, J. (2004). *Grundlagen der Tragwerksplanung – Sicherheitskonzept und Bemessungsregeln für den konstruktiven Ingenieurbau: Erläuterungen zu DIN 1055-100* (1. Aufl.). *Praxis Bauwesen*. Beuth.

Hasofer, A. M., & Lind, N. C. (1974). An exact and invariant first-order reliability format. *Journal of the Engineering Mechanics Division*, *100*(1), 111–121. https://doi.org/10.1061/JMCEA3.0001848

Holický, M., & Sýkora, M. (2016). Probabilistic models for wind actions. In *2016 Second International Symposium on Stochastic Models in Reliability Engineering, Life Science and Operations Management* (*SMRLO*). IEEE. https://doi.org/10.1109/SMRLO.2016.38

Hösl, M., & Dietsch, P. (2010). Assessment of timber structures. In H. Brüninghoff & P. Dietsch (Eds.), *Berichte aus dem Bauwesen. Assessment of timber structures. COST action E55 – Modelling of the performance of timber structures* (pp. 14–21). Shaker Verlag.

International Organization for Standardization. (2010, August 01). *Bases for design of structures – Assessment of existing structures* (ISO Standard No. ISO 13822:2010).

International Organization for Standardization. (2015). *General principles on reliability for structures* (ISO Standard No. ISO 2394:2015).

Spanos, P. D., & Wu, I. T. (Eds.) (1993). *Probabilistic structural mechanics*: *Advances in structural reliability methods*. IUTAM Symposium, San Antonio, Texas, USA. Springer. https://doi.org/10.1007/978-3-642-85092-9

Joint Committee on Structural Safety. (2001a). *Probabilistic Model Code*: *Part 1 – Basis of Design*. Retrieved March 07, 2016, from https://www.jcss-lc.org/jcss-probabilistic-model-code/

Joint Committee on Structural Safety (2001b). *Probabilistic Model Code*: *Part 2 – Load Models*. Retrieved March 29, 2016, from https://www.jcss-lc.org/jcss-probabilistic-model-code/

Joint Committee on Structural Safety. (2006). *Probabilistic Model Code*: *Part 3 – Resistance Models*. Retrieved March 08, 2016, from https://www.jcss-lc.org/jcss-probabilistic-model-code/

Joint Research Centre. (2015). *New European technical rules for the assessment and retrofitting of existing structures* (EUR, Scientific and technical research series). European Commission, Luxembourg.

Kasal, B., & Tannert, T. (2011). *In situ assessment of structural timber*. Springer Netherlands. https://doi.org/10.1007/978-94-007-0560-9

Köhler, J. (2010). Quantitative assessment – updating. In H. Brüninghoff & P. Dietsch (Eds.), *Berichte aus dem Bauwesen. Assessment of timber structures. COST action E55 – Modelling of the performance of timber structures* (pp. 120–134). Shaker Verlag.

Köhler, J. (2011). Die Aktualisierung als zentrales Element in den Erhaltungsnormen – Aspekte der Probabilistik. In H. Bahnholzer (Ed.), *Dokumentation SIA*: *D 0240. Erhaltung von Tragwerken – Vertiefung und Anwendung. Unterlagen zu den Einführungskursen* (pp. 33–36). Zürich.

Kotlínová, M., Kloiber, M., Vasconcelos, G., Lourençο, P. B., & Branco, J. M. (2008). Evaluation of wood density by means of distinct NDT. In L. Binda, M. di Prisco, & R. Felicetti (Eds.), *On Site Assessment of Concrete*, *Masonry and Timber Structures – SACoMaTiS 2008* (pp. 1061–1070). RILEM Publications SARL.

Linke, G., Rug, W., & Pasternak, H. (2022). Strength grading of timber in historic structures – methodology and practical application. In *6th International Conference on Structural Health Assessment of Timber Structures* (*SHATiS*) (pp. 63–68).

Loebjinski, M. (2021). *Bewertung der Tragfähigkeit von Holzkonstruktionen beim Bauen im Bestand*: *Ein Beitrag zur substanzschonenden Erhaltung von bestehenden Gebäuden* [Dissertation]. Brandenburgische Technische Universität, Cottbus. http://d-nb.info/1241545375/34

Loebjinski, M., Köhler, J., Rug, W., & Pasternak, H. (2019a). Development of an optimisation-based and practice orientated assessment scheme for the evaluation of existing timber structures. In *6th International Symposium on Life-cycle Analysis and Assessment in Civil Engineering* (*IALCCE*) (pp. 353–360). CRC Press.

Loebjinski, M., Linke, G., Rug, W., & Pasternak, H. (2019b). Redevelopment of a wooden roof construction under preservation order. In *5th International Conference on Structural Health Assessment of Timber Structures* (*SHATiS*) (pp. 912–921). Guimarães, Portugal.

Loebjinski, M., Rug, W., & Pasternak, H. (2020). The influence of improved strength grading in situ on modelling timber strength properties. *Buildings*, *10*(2), Article 30. https://doi.org/10.3390/buildings10020030

MathWorks. (2016). MATLAB. https://www.mathworks.com/products/matlab.html

McAllister, T. P., Wang, N., & Ellingwood, B. R. (2018). Risk-Informed mean recurrence intervals for updated wind maps in ASCE 7-16. *Journal of Structural Engineering*, *144*(5), Article 6018001. https://doi.org/10.1061/(ASCE)ST.1943-541X.0002011

Melchers, R. E. (1999). *Structural reliability analysis and prediction* (2. ed.). Wiley.

Palaia, L., Monfort, J., Sanchez, R., Gil, L., Álvarez, Á., López, V., et al. (2008). Ancient timber structure analysis applying NDT and traditional methods of assessment. In L. Binda, M. di Prisco, & R. Felicetti (Eds.), *On Site Assessment of Concrete*, *Masonry and Timber Structures – SACoMaTiS 2008* (pp. 1135–1143). RILEM Publications SARL.

Piazza, M., & Riggio, M. (2008). NDT methods for the assessment of structural timber: Report on the research carried out at the University of Torento (Italy). In L. Binda, M. di Prisco, & R. Felicetti (Eds.), *On Site Assessment of Concrete*, *Masonry and Timber Structures – SACoMaTiS 2008* (pp. 1039–1047). RILEM Publications SARL.

Pöhlmann, S., & Rackwitz, R. (1981). Zur Verteilungsfunktion von Werkstoffeigenschaften bei kontinuierlich durchgeführten Sortierungen. *Materialprüfung*, *23*(8), 277–278. https://doi.org/10.1515/mt-1981-230808

Raiffa, H., & Schlaifer, R. (2000). *Applied statistical decision theory*. Wiley.

Saporiti Machado, J., Riggio, M., & Descamps, T. (Eds.) (2015). *Combined use of NDT/SDT methods for the assessment of structural timber members. State of the art report*. UMONS – Université de Mons.

Schweizerischer Ingenieur- und Architektenverein. (2011). *Grundlagen der Erhaltung von Tragwerken* (SIA 269:2011). Zürich.

Sørensen, J. D. (2001). *Code calibration and timber experience* (COST Action E24). Institute of Building Technology and Structural Engineering, Aalborg University, Aalborg.

Sousa, H. S. (2013). *Methology for Safety evaluation of existing timber elements* [Dissertation]. Universidade do Minho Escola de Engenharia.

Sousa, H. S., Branco, J. M., & Lourenço, P. B. (2015a). A Holistic methodology for probabilistic safety assessment of timber elements combining onsite and laboratory data. *International Journal of Architectural Heritage*, *10*(5), 526–538. https://doi.org/10.1080/15583058.2015.1007177

Sousa, H. S., Machado, J. S., Branco, J. M., & Lourenço, Paulo, B. (2015b). Onsite assessment of structural timber members by means of hierarchical models and probabilistic models. *Construction and Building Materials*, *101*, 1185–1196. https://doi.org/10.1016/j.conbuildmat.2015.05.127

Spaethe, G. (1992). *Die Sicherheit tragender Baukonstruktionen* (2nd ed.). Springer-Verlag. https://doi.org/10.1007/978-3-7091-6690-1

Stauder, F. (2015). *Zuverlässigkeitskonzept für bestehende Tragwerke im Wasserbau* [Dissertation]. Technische Universität Kaiserslautern, Kaiserslautern.

Steenbergen, R., Sýkora, M., Diamantidis, D., Holický, M., & Vrouwenvelder, T. (2015). Economic and human safety reliability levels for existing structures. *Structural Concrete*, *16*(3), 323–332. https://doi.org/10.1002/suco.201500022

Turkstra, C. J. (1970). *Theory of Structural Design Decisions*: *Study No. 2*. Ontario, Canada.

Vrouwenvelder, T. (2001). *Reliability based code calibration – The use of the JCSS probabilistic model code*. JCSS, Workshop on Code Calibration, Zürich.

Vrouwenvelder, T. (2002). Developments towards full probabilistic design codes. *Structural Safety*, *24*, 417–432. https://doi.org/10.1016/S0167-4730(02)00035-8