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

**Maria Loebjinski[1], Wolfgang Rug[2/3], Hartmut Pasternak[4]**

**ABSTRACT:** The evaluation of the load-bearing capacity of existing structures is a central and important part in the work of structural engineers. At state, 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. This task needs to be addressed in the development of common codes and standards. As a contribution, 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, a modification of the target reliability for existing structures is discussed and a suggestion for different levels is given. In another step, this contribution presents a proposal for a stepwise procedure for the evaluation of the load-bearing capacity of structural timber in existing structures considering different levels of available information. Focus is set on the provision of a flexible semi-probabilistic evaluation concept; modified partial safety factors are calibrated on the resistance side for selected limit states. Additionally, options to consider updated material parameters from a survey on site supported by technical devices are discussed and further need for research is identified.

**KEYWORDS:** existing timber structures, evaluation, partial safety factor, code calibration

## 1 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 structures. The newly introduced Technical Specification *CEN/TS 17440:2020-10* [1] 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* [2] or may be defined in National Annexes. As the target reliability is defined based on an optimisation of failure consequences and efficiency of safety measures (see *ISO 2394:2015* [3] or *SIA 269/2011* [4]), an altered definition for existing structures is possible. Annex C of [1] 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* [1] 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. 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* [5] and with special focus on existing timber structures *EN 17121:2019* [6] provide the basis for a qualified assessment. Updated information from an assessment on site should be considered in the modified semi-probabilistic evaluation. Results are always specific for the structure at hand what complicates standardisation. Thus, a structured process including options to modify safety elements is needed to systematize the use of updated information in the evaluation.

## 2 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 [7], safety elements are defined by optimisation. What is more, [8] 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

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built structures satisfy public safety requirements [9]. The reliability index $\beta$ 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: Timber members in common limit states are designed for a 100% utilisation of the semi-probabilistic design-check equations with current partial safety factors (PSF) from EC 1 [10] and EC 5 [11] ($\gamma_G = 1.35$ for permanent loads, $\gamma_Q = 1.5$ for variable loads, $\gamma_M = 1.3$ for structural timber strength properties) and evaluated by reliability analyses. 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, PSF are calibrated for chosen limit states aiming for an optimised verification of existing timber structures with dominating limit states clearly defined. Besides, options to update PSF based on new information are discussed. Finally, a proposal for a stepwise evaluation procedure is presented. Selected limit states for the analyses are given in Figure 1. The numbering is used in the presentation of results.

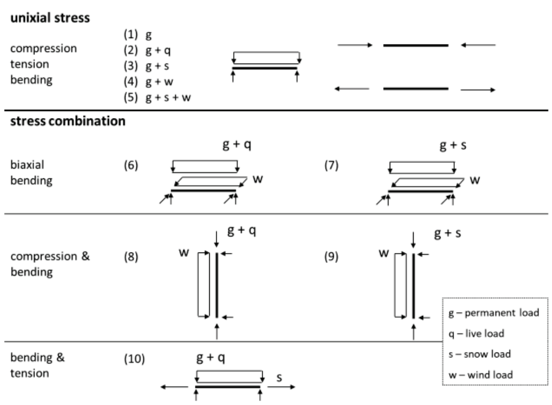


***Figure 1:*** *Loads, load directions and stresses*

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

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

with $R$ the resistance variable and $\theta_G$ its model uncertainty, $G$ the permanent action and $\theta_G$ the model uncertainty associated with it. $Q_1$ and $Q_2$ are two variable actions, $\theta_{Q1}$ and $\theta_{Q2}$ 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 [12]. What is more, $z_d$ is the design parameter to ensure a one-hundred percent utilisation of the semi-probabilistic design equation, see [13]. It can be calculated by eq. (2) for one an eq. (3) for two variable actions

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

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

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 [11]. Probabilistic parameters are given in Table 1. The *cov's* of the material strength are based on [14] and a broad literature study in [15]. The influence of alterations of the material *cov* are studied. *Cov's* for live loads are based on own calculations applying [16, 17], see [15]. Parameters for snow and wind loads are based on [17] (wind) and [18] (snow). Load change rates have been oriented on [13] 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 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* [18] and the *JCSS PMC* [14, 17].

***Table 1:*** *Probabilistic parameters,* $T_{ref} = 50\ years$

| | | Variable | | Distr. | $\mu$ | $V$ |
|---|---|---|---|---|---|---|
| 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 ($A \leq 20m^2$) | | $N$ | GUM | 1.00 | 0.40 |
| | Large room ($A > 20m^2$) | | | | 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** | | $\theta_f$ | N | 1.00 | 0.07 |
| | Load | Permanent load | $\theta_G$ | N | 1.00 | 0.05 |
| | | Live load | $\theta_N$ | N | 1.00 | 0.10 |
| | | Snow load[6] | $\theta_S$ | N | 1.00 | 0.10 |
| | | Wind load[6] | $\theta_W$ | N | 1.00 | 0.10 |

*[1] Indicative from [14], analyses for a range of values; [2] Based on [14]; [3] $T_{ref}$ = 50a, based on own calculations; [4] $T_{ref}$ = 50a, based on [13, 17, 18]; [5] Multiplicative, attached to variable*

The reliability analyses are performed by *First Order Reliability Method (FORM)* [19] in MATLAB® [20]. Calculations have been double-checked by exemplary hand calculations with the help of [21] and a selection of *Monte Carlo Simulations* (*MCS*) in MATLAB® [20]. To study the influence of the variables on the calculated reliability, sensitivity factors for selected configurations are shown. Based on the results, a proposal for the target reliability is worked out. $\gamma_M$ 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* [4] is considered in the modification of $\gamma_G$ 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 $\gamma_M$ based on a reference property are structured in an evaluation procedure for practical application. All calculations have been performed for $T_{ref} = 50\ years$.

# 3 RESULTS

## 3.1 Reliability analyses

### 3.1.1 Permanent loads (1)

Figure 2 and Table 2 show the results for permanent load with three different *cov's*. The blue line indicates the target of EN 1990 Annex C ($T_{ref} = 50\ years$) and consequence class (CC) 2. The sensitivity factors 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. For $V_G \geq 0.10$ the calculated reliability is lower than the target from EC 0, Annex C.

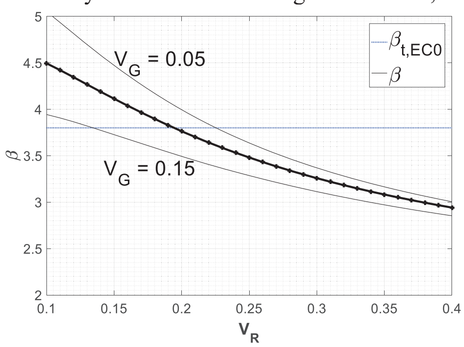


***Figure 2:*** *Reliability index for permanent loads (1)*

***Table 2:*** *Exemplary sensitivity factors*

| $V_R$ | $f$ | $G$ | $\theta_f$ | $\theta_G$ |
|---|---|---|---|---|
| | | $V_G = 0.05$ | | |
| **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 |

### 3.1.2 Permanent and one variable load (2), (3) & (4)

The analyses have been performed for one variable load with $V_Q = 0.25$ (Figure 3, Table 3). The reliability analyses show lower reliabilities than the target. Sensitivity factors reveal the increasing influence of the variable load with increasing load ratio.

Further analyses have been performed for one variable load (wind load) with $V_W = 0.16$ (Figure 4, Table 4). Recent analyses on wind load modelling can be found in [22]. Further studies indicate that the total wind load could be modelled using *cov's* in the range $V_{W,50} = 0.20 \ldots 0.25$, see e.g. [23, 24]. For further calculations in this study $V_{W,50} = 0.16$ for the time invariant part based on the calculation applying the coefficients in [17] as indicated above and with regard to studies in [25, 26] and $V_{W,\theta} = 0.10$ for the time-invariant part is assumed.

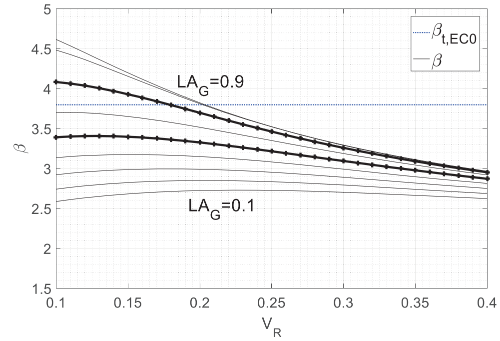


***Figure 3:*** *Reliability index for permanent and one variable load with* $V_Q = 0.25$*, (2)/(3)*

***Table 3:*** *Exemplary sensitivity factors,* $V_Q = 0.25$

| $V_R$ | $f$ | $G$ | $Q$ | $\theta_f$ | $\theta_G$ | $\theta_Q$ |
|---|---|---|---|---|---|---|
| | | | $LA_G = 0.3$ | | | |
| **0.20** | -0.80 | 0.16 | 0.45 | -0.30 | 0.12 | 0.15 |
| **0.25** | -0.87 | 0.15 | 0.35 | -0.26 | 0.11 | 0.13 |
| **0.30** | -0.91 | 0.14 | 0.29 | -0.23 | 0.10 | 0.11 |
| | | | $LA_G = 0.5$ | | | |
| **0.20** | -0.68 | 0.08 | 0.64 | -0.26 | 0.06 | 0.21 |
| **0.25** | -0.77 | 0.08 | 0.55 | -0.23 | 0.06 | 0.19 |
| **0.30** | -0.84 | 0.08 | 0.47 | -0.21 | 0.06 | 0.17 |
| | | | $LA_G = 0.7$ | | | |
| **0.20** | -0.61 | 0.04 | 0.72 | -0.22 | 0.03 | 0.24 |
| **0.25** | -0.70 | 0.04 | 0.65 | -0.21 | 0.03 | 0.22 |
| **0.30** | -0.77 | 0.04 | 0.58 | -0.19 | 0.03 | 0.20 |

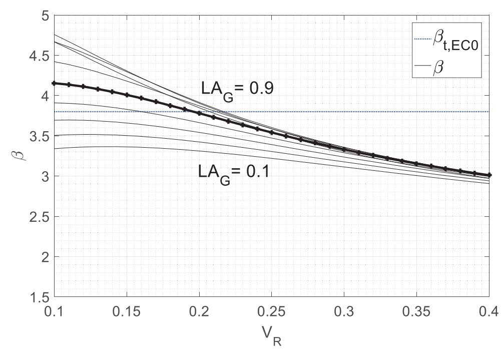


***Figure 4:*** *Reliability index for permanent and one variable load with* $V_Q = 0.16$ *(4)*

***Table 4:*** *Exemplary sensitivity factors,* $V_Q = 016$

| $V_R$ | $f$ | $G$ | $W$ | $\theta_f$ | $\theta_G$ | $\theta_W$ |
|---|---|---|---|---|---|---|
| | | | $LA_G = 0.3$ | | | |
| **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 |


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### 3.1.3 Permanent, snow and wind load (5)

Figure 5 illustrates the reliability for flexural stress from combinations of permanent, snow and wind load. An analysis for Germany showed that for roof angles 30° < α < 75° the load ratio of permanent loads is approximately 50% for normal roof weights (comparison of mean values, coastal areas excluded). Thus, exemplary sensitivity factors for $LA_G = 0.5$ and two different *cov's* of the material strength are given in Table 5**.** Due to the assumed low variability of the wind load its influence on the reliability is rather small. Note that these values are exemplary for $LA_G = 0.5$ and the influence of variable loads increases with greater load ratio.

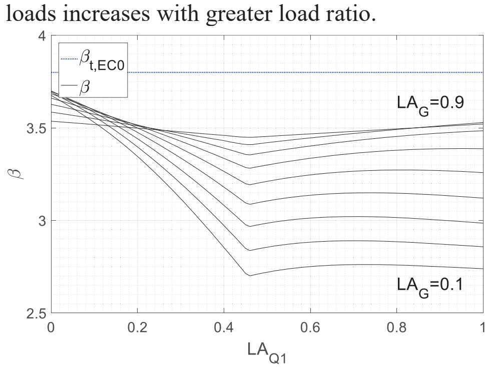


***Figure 5:*** *Reliability index for permanent and variable load 1 (snow) and variable load 2 (wind) with $V_R = 0.25$, uniaxial stress (5)*

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

| $V_R$ | $f$ | $G$ | $S$ | $W$ | $\theta_f$ | $\theta_G$ | $\theta_S$ | $\theta_W$ |
|---|---|---|---|---|---|---|---|---|
| | | | | $LA_{Q1} = 0.3$ | | | | |
| **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 |

### 3.1.4 Biaxial bending (6) & (7)

Biaxial flexural stress from permanent load and snow (6) or live (7) load in one direction and wind load in the other direction, as relevant for e.g. purlins or half-timber walls, have been analysed, considering different load presence time $n_p$ and load change rate $n_r$. Flexural strengths in both directions are correlated with $\rho = 0.8$. The ratio of width and height has been assumed $b/h = 1/2$; a ratio was needed to calculate the design parameter $z_d$. However, the influence of the latter has been turned out to be rather small. A load distribution factor $k_m = 0.7$ for rectangular cross sections has been applied according to EC 5-1 [11]. Figure 6 and Figure 7 show the results. It has turned out that the difference to one-axial bending is rather small for the considered configurations. Sensitivity factors are very similar to Table 5 and thus not repeated here, details can be found in [15].

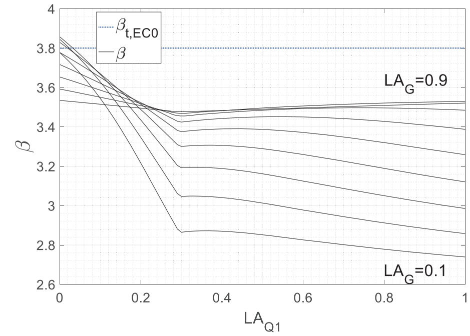


***Figure 6:*** *Reliability index for permanent and variable load 1 (snow) and variable load 2 (wind) with $V_R = 0.25$, biaxial bending (6)*

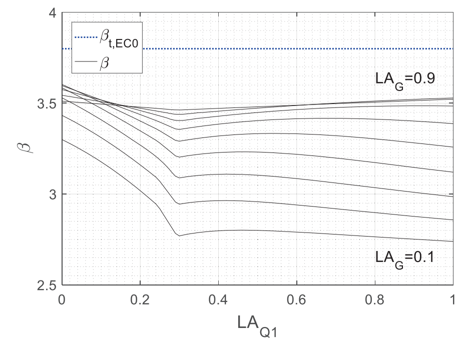


***Figure 7:*** *Reliability index for permanent and variable load 1 (live load) and variable load 2 (wind) with $V_R = 0.25$, biaxial bending (7)*

### 3.1.5 Compression + bending (8) & (9)

The combination of compression and flexural stress has been analysed for compression from permanent and snow load (8) / live load (9) and bending from wind load as e.g. relevant for struts in roof structures of half-timbered walls. Strength properties are correlated with $\rho = 0.8$. The ratio of width and height is $b/h = 1/2$. Figure 8 and Figure 9 show the reliability. The analyses show that for $LA_{Q1}$ the reliability is rather constant for an increasing ratio of variable load one (snow in (8); live load in (9)). Table 6 and Table 7 sensitivity factors for $LA_G = 0.5$.

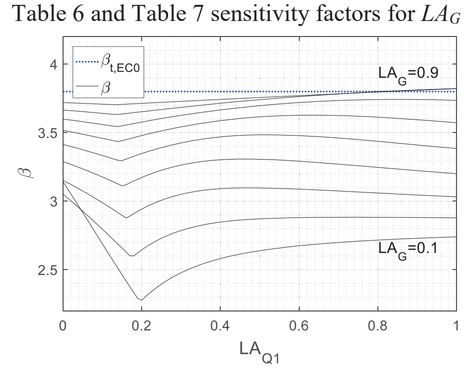


***Figure 8:*** *Reliability index for compression from permanent and variable load 1 (snow load) and bending from variable load 2 (wind) with $V_{R,m} = 0.25$, $V_{R,c} = 0.20$ (8)*

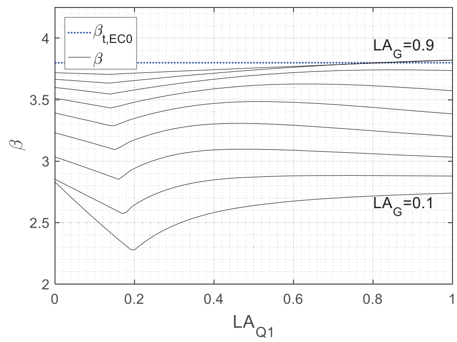


***Figure 9:*** *Reliability index for compression from permanent and variable load 1 (live load) and bending from variable load 2 (wind) with* $V_{R,m} = 0.25$*,* $V_{R,c} = 0.20$ *(9)*

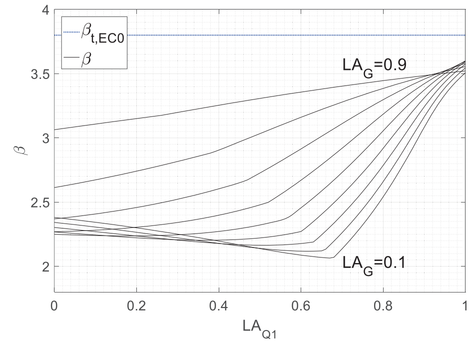


***Figure 10:*** *Reliability index for tension from permanent and variable load 1 (snow) and bending from variable load 2 (live load) with* $V_{R,m} = 0.25$*,* $V_{R,t} = 0.30$ *(10)*

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

| $f_m$ | $f_c$ | $G$ | $S$ | $W$ | $\theta_f$ | $\theta_G$ | $\theta_S$ | $\theta_W$ |
|---|---|---|---|---|---|---|---|---|
| | | | | $LA_{Q1} = 0.3$ | | | | |
| -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)*

| $f_m$ | $f_c$ | $G$ | $S$ | $W$ | $\theta_f$ | $\theta_G$ | $\theta_S$ | $\theta_W$ |
|---|---|---|---|---|---|---|---|---|
| | | | | $LA_{Q1} = 0.3$ | | | | |
| -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 |

### 3.1.6 Tension + bending (10)

The combination tension and flexural stress has been analysed for tension from permanent/snow load and bending from live load (e.g. tension beams in collar beam roof structures). Strength values are correlated ($\rho = 0.8$). The ratio of width/height is $b/h = 1/2$. Figure 10 shows the reliability, Table 8 shows exemplary sensitivity factors, that reveal the dominating influence of the combination of snow load and tension stress. The reliability has been found out to be comparatively low, which is due to the disadvantageous combinations of variables with a high variability. However, it has to be noted that in existing roof structures this limit state for the mentioned elements is commonly not characterised by a utilisation of 100 %.

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

| $f_m$ | $f_t$ | $G$ | $N$ | $S$ | $\theta_f$ | $\theta_G$ | $\theta_N$ | $\theta_S$ |
|---|---|---|---|---|---|---|---|---|
| | | | | $LA_{Q1} = 0.3$ | | | | |
| -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 |

## 3.2 Summary and discussion of target reliability

The analyses of these rather simple limit states result in significant lower reliability indices than the target from *EC 0 Annex C* [2] 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 utilisation of 100%; verification is often governed by connections which is not in the focus of this work. Based on the presented analyses $\beta_{t,exis} = 3.2$ is suggested.

According to [27] 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 optimisation. [28] suggest to apply $\Delta\beta = -0.5$ for a target reliability of existing structures and $\Delta\beta = -1.5$ for a minimum reliability. [29] suggests to move one line higher for the relative costs of safety measures from *ISO 2394:2015* [3] 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


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are not affected by damages or major changes. During calibration of partial safety factors, 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 optimisation in [30]. Alterations of structural members should be verified using the same safety elements as for new structures.

## 3.3 Calibration of partial safety factors

### 3.3.1 Method

The major work in engineering practice is performed applying the semi-probabilistic safety format using partial safety factors (PSF). For the limit states (1) – (9) (Figure 1) and the target reliability levels proposed above the PSF $\gamma_{M,opt}$ for the material strength has been calibrated. The general optimisation procedure is described in [3, 31, 32] and given in eq. (4)

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

with $w_i$ the weighting factor for the LSF and $w_{pen}$ a penalty factor, penalising a reliability lower than the target more than a higher one [9]. Different penalty functions have been investigated in [13]. For this work, penalty factors from eq. (5) have been applied.

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

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 [15]. For permanent and live load, the weighting factors in Table 9 have been considered and compared to results assuming $LA_G = 0.5$ and $V_N = 0.25$ only. Results of the three options have turned out to lead to similar results.

***Table 9:*** *Weighting factor $w_i$ for permanent and live loads*

| | $V_N$ | $LA_Q$ | $w_i$ a | $w_i$ b | Remarks |
|---|---|---|---|---|---|
| 1) | 0,25 (A > 20m²) | 0,3 | 0,50 | 0,35 | *Live load categories office, lobby and living room with A > 20m², and hotel and class rooms.* |
| 2) | 0,25 (A > 20m²) | 0,5 | 0,40 | 0,50 | *Live load categories office, lobby and living room with A > 20m², and hotel and class rooms.* |
| 3) | 0,40 (A ≤ 20m²) | 0,5 | 0,10 | 0,15 | *Live load categories office, lobby and living room with A ≤ 20m², and hospital rooms.* |
| | | | $\Sigma = 1$ | $\Sigma = 1$ | |

During calibration the PSF for permanent and variable loads have been fixed to $\gamma_G = 1.20$ and $\gamma_Q = 1.50$, respectively. The value $\gamma_G = 1.20$ takes into account a reduced variability of permanent loads obtained in a survey on site and is chosen based on [4]. The reduced variability is also considered by a reduced variability of permanent actions of 7% instead of 10% to consider an investigation on site. The results are based on a load ratio of permanent loads of $LA_G = 0,5$ and further statistical parameters given in Table 1.

### 3.3.2 Results

Table 10 and Table 11 show the results for the suggested reliability index for the evaluation level. In [15] 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** |

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

### 3.3.3 Summary and Proposal

Keeping in mind practical applicability, the number of PSF should be limited. Thus, a simplified set of optimised 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 $\gamma_Q = 1.5$ 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.

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

| stress | loads: permanent | loads: 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 $\beta_{t,eval} = 2{,}9$, live load categories A and B, snow and wind loads, load ratio of permanent load $LA_G \geq 0.5$, $\gamma_{G,up} = 1.20$, $\gamma_Q = 1.50$

In order to apply the results (Table 10, Table 11) for an updated material variability from material tests, a sufficient number of tests has to be considered. A proposal in [26] 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 [33] where it is stated that structural members directly affected by changes should be verified applying the same requirements as for new structural members.

### 3.3.4 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 11 to Figure 16 (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 $\gamma_{G,up} = 1.20$, $\gamma_Q = 1.50$, $\gamma_{M,opt} = 1.20$.

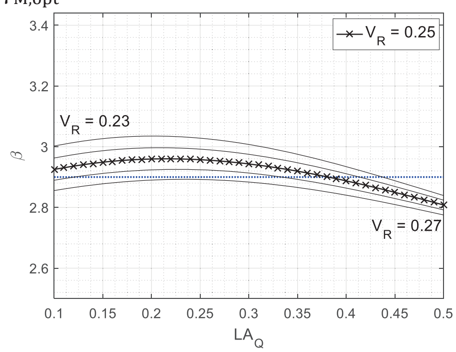


***Figure 11:*** *Reliability index for uniaxial stress, permanent and one variable load with* $V_Q = 0.25$

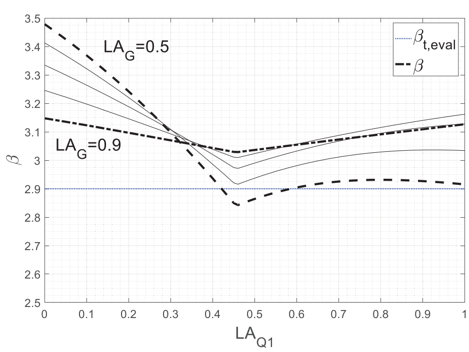


***Figure 12:*** *Reliability index for uniaxial stress, permanent, snow and wind load,* $V_R = 0.20$

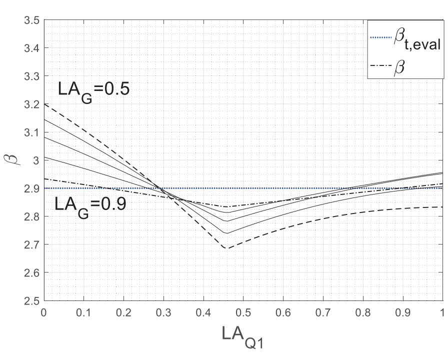


***Figure 13:*** *Reliability index for uniaxial stress, permanent, snow and wind load,* $V_R = 0.25$

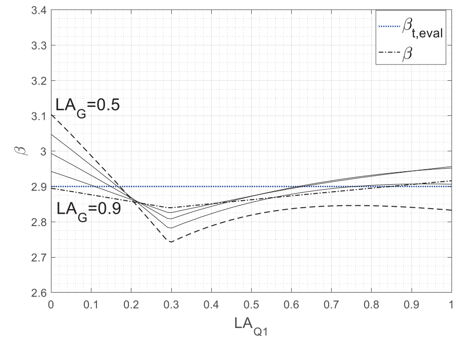


***Figure 14:*** *Reliability index for two-axial bending, permanent, snow and wind load,* $V_R = 0.25$

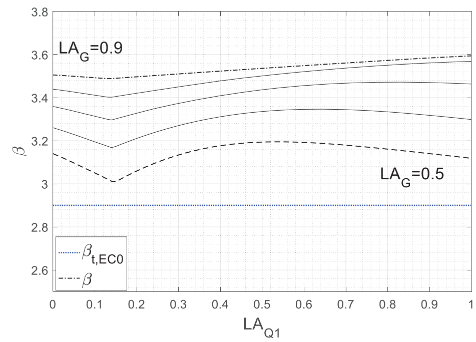


***Figure 15:*** *Reliability index for compression and bending, permanent, snow and wind load,* $V_{R,m} = 0.25$*,* $V_{R,c} = 0.20$

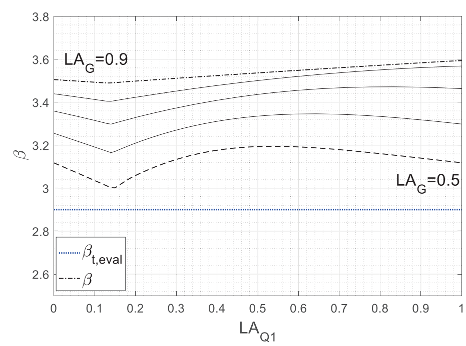


***Figure 16:*** *Reliability index for compression and bending, permanent and live and wind load,* $V_{R,m} = 0.25$*,* $V_{R,c} = 0.20$

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 [15].


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### 3.4 Update of Partial Safety Factors based on Material Update

A great potential when analysing 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. [34]. 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. [35]. This is, however, not in the focus of this work.

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) or the *Adjusted Partial Safety Factor Method* (APFM) as described in [36]. 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 [2]. 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 by updated sensitivity factors 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 [37] 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. (6) and (7), respectively, see [38]. $\mu_{y,code}$ 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.

$$\mu_{y|x_{meas}} = \mu_{y,code} \cdot \left(1 + \rho_{x,y} \cdot V_{y,code} \cdot \frac{x_{meas} - \mu_{x,code}}{\mu_{x,code} \cdot V_{x,code}}\right) \tag{6}$$

$$\sigma_{y|x_{meas}} = V_{y,code} \cdot \mu_{y,code} \cdot \sqrt{1 - \rho_{x,y}^2} \tag{7}$$

Eq. (8) is derived from eq. (6) and (7).

$$V_{y|x_{meas}} = \frac{\sigma_{y|x_{meas}}}{\mu_{y|x_{meas}}} \tag{8}$$

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

with eq. (9) for the PSF of a lognormal distributed resistance variable, the updated PSF $\gamma_{m,\mathrm{up}}$ can be calculated according to eq. (10). Here $\alpha_R$ is the sensitivity factor of the resistance variable, $\beta$ is the target reliability and $q$ is the quantile of the distributed used to define the characteristic value for semi-probabilistic design.

$$\gamma_m = exp\left(V_R \cdot \left(\alpha_R \cdot \beta + \Phi^{-1}(q)\right)\right) \tag{9}$$

$$\gamma_{m,\mathrm{up}} = exp\left(\frac{V_{y,code} \cdot \sqrt{1-\rho_{x,y}^2}}{\left(1+\rho_{x,y} \cdot V_{y,code} \cdot \frac{x_{meas}-\mu_{x,code}}{\mu_{x,code} \cdot V_{x,code}}\right)} \cdot \left(\alpha_R \cdot \beta + \Phi^{-1}(q)\right)\right) \tag{10}$$

The model uncertainty factor $\gamma_{Rd}$ is considered by eq.(11).

$$\gamma_{M,\mathrm{up}} = \gamma_{Rd} \cdot \gamma_{m,\mathrm{up}} \tag{11}$$

Application examples can be found in [38, 39]. 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 [40] and left to further work.

### 3.5 Suggestion of a 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 [41]. The stepwise evaluation procedure embraces three *Knowledge Levels* (terminology based on [30]) 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 13. 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 [35]. An application example can e.g. be found in [39].

***Table 13:*** *Proposal for a stepwise evaluation procedure*

| | | |
|---|---|---|
| **semi-probabilistic <u>without</u> update - *KL 1*** | | |
| *visual grading in situ* | | |
| *partial safety factors from EN 1990, EN 1995-1-1, EN 1995-1-1/NA* | | |
| **semi-probabilistic <u>with</u> update - *KL 2*** | | |
| ***KL 2a)*** | ***KL 2b)*** | ***KL 2c)*** |
| *update of permanent loads:* $\gamma_{G,\mathrm{up}} = 1{,}20$ *(unfavourable)* | | |
| *variable loads:* $\gamma_Q = 1{,}50$ | | |
| *visual grading in situ* | *grading supported by tech. devices* | *update of material parameters* |
| ***strength grade:*** *visual grading* | ***strength grade:*** *grading supported by tech. devices* | *update of* ***material properties*** *by material tests* |
| *PSF on resistance side as optimised* $\boldsymbol{\gamma_{M,opt}}$ *for actual limit states* | | *PSF on resistance side for updated property* $\boldsymbol{\gamma_{M,up}}$ |
| **probabilistic - *KL 3*** | | |
| *grading supported by technical devices / update of material parameters* | | |
| *probabilistic evaluation by approximation or simulation update of material model based on material tests* | | |

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 $\rho > 0.6$.

# 4 CONCLUSION

A responsible use of energy and resources are important elements of the development of a sustainable economy. The preservation of existing structures plays a central role 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 their application on innovative structural materials. The evaluation of existing structures needs to be integrated in the concept of Eurocodes considering the special requirements that come along with this challenge. In this context, rules and requirements need to be flexible so that they can be adopted for different structural tasks.

The presented works contains analyses of the reliability of timber members according to current standards. The analyses show a scatter of the reliability depending on the considered LSF. Sensitivity factors indicate a great influence of the material strength on the calculated reliability. Based on these results, the target reliability is discussed and modified PSF depending on the LSF and, as a second option, considering an update of material properties are presented.

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. 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. [23, 24] for wind loads. Calculating the implicit reliability level of design codes, this might have an influence on the resulting reliability level. 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, a centrals task in the development of codes and standards for existing structures, it has to be discussed how the target reliability has to be be defined for a calibration of PSF for existing structures.


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# REFERENCES


[1] *Assessment and retrofitting of existing structures*, TS 17440:2019 (E), CEN/TC 250, Oct. 2020.

[2] *Basis of structural design*, DIN EN 1990:2010-12, CEN, Dec. 2010.

[3] *General principles on reliability for structures*, ISO 2394:2015(E), ISO, Schweiz, 2015.

[4] *Grundlagen der Erhaltung von Tragwerken*, SIA 269:2011, SIA, Zürich, Jan. 2011.

[5] *Bases for design of structures - Assessment of existing structures*, ISO 13822:2010(E), ISO, Aug. 2010.

[6] *Conservation of cultural heritage – Historic Timber Structures – Guidelines for the On Site Assessment*, EN 17121:2019-12, CEN, 2019.

[7] C. J. Turkstra, *Theory of Structural Design Decisions: Study No. 2*. Ontario, Canada, 1970.

[8] R. E. Melchers, *Structural reliability analysis and prediction,* 2nd ed. Chichester: Wiley, 1999.

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

[10] *Eurocode 1: Actions on structures – Part 1-1: General actions – Densities, self-weight, imposed loads for buildings*, EN 1991-1-1:2010-12, CEN, Dec. 2010.

[11] *Design of timber structures – Part 1-1: General – Common rules and rules for buildings*, EN 1995-1-1:2010-12, CEN, Dec. 2010.

[12] J. Ferry Borges and M. Castanheta, *Structural Safety,* 2nd ed. Lisbon, 1971.

[13] M. Baravalle, "Risk and Reliability Based Calibration of Structural Design Codes," Dissertation, Norwegian University of Science and Technology, Trondheim, 2017.

[14] JCSS, "Probabilistic Model Code: Part 3 - Resistance Models," Joint Committee on Structural Safety, 2006. Accessed: Mar. 8 2016. [Online]. Available: http://www.jcss.byg.dtu.dk/

[15] M. Loebjinski, *Bewertung der Tragfähigkeit von Holzkonstruktionen beim Bauen im Bestand: Ein Beitrag zur substanzschonenden Erhaltung von bestehenden Gebäuden*. Dissertation, Schriftenreihe des Lehrstuhls für Stahl- und Holzbau, Band 19. Brandenburgische Technische Universität, Cottbus, 2021. [Online]. Available: http://d-nb.info/1241545375/34

[16] CIB, "Actions on Structures: Live Loads in Buildings," CIB Report W81, 1989.

[17] JCSS, "Probabilistic Model Code: Part 2 - Load Models," Joint Committee on Structural Safety, 2001. Accessed: Mar. 29 2016. [Online]. Available: http://www.jcss.byg.dtu.dk/ Publications/Probabilistic_Model_Code

[18] J. Grünberg, *Grundlagen der Tragwerksplanung - Sicherheitskonzept und Bemessungsregeln für den konstruktiven Ingenieurbau: Erläuterungen zu DIN 1055-100,* 1st ed. Berlin: Beuth, 2004.



https://doi.org/10.52202/069179-0394

[19] A. M. Hasofer and N. C. Lind, “An Exact and Invariant First-Order Reliability Format,” *American Society of Civil Engineers/ Engineering Mechanics Division*, vol. 100, no. 1, pp. 111–121, 1974.
[20] MathWorks, *MATLAB*, 2016. [Online]. Available: https://www.mathworks.com/products/matlab.html
[21] G. Spaethe, *Die Sicherheit tragender Baukonstruktionen,* 2nd ed. Wien: Springer-Verlag, 1992.
[22] J. Botha, Retief, Johan, V., and C. Viljoen, “Hierarchical Bayesian wind load models for reliability assessment and calibration of standards,” in *12th International Conference on Structural Safety and Reliability (ICOSSAR)*, Vienna, 2017, pp. 2470–2479.
[23] M. Holický and M. Sýkora, “Probabilistic Models for Wind Actions,” in *2016 Second International Symposium on Stochastic Models in Reliability Engineering, Life Science and Operations Management (SMRLO)*, Beer Sheva, Israel, Feb. 2016 - Feb. 2016.
[24] T. P. McAllister, N. Wang, and B. R. Ellingwood, “Risk-Informed Mean Recurrence Intervals for Updated Wind Maps in ASCE 7-16,” *J. Struct. Eng.*, vol. 144, no. 5, p. 6018001, 2018, doi: 10.1061/(ASCE)ST.1943-541X.0002011.
[25] S. Glowienka, “Zuverlässigkeit von Mauerwerkswänden aus großformatigen Steinen: Probabilistische Analyse von großformatigem Mauerwerk aus Kalksandstein und Porenbeton mit Dünnbettvermörtelung,” Dissertation, Technische Universität Darmstadt, 2007.
[26] A. M. Fischer, “Bestimmung modifizierter Teilsicherheitsbeiwerte zur semiprobabilistischen Bemessung von Stahlbetonkonstruktionen im Bestand,” Dissertation, Technische Universität Kaiserslautern, 2010. Accessed: Mar. 29 2016. [Online]. Available: https://kluedo.ub.uni-kl.de/frontdoor/index/index/docId/2259
[27] D. Diamantidis, M. Holický, and M. Sýkora, “Reliability and Risk Acceptance Criteria for Civil Engineering Structures,” in Vol. 16, No. 2, paper #8, *Transactions of the VŠB – Technical University of Ostrava: Civil Engineering Series*, 2016.
[28] D. Diamantidis, M. Holický, and M. Sýkora, “Target Reliability Levels Based on Societal, Economic and Environmental Consequences of Structural Failure,” in *12th International Conference on Structural Safety and Reliability (ICOSSAR)*, Vienna, 2017, pp. 644–653.
[29] T. Vrouwenvelder, “Developments towards full probabilistic design codes,” *Structural Safety*, vol. 24, pp. 417–432, 2002.
[30] Joint Research Centre, “New European technical rules for the assessment and retrofitting of existing structures,” European Commission, Luxembourg, EUR, Scientific and technical research series, 2015.
[31] J. D. Sørensen, “Code Calibration and Timber Experience,” Institute of Building Technology and Structural Engineering, Aalborg University, Aalborg, COST Action E24, 2001.
[32] T. Vrouwenvelder, “Reliability Based Code calibration - The use of the JCSS Probabilistic Model Code,” JCSS, Workshop on Code Calibration, Zürich, 2001.
[33] Fachkommission Bautechnik der Bauministerkonferenz (ARGEBAU), “Hinweise und Beispiele zum Vorgehen beim Nachweis der Standsicherheit beim Bauen im Bestand,” Apr. 2008. Accessed: Jan. 15 2016. [Online]. Available: http://www.bvpi.de/bvpi-content/aktuelles/argebau.htm
[34] H. S. Sousa, J. M. Branco, and P. B. Lourenço, “A Holistic Methodology for Probabilistic Safety Assessment of Timber Elements Combining Onsite and Laboratory Data,” *International Journal of Architectural Heritage*, vol. 10, no. 5, pp. 526–538, 2015, doi: 10.1080/15583058.2015.1007177.
[35] G. Linke, W. Rug, and H. Pasternak, “Strength grading of timber in historic structures – methodology and practical application,” in *6th International Conference on Structural Health Assessment of Timber Structures (SHATiS)*, 2022.
[36] FIB, *Partial factor methods for existing concrete structures: Recommendation*. Lausanne, Switzerland: Fédération internationale du betón, 2016.
[37] M. Loebjinski, J. Köhler, W. Rug, and H. Pasternak, “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)*, London: CRC Press, 2019, pp. 353–360.
[38] J. Köhler, “Die Aktualisierung als zentrales Element in den Erhaltungsnormen - Aspekte der Probabilistik,” in *Dokumentation SIA*, D 0240, *Erhaltung von Tragwerken - Vertiefung und Anwendung: Unterlagen zu den Einführungskursen*, H. Bahnholzer, Ed., Zürich, 2011, pp. 33–36.
[39] M. Loebjinski, G. Linke, W. Rug, and H. Pasternak, “Redevelopment of a wooden roof construction under preservation order,” in *5th International Conference on Structural Health Assessment of Timber Structures (SHATiS)*, 2019, pp. 912–921.
[40] S. Pöhlmann and R. Rackwitz, “Zur Verteilungsfunktion von Werkstoffeigenschaften bei kontinuierlich durchgeführten Sortierungen,” *Materialprüfung*, vol. 23, no. 8, pp. 277–278, 1981.
[41] M. Loebjinski, W. Rug, and H. Pasternak, “The Influence of Improved Strength Grading In Situ on Modelling Timber Strength Properties,” *MDPI buildings*, vol. 10, no. 2, p. 30, 2020, doi: 10.3390/buildings10020030.