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POSSIBILITIES FOR A MODIFICATION OF PARTIAL SAFETY FACTORS FOR EXISTING TIMBER STRUCTURES

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ABSTRACT: Structural changes, changes of use or damaged members lead to an evaluation of the load-bearing capacities of existing timber structures. Current Eurocodes do not contain special regulations for existing structures. Hence, the regulations for new structures are applied. Within a semi-probabilistic design format, the material is graded visually in situ, in general without technical devices. Load-bearing capacities are often underestimated as load and material properties are taken from the code and not updated. This contribution analyses the potential for an adaption of partial safety factors (PSF) using the Design Value Method (DVM). For a practical example a First Order Reliability Method (FORM) analysis is performed and PSF are calculated. Provided that a qualified survey in situ and a grading using technical devices are done, a potential for an adjustment of the partial safety factor depending on the strength property could be figured out.

KEYWORDS: partial safety factor (PSF), existing timber structures, code calibration, first order reliability

1 INTRODUCTION

Our built environment is a central part of our modern society. It is our responsibility to preserve, maintain and use our existing structures. They are part of our history, often of our cultural heritage and objects to learn from for future constructions. What is more, our planetary boundaries remind us to act responsible with resources and energy. Hence, building with existing structures is an important social task and already a great part of the project volume in civil engineering. Within the CEN member states, the so called Eurocodes form the basis of design and verification of load-bearing capacities of structures. Current Eurocodes do not contain recommendations for the evaluation of load bearing capacities of existing structures. The principles for the verification of load-bearing capacities for new structures are applied for existing structures, too. In some countries special rules for existing structures are available, but a common approach does not exist yet. Hence, the potential of a qualified survey is not fully used and load-bearing capacities are often underestimated. It has to be analysed which changes in the design concept are necessary for the evaluation of existing structures and how it is possible to include individual data.

In a first step an adjustment of the target reliability for existing structures is discussed. Besides, partial safety factors (PSF) are calculated using the Design Value Method (DVM) from [1] as described in [2] and compared to current regulations. What is more, a historic timber ceiling is used as a practical example for an evaluation of the reliability of an existing timber structure using First Order Reliability Method (FORM). Besides, open research fields are identified.

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2 TARGET RELIABILITY FOR EXISTING STRUCTURES

The target reliability for the design of structures is given in EN 1990:2010-12 Annex C [3]. For consequence class CC2 the value is β = 3.8 in reference period T_{ref} = 50 years. Target values under economic optimisation are given in ISO 2394:2015 [1].

When evaluating load-bearing capacities of an existing structure, the design situation is different compared to the erection of a new one. Deviations from the defined target reliability of EN 1990:2010-12 [3] can be accepted under special circumstances, e.g. updated information gained by a qualified survey in situ [4] [5], damage free erection and first years of use (Matousek and Schneider [6] out that most of the damages occur while erection and first five years of use) and a good technical know-how from codes and standards [7]. Problems when evaluating existing structures are legal constraints for alterations and time constraints for constructive measures. What is more, the cost efficiency of strengthening measures is often lower compared to alterations during the planning process for a new structure (see e.g. [7] [8]).

Steenbergen et al. [9] suggest to adjust the target reliability for existing structures with $\Delta\beta = 0.5$ and the minimum reliability with $\Delta\beta = 1.5$ on the basis of the target values for new structures. An adaption can also be done on the basis of economic optimisation. Vrouwenvelder [8] suggests to move in the table of ISO 2394:2015 [1] for example from “normal costs of safety measure” to “large costs of safety measure” to define a suitable target reliability.

Based on the publications mentioned above, the target reliability for existing structures is set to $\beta_{t,exis} = 3.2$ and the minimum value to $\beta_{min,exis} = 2.5$ for the following studies in this contribution. This adjustment can solely be applied under certain circumstances. The structure has to be free from major damages and, especially for timber, free from enhanced moisture content that leads to structural derogation. What is more, a qualified survey in situ has to be carried out. If a semi-probabilistic design concept is applied, a careful grading using technical devices has to be done. In [10] a state of the art report concerning in situ grading that shows the potential of the ultrasonic method as one of several SDTs/NDTs for in situ grading can be found.

3 MATERIAL PARAMETERS

The definition of the material model is crucial for reliability analysis and the calibration of partial safety factors. An important parameter is the coefficient of variation (COV). It is defined as

$$COV_x = \frac{\sigma_x}{m_x} = \frac{\sqrt{Var(x)}}{E(x)} \tag{1}$$

where σ_x is the standard deviation, m_x is the mean value, *Var(x)* is the variance and *E(x)* is the expected value. An extensive literature study has been carried out to determine the coefficient of variation of bending, compression and tension strength for graded timber.

In a research project [11] own material testing (bending tests) is done. First bending tests have been carried out using 209 spruce specimen. The literature study and first own results show, that the coefficient of variation of strength properties of graded material is lower than of ungraded material. It can be expected, that the coefficient of variation is lower for material of better quality. First analyses of the results of testing described in [11] show a coefficient of variation of the bending strength of $COV_m = 0.36$ if the material is not graded. For visual grading $COV_m = 0.27$ (C18), $COV_m = 0.31$ (C24) and $COV_m = 0.23$ (C30) is obtained. Here, strength class C24 has the highest value. This is not unusual, as visual grading is, within the borders of grading rules, subjective and material is often classified into the middle class when visual grading is done. The results of the literature study are summarised in Table 1.

***Table 1:** Coefficients of variation for timber strength properties in different grades from literature study (visual grading)*

Material strength	**Grade**		
	C 30	**C 24**	**C 18**
Bending			
Range	0.19…0.34	0.19…0.39	0.20…0.36
Compression			
Range	0.08…0.20	0.14…0.20	0.17…0.22
Tension			
Range	0.21…0.38	0.28…0.40	0.26…0.43

At first, it becomes clear that the coefficient of variation depends on the strength property, where compression strength has the lowest and tension strength the highest value. This is due to the fact that already small structural deviations as e.g. knots have a great influence on the tension strength. What is more. the literature study has shown, that older studies show higher coefficients of variation than newer ones. It can be assumed that by the continuous development of grading standards an improvement of the visual grading has been realised.

The literature has also shown that in almost each study the coefficient of variation was lower in higher strength grades. However, a clear boundary between the classes could not be identified. For this analyses the following values are assumed. These values are under current evaluation and changes during later research and testing are possible.

***Table 2:** Coefficients of variation for timber strength properties in different grades for this contribution*

Material strength	**Grade**		
	C 30	**C 24**	**C 18**
Bending	0.22	0.25	0.30
Compression	0.15	0.18	0.21
Tension	0.28	0.30	0.35

The values shown above are based on visual grading, as this is what is mostly being done when evaluating existing structures. In the literature few work dealing with the improvement of the grading in situ using technical devices could be found. The improvement of strength grading in situ using technical devices is studied in a related research project (see e.g. [11]). It is an ongoing project and the values shown in Table 2 will be updated considering the results gained.

Besides, here just material better than strength class C18 is taken into account. If material of lower quality is used, problems concerning the load-bearing behaviour may occur. This has to be studied more detailed for the individual case.

4 DESIGN VALUE METHOD FOR AN ESTIMATION OF MODIFIED PSF

Within a semi-probabilistic code format, design values for resistance variables R_d are defined as

$$R_d = \frac{\eta \cdot R_k}{\gamma_M} \tag{2}$$

where E_k is the characteristic value defined as a quantile of an assumed distribution function, η a conversion factor taken into account certain material properties and γ_M the partial safety factor. Design values for load variables E_d are defined as

$$E_d = \gamma_F \cdot E_k \tag{3}$$

with E_k the characteristic value of the load and γ_F the partial safety factor. The partials safety factors can be derived from this relation using certain assumptions for the actual distributions function of the variables. Definitions of design values can be found in [1] or [3]. A complete derivation of the formulas for the calculation of the PSF for certain distribution functions is not shown here and can be found in [2], [12] or [13]. A simplified calculation of PSF for timber material properties, and different loads with fixed sensivity factors has been published in [13]. Here, the model uncertainty has been considered by including it into the COV of the basic variable. Another option is to include the model uncertainty using a separate model factor as described in [2] . The PSF for the material side γ_M and for the demand side γ_F (type of action not specified) is:

$$\gamma_M = \gamma_{Rd} \cdot \gamma_m \tag{4}$$

$$\gamma_F = \gamma_{Sd} \cdot \gamma_f \tag{5}$$

where γ_{Rd} is the model uncertainty factor for the resistance (resp. γ_{Sd} for demand variables), γ_m is the factor for the material itself, γ_{Rd} is the model uncertainty factor for the action an γ_f is the factor for the load.

Assuming a lognormal distribution function for the material resistance, the partial safety factor γ_m can be calculated

$$\gamma_m = \exp\left(COV_R \cdot \left(\alpha_R \beta + \varphi^{-1}(q)\right)\right) \tag{6}$$

where COV_R is the coefficient of variation of the material strength, α_R is the sensivity factor (recommended value in EN 1990:2010-12, $|\alpha_R| = 0.8$), β is the target reliability and q is the quantile (for material strength the 5%-quantile is applied). The partial safety factor for normal distributed actions can be calculated

$$\gamma_g = \frac{1 - \alpha_E \beta COV_G}{1 + COV_G \cdot \varphi^{-1}(q)} \tag{7}$$

with additionally α_E the sensivity factor (recommended value in EN 1990:2010-12, $|\alpha_R| = 0.7$) and COV_E is the coefficient of variation of the demand variable. For variable actions that are modelled using a Gumble distribution the PSF is

$$\gamma_q = \frac{1 - \left(\frac{COV_Q\sqrt{6}}{\pi} \cdot \left(0,5772 + \ln\left\{-\ln \varphi\left(-\alpha_E \beta\right)\right\}\right)\right)}{1 - \frac{COV_Q\sqrt{6}}{\pi} \cdot \left(0,5772 + \ln\left(-\ln\left(q\right)^T\right)\right)} \tag{8}$$

The lower case indices in Eq. (6), (7) and (8) indicate, that the PSF does not consider a model uncertainty.

The model factor γ_{Rd} for resistance variables and γ_{Sd} for load variables can be calculated using a normal or a lognormal distribution. In this contribution the model factors are calculated with a normal distribution and using a reduced sensivity factors for accompanying variables. It is calculated γ_{Rd} = 1.06 for the resistance side and γ_{Sd} = 1.05 for permanent actions and γ_{Sd} = 1.11 for variable actions.

The PSF are calculated and results are shown in Table 3 considering the values from Table 2. The COV for permanent actions is assumed to be $COV_G = 0.10$, for live load $COV_Q = 0.20$ [14]. An own simulation of the live load has shown that this assumption is justified for larger rooms under normal loads. For small rooms (A </= 20m²) a higher value has to be used, see also [15]. For snow load $COV_Q = 0.25$ [16] and for wind load $COV_Q = 0.16$ (calculated based on [17]).

Table 3: *PSF calculated with fixed sensivity factors α_i as defined in EN 1990:2010-12, reference period T_{ref} = 50 years*

		Calculation with fixed sensivity factors for β **= 3.2**	β **= 2.5**	**Current Eurocodes [3] & [18]**
Material strength solid wood				
Bending	C30	1.29	1.13	
	C24	1.32	1.14	
Compression	C30	1.21	1.10	
	C24	1.24	1.11	1.3
Tension	C30	1.36	1.15	
	C24	1.39	1.16	
Actions				
Permanent action		1.28	1.22	1.35
Variable action	Live Load	1.91	1.66	
	Snow Load	2.14	1.83	1.5
	Wind load	1.73	1.53	

For permanent actions, it can be seen that a potential for an optimisation of the PSF is given if loads are updated and thus an adaption of the target reliability is justified. The Swiss code SIA 269:2011 [19] allows an adaption to $\gamma_G = 1.20$ if the geometry is investigated by a qualified survey in situ.

The values calculated within this simplified format, the PSF for variable actions γ_Q seem to be very low. However, the statistical parameters depend on a lot of influencing factors. For live load this is the size of the room and the type of use, snow and wind load depend on the location. Besides, the reliability of a component also depends on the load ratio of permanent and variable action. Thus, the determination just based on the simplified method does not lead to justifying results. The example in section 5 shows, that's this result is not transferrable on different design cases in general.

With respect to the material side, a potential for an optimisation of the PSF can be derived for bending and compression strength. The tension strength is characterised by a great COV. Tension members in existing structures have to be investigated carefully regarding their structural characteristics as knots and cracks, to determine their load-bearing capacity. The PSF depend on the quality of the material. Thus, defining a general potential for an optimisation of PSF justified by the fact that an investigation of the structure is possible is not sufficient. A qualified survey in situ has to be carried out to determine the individual condition of the structure and its elements.

This contribution is part of a current research project which aims to develop means to include information gained within a qualified survey in situ into the semi-probabilistic design concept to make use of load-bearing capacities. Here, the approach shown above alone is not sufficient, as too many input parameters cannot be considered. A reliability-based code calibration is needed to create a practice-orientated, consistent design concept. To calibrate PSF for existing timber structures, reliability analyses for a number of practical problems have to be performed. In the following section a practical example is used to demonstrate the influencing factors on a reliability problem and the potential of an adjusted concept for the verification of load-bearing capacities of existing structures.

5 PRACTICAL EXAMPLE – RELIABILITY OF BEAM OF A TIMBER CEILING UNDER BENDING LOAD

5.1 SYSTEM AND LOADS

A historic timber ceiling above ground floor of a clinical centre named "Klinikum Klosterheide" in Lindow, Germany is considered as an example case. The contractor was BaSys GmbH Lenzen (Germany).

- Errichtungszeit?
- Fotos?

The dimensions of the ceiling are 59.36 x 12.62 m. Beams are spanning in cross direction in three sections in three lengths (Pos. DB1 & DB4: 4.30m; DB3: 2.58m: Pos. DB2, DB5 & DB6: 5.32m). A section of the floor plan is shown exemplary in Figure 1. Cross sections of the beams are 140/240mm and 160/240mm.

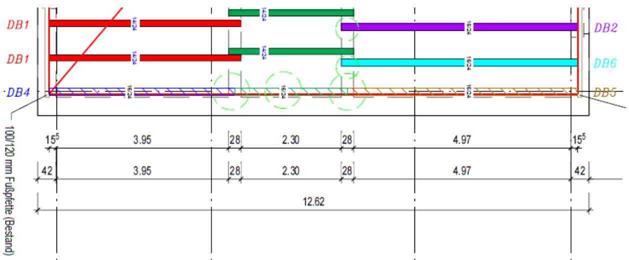

Figure 1: *Timber ceiling – section of floor plan (copyright BASYS GmbH Lenzen)*

A beam of a timber ceiling with a span of l = 5.32 m and a cross section 160/240 cm is analysed (see Table 4). The material is graded by visual inspection to C24. Loads are permanent action and live load (see Table 5).

Table 4: *Practical example – geometry*

Geometry	
Ceiling	12.09 x 59.36m²
Distance of beams	e_{max} = 0.805 m
Most unfavourable Position	Pos. DB2
Cross section Pos. DB2	16/24
Span Pos. DB2	l = 5.35 m

Table 5: *Practical example – loads*

Loads		
Self-weight	2.09 kN/m²	→ $g_{k,1}$ = 1.68 kN/m
Permanent action	2.04 kN/m²	→ $g_{k,2}$ = 1.64 kN/m
Variable load + partition walls	2.8 kN/m²	→ q_k = 2.25 kN/m

5.2 SEMI-PROBABILISTIC EVALUATION

5.2.1 PSF from current Eurocodes

Within a semi-probabilistic evaluation using $\gamma_M = 1.3$, $\gamma_G = 1.35$ and $\gamma_Q = 1.5$ the load-bearing capacity could not be verified as

$$\eta_{EC} = \frac{\sigma_{m,d}}{f_{m,d}} = 1.24 > 1 \tag{9}$$

with $\sigma_{m,d}$ the design value of the stress and $f_{m,d}$ the design value of the material resistance. In practice, the structure was strengthened as Timber Concrete Composite (TCC). For this contribution, the influence of an adaptions of γ_G as given SIA 269:2011 is studied.

5.2.2 Adaption of PSF as given in SIA 269:2011

SIA 269:2011 [19] allows an adaption of the partials safety factor for permanent actions to $\gamma_G = 1.20$ if geometry is determined by a qualified survey in situ. The structure has investigated carefully on site, dimensions have been double checked and permanent actions have been updated. The evaluation using the modified PSF from SIA 269:2011 gives

$$\eta_{EC} = \frac{\sigma_{m,d}}{f_{m,d}} = 1.16 > 1 \tag{10}$$

The load-bearing capacity can still not be verified. However, the excess is reduced significantly. At this point, no update of material parameters has been considered. To evaluate the benefit generated from a (potential costly) determination of material parameters and to identify the main influencing factors on this specific evaluation situation a reliability analysis for this problem is performed.

5.3 RELIABILITY ANALYSIS

To perform a reliability analysis, the First Order Reliability Method (FORM) has been applied. The variables are modelled as shown in Table 6.

Table 6: *Practical example – variables for reliability analysis*

Variable	Distr.	Mean μ [N/mm²]	COV
Material strength	LN	37.1	0.25
Permanent action	N	3.32	0.10
Live load	GUM	2.58	0.20
Model uncertainty resistance	N	1	0.05
Model uncertainty loads	N	1	0.10

Mean and standard deviation for the variables have been calculated considering the characteristic values from

Table 5 and the coefficients of variations as given in Table 6. The characteristic value of the material strength has been assumed to be the 5% quantile of a lognormal distribution, the characteristic value of the permanent action as the 50% quantile of a normal distribution and of the variable action as the 98% quantile in a reference period of $T_{ref} = 1\ year$. What is more, Table 6 shows that for resistance model uncertainty a low value is chosen. This is justified by a qualified survey in situ and a detailed investigation of critical elements.

The reliability analysis has been performed for a reference period of $T_{ref} = 50\ years$. A value of $\beta \approx 2.7$ has been calculated. As described in section 2, the defined target reliability is $\beta_{t,exis} = 3.2$, the minimum value is $\beta_{0,exis} = 2.5$. It is suggested by [9] that for values lower than the minimum value, immediate safety measures should be carried out. The target value defines an optimal upgrade strategy. Options for technical measures are an intensified monitoring, reduction of loads, strengthening measures or, as a last option, demolition [20] [21]. In this case it seems to be a promising option to strengthen the most critical elements. This would be a smaller intervention than the TCC construction as carried out in practice.

Within a reliability analysis sensivity factors α_i are calculated. These are defined as

$$\sqrt{\sum \alpha_i^2} = 1 \qquad (11)$$

The values show the influence of each basic variable on the reliability index. They are shown for this example in Table 7. What is more, the corresponding set of PSF is calculated by applying the sensivity factors determined within the reliability analysis under the assumption, that the reliability index that is reached would be accepted. Table 7 shows the results.

Table 7: *Practical example – partial safety factors and sensivity factors for a target reliability of $\beta = 3.2$*

Variable	$\lvert \alpha \rvert$	**PSF**	**PSF** incl. model
Model factor loads	0.36	1.12	-
Model factor resistance	0.20	1.03	-
Permanent action	0.20	1.06	1.19
Variable action	0.41	1.40	1.56
Material strength	0.79	1.19	1.23

This is the optimal set of PSF for this individual case. Significant deviations from the calculation with fixed sensivity factors for the value $\beta_0 = 3.2$ as given in Table 3 can be seen. The results are shown to emphasize that an optimal set of PSF can only be calculated for specific design situations. Code calibration work, and the PSF given in a code, have to consider all relevant design situations. By a defined set of PSF, deviations from the target reliability have to accepted. The aim is to minimize this deviation considering all relevant design situations. For this example, the sensivity factor for the variable load is lower than suggested in Eurocode 0 (here $\alpha_E = 0.7$). However, the model uncertainty has a comparatively big influence. The sensivity factor for the material strength is $|\alpha_R| = 0.85$. Hence, the material scatter has a great influence on the reliability and updating the material parameters by technical investigation would have a big influence on the calculated reliability index.

5.4 POTENTIAL OF AN INVESTIGATION SUPPOTED BY TECHNICAL MEANS

Studies of a related research project [10] show that by visual strength grading timber is often graded into strength class C24. Bending tests showed that a great part of the material would fit the requirements of strength class C30. Thus, strength grading in situ using non-/semi-destructive means would lead to the exploration of load bearing capacities. For this example, the classification of the material into strength class C30 would lead to

$$\eta_{new} = \frac{\sigma_{m,d}}{f_{m,d}} = 0.99 \approx 1 \qquad (12)$$

The corresponding reliability index would be $\beta = 3.7$. What is more, grading the material in situ with technical means would provide more detailed information on the material used in the structure. Material parameters can be updated and uncertainties are lower. This could for example be used by a reduction of the COV of the material strength. For the example studied in this contribution, a reduction to $COV_R = 0.22$ would lead to a reliability of $\beta_{up} = 2.78$, a reduced PSF for the material strength but an increased PSF for the variable action as its influence on the calculated reliability increases.

For code calibration work, partial safety factors can also defined as input parameters. This way it is possible to fix a value and just modify certain factors. This way the results of an investigation in situ can be included in an optimisation of the PSF of the material and the other factors are fixed. A first analysis is published in [22] More detailed studies are still in progress and will published later on.

6 CONCLUSION

Current Eurocodes do not contain recommendations for the evaluation of existing structures. A qualified investigation of a structure in situ allows a reduction of the target reliability as uncertainties in the evaluation procedure are reduced. However, the definition of partial safety factors is an optimisation problem that aims to reduce the deviation of a target index over all relevant design tasks. It is shown, that a calculation of PSF with fixed sensivity factors does not lead to satisfying results. Code calibration work is an optimisation problem that need an iterative procedure as for example described in [9]. Within such a format, it is possible to adjust just one or two of the PSF and keep others, like for variable actions, constant as defined in the code.

An adjustment of the PSF should be linked to a qualified investigation in situ. The update of material parameters by measurements would allow an adjustment of the safety factors. First results show, that a potential for an adjustment of the PSF for existing timber structures depend on the material property. The COV of different material properties is different, as it is a naturally grown, anisotropic material.

For code calibration work, more analyses including different structural systems and loadings have to be performed. What is more, requirements of a qualified survey in situ and the amount and quality of data has to be defined to develop an optimised concept for the evaluation of existing structures. This includes the development of possibilities to include individual information of a structure into the design concept. Possibilities are to modify the PSF depending on the COV to be measured in situ as in [4] or the development of a formula to update the PSF directly taking into account the update of a material property by measurements in situ as suggested in [23].

An important part is the improvement of the grading in situ, current work is presented in [11].

7 ACKNOWLEDGEMENT

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