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Approaches for an optimisation of partial safety factors for historic timber structures

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Abstract: The aim of this contribution is to analyse the potential for a modification of partial safety factors (PSF) for existing timber structures. Target and minimum reliability level for historic structures are discussed first. PSF are calculated with fixed sensivity factors to study the effects of a variation of input parameters. Additionally, two structural elements of representative timber constructions are analysed. Realised safety levels using PSF on the demand side from EN 1990:2010-12 and for an adaption of γ_G as in SIA 269:2011 for selected failure modes are determined using First Order Reliability Method (FORM). Besides, the influence of a modification of γ_M to $\gamma_M = 1.25$ on the reliability level is studied. A potential for an optimisation of PSF for existing timber structures can be concluded.

1 Introduction

In recent standards of civil engineering the design of new-built constructions is being done using semi-probabilistic partial safety factor method. Using these factors, a target reliability level shall be ensured. However, these factors have been calibrated using the determinations of former codes without exact probabilistic determination. These factors are defined for new structures. When working with existing structures, circumstances are different: information can be gathered on-site, the structure has proven itself and safety measures often include more effort and less benefit when compared to the application at new structures. This is why it should be analysed if partial safety factors (PSF) should be adjusted to reach a more economic design of existing structures.

In some countries, adjusted PSF already exist. To be named here are SIA 269:2011 [1] which includes a modified safety factor for permanent actions. What is more, NEN 8700:2011-12 provides adjusted factors for permanent and variable actions dependant on two target reliability levels (target i.e. reconstruction and minimum i.e. disapproval level). Besides, the working group of CEN/TC250/WG02 published a report in 2015 which among others contains adjusted PSF as well. However, in these documents no special reference to construction material is made and PSF are mainly adjusted on the demand side. For some construction materials suggestions already exist in this field. For example, in Germany some standards including adjusted PSF on the material side for concrete structures can already be used (e.g. [2] [3] [4]). In [5] a summary can be found. However, no special regularities for existing timber structures exist yet. What is more, there are few possibilities to consider properties measured on-site in design in a standardised way. That is why this contribution analyses the potential for an adjustment for PSF for existing timber structures.

2 Symbols and Statistical Parameters for Historic Timber Structures

The following symbols are used:

a	Parameter of Gumbel distribution	E_d	Design value of action
α_E	Sensivity factor for actions	E_k	Characteristic value of action
α_R	Sensivity factor for resistance	m	Mean value
β_0	Minimum reliability index	q	Fractal value
β_t	Target reliability index	R_d	Design value of resistance
COV	Coefficient of variation	R_k	Characteristic value of resistance
γ_M	Partial Safety Factor for material	σ	Standard deviation
γ_G	Partial Safety Factor for permanent action	u	Parameter of Gumbel distribution
γ_Q	Partial Safety Factor for variable action		

For the following calculations the statistical parameter presented in table 1 are applied.

Table 1: Statistical Parameters for Modelling Historic Timber Structures

	Basic variable	Distr.	m_x /X_k	COV	Remarks
Resistance	**Strength properties**				
	Compression	LN	-	0.20	COV value from [6]
	Bending	LN	-	0.25	COV value from [6]
	Tension	LN	-	0.30	COV value from [6]
Demand	**Permanent load**				
	Construction dead load	N	1.0	0.10	COV value from JCSS PMC [6]
	Live load				
	Residence	GUM	1.10	0.20	COV value from CIB W81 [7]
Model	**Model uncertainty**				
	Resistance	N	1.0	0.05	
	Demand – self weight	N	1.0	0.05	
	Demand – variable actions	N	1.0	0.10	

A model uncertainty of 5% can be realised by a qualified survey on-site using appropriate techniques. These techniques have evolved significantly during the last years, see e.g. [8]. Furthermore, the cross section has to be determined on-site. This is why no scatter is taken into account for geometrical properties.

3 Target Reliability for Historic Structures

3.1 Basics

When assessing and evaluating an existing structure the design situation is different compared to new buildings. Parameters for demand and resistance can be updated. What is more, (possible) damages can be taken into account directly. This enhanced knowledge level leads to the possibility to adjust the reliability index as a general safety distance between action and resistance without lowering the implied socially accepted safety level. This chapter summarises suggestions from the literature for adjusted target reliability indices for historic structures. Using the cited references, minimum and target values are defined for the following studies. The following calculations are considering a reference period of T_{ref} = 50 years. Therefore, target reliability indices have to be transformed into this reference period, if not already given in the reference. The transformation is done as follows [9].

$$\Phi(\beta_{t,n}) = [\Phi(\beta_{t,1})]^n \tag{1}$$

As in this contribution CC2 is being looked at, these values are highlighted by light grey shades.

3.2 Target values by classification depending on reliability/ consequence class

EN 1990:2010-12 includes the target reliability dependent on the reliability class (RC). Steenbergen et al. suggest to lower this value by $\Delta\beta = 0.5$ for the target value and $\Delta\beta = 1.5$ for the minimum value for existing structures [10]. The JRC Science and Policy Report worked out by the group CEN/TC260/WG02 from 2015 [11] gives reliability indices dependent on the reliability class (RC) for existing structures. These values are consistent with the minimum values from NEN 8700:2011-12. Table 2 summarises the values from the mentioned references.

Table 2: Target reliability index in different reliability classes, reference period T_{ref} = 50 years

	β_{New}	$\beta_{Min,Existing}$			$\beta_{Target,Existng}$		
Reliability class	EN 1990:2010-12 [12]	Steenbergen et al. [10]	CEN Report [11] NEN 8700:2011[13]		Steenbergen et al. [10]	NEN 8700:2011 [13]	
			wn	wd		wn	wd
RC 0/ CC1a	-	-	1,8	0,8	-	-	-
RC 1/ CC1b	3,3	1,8	1,8	1,1	2,8	2,8	1,8
RC 2/ CC 2	3,8	2,3	2,5		3,3	3,3	2,5
RC 3/ CC 3	4,3	2,8	3,3		3,8	3,8	3,3
wn – wind not dominant, wd – wind dominant							

3.3 Target values considering an economic optimisation

The classification of the reliability index can also be done using the costs for the safety measure and the consequences of failure. This classification is used in ISO 2394:2015 [14] and the Probabilistic Model Code [15]. The Swiss code SIA 269:2011 [1] uses a similar classification, but instead of the "relative costs of safety measures" the "efficiency of measures" considered. Diamantidis et al. [16] suggest a reduction of the target reliability index for existing structures under economic optimisation using a value of 0.5 (applied on the values from ISO 2394:1998). To consider increased costs of safety measures, Vrouwenvelder (2002) suggests to move in the table for the reliability index from Probabilistic Model Code to the top, i.e. from e.g. low costs to normal costs [17].

Table 3: Target reliability for 1 year/ 50 years reference period for ultimate limit state

Relative costs of safety measure/ efficiency of measure	**Low consequences of failure**				**Moderate consequences of failure**				**Great consequences of failure**			
	New structures		Existing structures [16]		New structures		Existing structures [16]		New structures		Existing structures [16]	
	[1] [14] [15]	Eq. (1)			[1] [14] [15]	Eq. (1)			[1] [14] [15]	Eq. (1)		
	T=1a	T=50a	T=1a	T=50a	T=1a	T=50a	T=1a	T=50a	T=1a	T=50a	T=1a	T=50a
Great/ Low	3.1	1.7	2.6	0.81 1.20	3.3	1.8	2.8	1.17 1.30	3.7	2.7	3.2	1.83 2.20
Normal/ Mean	3.7	2.7	3.2	1.83 2.20	4.2	3.2	3.7	2.55 2.70	4.4	3.5	3.9	2.82 3.00
Low/ Great	4.2	3.2	3.7	2.55 2.70	4.4	3.5	3.9	2.82 3.20	4.7	3.8	4.2	3.21 3..30
Dark grey shades: derived from Vrouwenvelder [17]												

Based on the cited literature, a target value for existing structures $\boldsymbol{\beta_t = 3.2}$ and a minimum value $\boldsymbol{\beta_0 = 2.5}$ (T_{ref} = 50 years) are defined for the following studies.

4 Simplified Calculation of Partial Safety Factors for Existing Timber Constructions

4.1 Definition of Partial Safety Factors

General remarks

Using fixed sensitivity factors, partial safety factors can be calculated in a simplified way. Sensitivity factors are α_E = -0.7 and α_R = 0.8 as defined in EN 1990:2010-12 Annex C. Hence, resistance and action side can be considered as independent [12]. In this approach, model uncertainties are considered by an increase of the coefficient of variation of the variable. However, another option is to define uncertainty factors to multiply with the PSF of the variable. For detailed explanations see fib Bulletin 80 from 2016 [18].

Material Resistance

The material resistance is assumed to be lognormal distributed. For the characteristic value the 5% quantile is used. Partial safety factors can be calibrated as follows:

$$\gamma_M = \frac{R_k}{R_d} = \frac{m_R \cdot \exp\left(-0{,}5 \cdot COV_R^2 - COV_R \cdot \Phi^{-1}(q)\right)}{m_R \cdot \exp\left(-0{,}5 \cdot COV_R^2 - \alpha_R \cdot \beta \cdot COV_R\right)} = \exp\left(COV_R \cdot \left(\alpha_R \cdot \beta - \Phi^{-1}(q)\right)\right) \tag{2}$$

Dead Loads

Dead loads are assumed to be normal distributed. EN 1990:2010-12 allows to use the 50% quantile as a characteristic value, if the construction is not too sensitive against changes of dead loads. Therefore, the partial safety factor can be calculated as:

$$\gamma_G = \frac{E_d}{E_k} = \frac{m_E \cdot \left(1 - \alpha_E \cdot \beta \cdot COV_E\right)}{m_E \cdot \left(1 + COV_E \cdot \Phi^{-1}(q)\right)} = \frac{1 - \alpha_E \cdot \beta \cdot COV_E}{1 + COV_E \cdot \Phi^{-1}(q)} = 1 - \alpha_E \cdot \beta \cdot COV_E \tag{3}$$

Variable Loads

To model variable loads, exponential distributions are used. Here, Gumbel distribution is applied. For variable loads, the reference period is of interest. Characteristic values are defined in EN 1990:2010-12 as 98% quantile for a reference period T_{ref} = 1 year, or as the modal value in T_{ref} = 50 years. The partial safety factor can be calculated using:

$$\gamma_Q = \frac{E_d}{E_k} = \frac{u - \frac{1}{a}\ln\left(-\ln \Phi(\alpha_E \cdot \beta)\right)}{u - \frac{1}{a} \cdot \ln\left(-\ln(p)\right)} \tag{4}$$

Parameters: $a = \frac{\pi}{\sigma \cdot \sqrt{6}}$ $\quad u = \mu - \frac{0{,}5772}{a}$

With $COV = \frac{\sigma}{\mu}$ it becomes:

$$\gamma_Q = \frac{1 - \left(\frac{COV_E \sqrt{6}}{\pi} \cdot \left(0{,}5772 + \ln\left\{-\ln \Phi\left(-\alpha_E \beta\right)\right\}\right)\right)}{1 - \frac{COV_E \sqrt{6}}{\pi} \cdot \left(0{,}5772 + \ln\left(-\ln(q)^T\right)\right)} \tag{5}$$

4.2 Results

Table 4 shows the results of the calculation of partial safety factors using fixed sensivity factors. The coefficients of variation of the material properties for this calculations are selected based on the Probabilistic Model Code of the JCSS Part 3.5 [6]. In [5] a study regarding the influence of different wood species using this simplified method has been done.

Table 4: Partial Safety Factors calibrated with fixed sensivity factors

Basic variable		Mat[1]	Distr.	COV			$\lvert\alpha_i\rvert$	γ_{mod}[2]			Ref[3]
				COV Variable	COV Model	COV Total		$\beta_{t.new}$ =3.8	$\beta_{t.exis}$ =3.2	$\beta_{0.exis}$ =2.5	
Resistance	**Compression strength**										
		Soft wood	LN	**0.20**	0.05[4]	0.20	0.8	**1.34**	**1.21**	**1.08**	[6]
	Tension strength										
		Soft wood	LN	**0.30**	0.05[4]	0.30	0.8	**1.52**	**1.32**	**1.11**	[6]
	Bending strength										
		Soft wood	LN	**0.25**	0.05[4]	0.25	0.8	**1.42**	**1.25**	**1.09**	[6]
Demand	**Self-weight**										
		Soft wood	N	**0.10**	0.05[4]	0.11	0.7	**1.29**	**1.25**	**1.19**	[6]
	Live load										
	Residence	-	GUM	**0.20**	0.10	0.22	0.7	**2.06**	**1.84**	**1.61**	[7]
	Office	-		**0.19**		0.21		**2.01**	**1.79**	**1.58**	[7]
	Further variable loads										
	Wind	-	GUM	**0.16**	0.10	0.19	0.7	**1.90**	**1.71**	**1.52**	[19]
	Snow	-	GUM	**0.25**	0.10	0.27	0.7	**2.33**	**2.05**	**1.77**	[19]

[1] The values depending on the kind of wood in DIN 68264:2003 are related to flawless wood
[2] exis – existing structure. new – new structure. t – target value. 0 – minimum value
[3] References for assumption for $COV_{Variable}$ which refers to the basic variable (material or action)
[4] A qualified survey on-site is prerequisite!

For material resistances, the calculated partial safety factor is strongly dependent on the coefficient of variation of the material strength. As different timber material properties have different coefficients of variation, it is important which property is considered. That is a significant difference in comparison with other materials. Using $COV_R = 0.20$ for the compression strength and $\beta_{t,new} = 3.8$ a slightly greater value for γ_M ($\gamma_M = 1.34$) is calculated than fixed in the recent standard (in EN 1995-1-1/2010-12 $\gamma_M = 1.3$). This could arise from the simplifications of this method. For higher coefficients of variation (i.e. for bending and tension strength), the calculated value for γ_M is higher. However, when lowering the target reliability for existing structures as suggested above, a potential for an optimisation of the PSF, especially for compression and bending strength, can be indicated. As a main indicator for the scatter of the material properties the recommended values from the JCSS Probabilistic Model Code [6] are used. For further analysis, a division dependant on grading classes would be useful. Here, testing is currently carried out at HNE (Germany).

It can be seen, that partial safety factors (PSF) for variable loads in recent standards are probably too low, as they are fixed in the standard to $\gamma_Q = 1.50$. For dead loads the partial safety factor reaches the requirements for new buildings, it is fixed in EN 1990:2010-12 to $\gamma_G = 1.35$. A potential for an optimisation for existing structures can be seen.

The results show, that a potential for an optimisation of partial safety factors for historic timber constructions generally exist. As these results are calculated using a simplified method, probabilistic parameter studies are recommended for an accounted adaption.

5 FORM Analysis of Structural Timber Elements

5.1 General Specifications and Aim of Analysis

The safety level which is reached using PSF of EN 1990:2010-12 and EC 1995-1-1:2010-12 under application of the load which can be taken by the construction under the assumption of a one-hundred percent workload is determined. This load is calculated from the deterministic design equation of the latest design standards EN 1995-1-1:2010-12 and used in the probabilistic calculation. Analysed are a tie beam under permanent load and a beam of a ceiling under permanent and live load. Calculations have been carried out using MATLAB. A step-by step example for a FORM analysis can be found in [5]. Used statistical parameters are shown in table 1. These calculations aim to evaluate the results of the simplified calculations.
As mentioned above, some standards already include adjusted PSF for existing structures. SIA 269:2011 [1] permits γ_G = 1.20 for updated permanent actions. NEN 8700:2010-12 includes γ_G = 1.20 (disapproval) and γ_G = 1.30 (reconstruction) [13]. To analyse the potential for an optimisation of the PSF on the material side, the influence of an adjustment of the PSF for the permanent action to γ_G = 1.20 is studied and the influence of an additional modification of γ_M to an exemplary value of γ_M = 1.25 on the reliability level are analysed.

5.2 Tie-beam

Analysed was a tie-beam, material C24 (EN 338:2009 [20]) under normal moisture conditions (figure 1). The tension strength has the biggest scatter and is therefore the most unfavourable material property for a statistical analysis with only dead load. Using a coefficient of variation for the load COV_E = 10% and a model uncertainty θ_E = 5% as stated above, the following results can be calculated (figure 2).

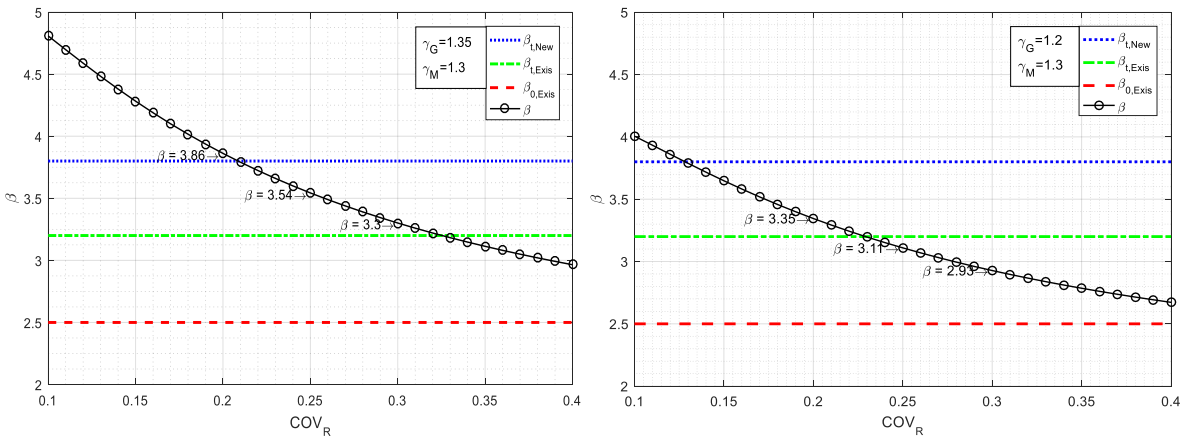

Figure 1: Safety Level Tie-Beam under Dead Load

Obviously, for this example the reached safety index is highly dependent on the coefficient of variation of the material strength. In the Probabilistic Model Code of the JCSS Part Timber [6] a coefficient of variation of COV_R = 0.30 is recommended for the tension strength of structural

timber. As indicated in the figure, for this value the safety index is $\beta = 3.3$ when $\gamma_G = 1.35$, which is lower than the target value for new buildings ($\beta_t = 3.8$). Assuming that we have an existing structure and updated information on actual load and material properties this value is acceptable. If the coefficient of variation can be determined using testing on-site and a lower value is reached, the realised safety level increases significantly. A potential for an adjustment of the partial safety factor for the material property for this structural element can be seen.
An adjustment of $\gamma_G = 1.20$ has a significant influence (see figure 3). Here, adjusting the PSF for the material does not seem to be reasonable when a tension force is assumed. However, for material properties that have a lower COV (bending, compression), the PSF can be optimised.

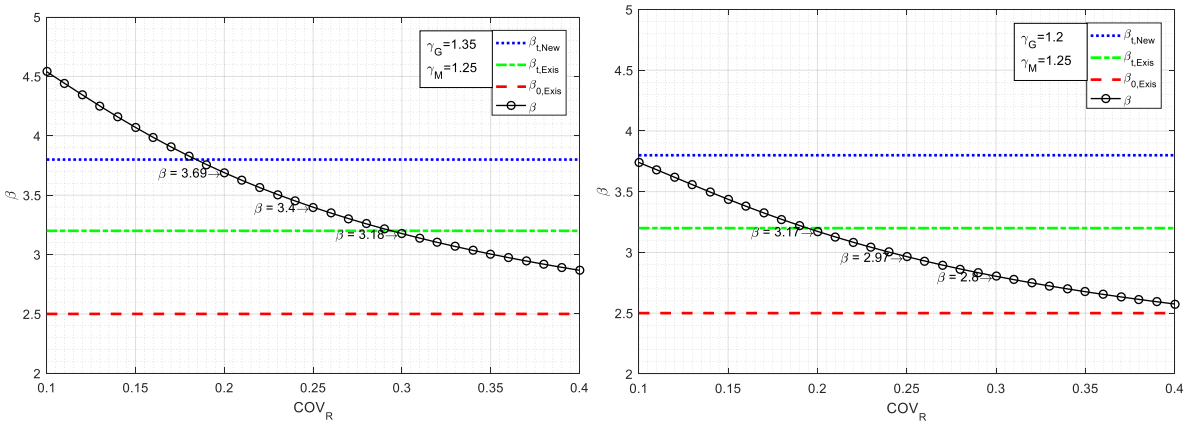

Figure 2: Safety Level Tie-Beam under Dead Load – PSF on material side adjusted

5.3 Beam of a Timber Ceiling

The second example is a beam of a historic timber ceiling under dead and live load. A possible cross section is shown in figure 4. This element is analysed concerning bending load.

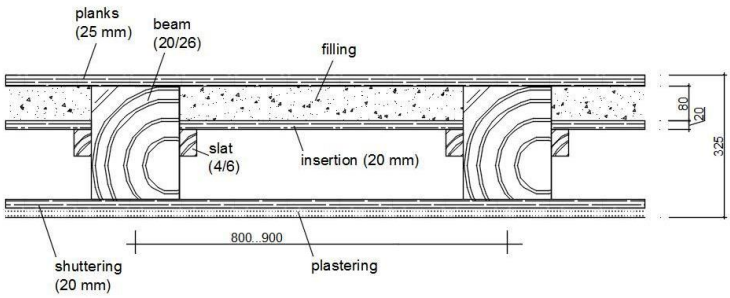

Figure 3: Historic Timber Beam Ceiling – Example for construction [21]

A COV_E of 10% for the dead load has been chosen on the safe side. Model uncertainty is again a coefficient of variation of 5%, a qualified survey on-site has to be assumed. Using this assumptions, the following relation between coefficient of variation for the material resistance (COV_R) and the reliability index β can be drawn for a bending load (figure 5 and 6). The load ratio between variable and permanent actions have been chosen based on self-weight of common structures and typical live-load classes. A load ratio of $q_k / (g_k + q_k) = 0.5$ is the common case for many structures. This is also covered by the assumptions below.

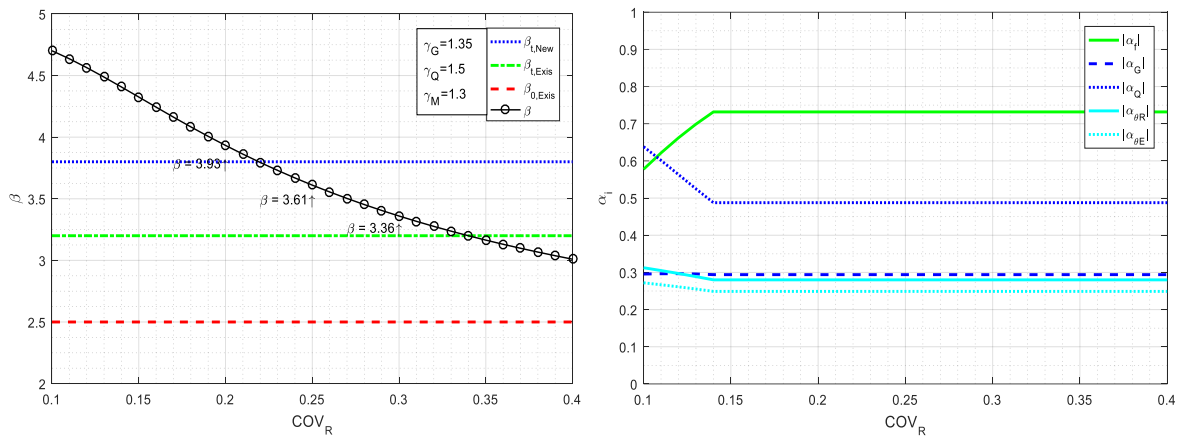

Figure 4: Beam of Ceiling under Dead and Live Load, influence of COV_R on Safety Level and Sensivity Factors, Load Ratio $q_k / (q_k + g_k) = 0.3$

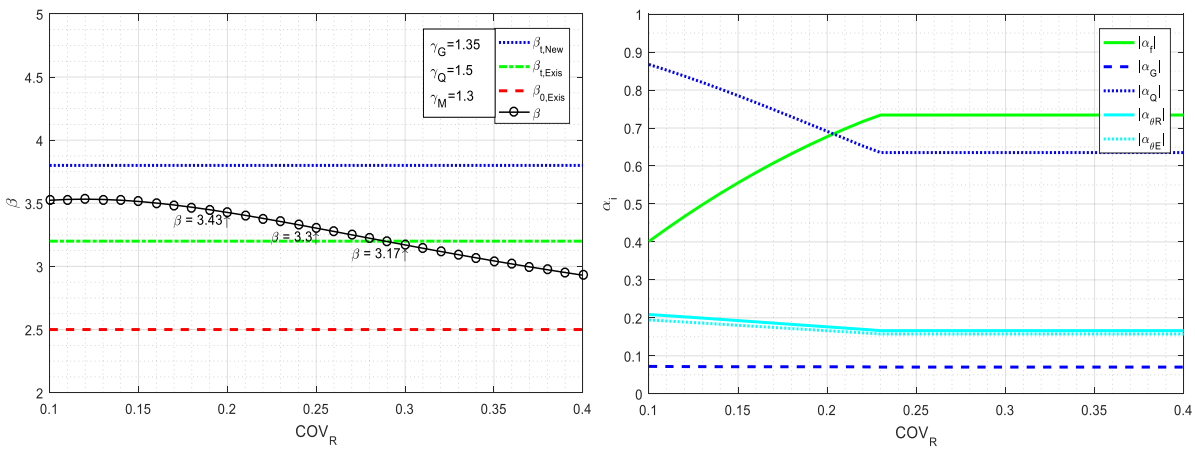

Figure 5: Beam of Ceiling under Dead and Live Load, influence of COV_R on Safety Level and Sensivity Factors, Load Ratio $q_k / (q_k + g_k) = 0.7$

An appropriate coefficient of variation for the bending strength is COV = 0.25 (JCSS [6]). The reliability index $\beta = 3.61$ for $q_k/ (q_k + g_k) = 0.3$ and $\beta = 3.30$ for $q_k/ (q_k + g_k) = 0.7$ is reached. Again, this is a little too low for the requirements of a new building, but for an existing one this is acceptable (again under assumption that updated data for load and resistance are used!). Besides, the influence on the variable action in the reliability when having low COV of the material strength can be seen. Studying the development of the sensitivity factors it can be seen, that even when considering a load ration $q_k/ (q_k + g_k) = 0.7$ the influence of the material property on the reliability index is higher than the influence induced be the variable action when having a coefficient of variation COV > 0.20. Hence, for a beam subjected to bending by live load the influence of the material scatter is the dominating influence. This emphasizes the importance of a qualified survey on-site to determine actual material parameters. The influence of permanent actions and model uncertainties is significantly lower.

To study the effect of an adjustment of the PSF for permanent action and material property, the load ratio $q_k/ (q_k + g_k) = 0.5$ is chosen as an example, as this is the common case (figure 7).

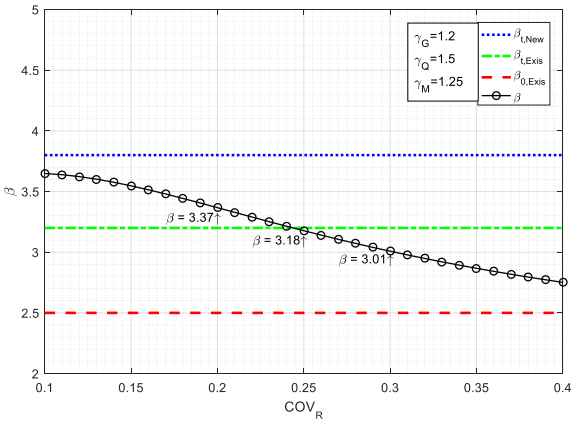

Figure 6: Beam of Ceiling under Dead and Live Load, influence of COV_R on Safety Level, γ_G and γ_M adjusted, Load Ratio $q_k / (q_k + g_k) = 0.5$

It can be seen that when adjusting both γ_G and γ_M as indicated in the figure the target reliability defined before for existing structures is approximately reached

6 Conclusion

Both, the probabilistic studies and the simplified calculations show, that partial safety factors are highly dependent on the coefficient of variation of the material strength. For the tension strength, which has a comparatively high coefficient of variation, the target reliability index respectively the partial safety factor of EN 1995-1-1:2010-12 for new buildings cannot be verified under the given assumption. Nevertheless, a potential for an optimisation for PSF especially for bending and compression strength for existing timber structures can be concluded. An exemplary adjustment to $\gamma_M = 1.25$ has been studied for these structural elements.
Besides, sensivity factors for the beam under dead and live load have been studied. The sensitivity factor for dead loads was not high. Hence, the influence on the reliability level is not high which is due to a low coefficient of variation used. Concerning the beam of the ceiling, the sensitivity factor for the live load is lower than the sensitivity factor for material strength for coefficients of variation of the material above COV = 0.20. As for timber material properties coefficients of variation of the bending strength is in most cases higher than that, the influence of the material scatter is higher than the influence of the variation of the variable load. This emphasizes the importance of a qualified survey on-site when evaluating an existing building, as load-bearing behaviour can be modelled more appropriate which has a great influence on the reliability analysis which is an important finding.

7 Outlook and Further Research

In further research more structural elements and loads have to be studied, especially roof structures, to create broadly applicable results. Besides, the influence of more than one material resistance in the limit state function has not been analysed. What is more, testing on-site has to be done to evaluate the accuracy of the chosen probabilistic parameters. Predictions have to be made, how reliable the results concerning material strengths determined with different techniques are. For practical application a distinction of PSF between material properties would be possible. The effect of the timber grade on the COV of the material hast to be studied, too.
For further research the following fields can be summarised:

- Probabilistic parameter studies on the reliability of elements under snow and wind load
- Probabilistic parameter studies including combined strain (e.g. bending & compression)

- Testing on-site concerning COV's in different timber for timber in existing structures
- Development of a consistent concept for optimised partial safety factors for existing timber structures dependant on the knowledge level gained on-site

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