Reliability levels obtained by Eurocode partial factor design - A discussion on current and future reliability levels

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To assess the reliability of a structure, reliability based codes such as EN1990:2002 allow for the application of full-probabilistic methods and semi-probabilistic methods (i.e. the partial factor method). In principle, both methods should be equivalent and lead to (approximately) the same reliability level. In this study, this equivalence is assessed by investigating the structural reliability of a large number of structural elements designed according to EN1990:2002 partial factor method. Several material types, failure modes, variable loading types and load ratios are investigated. Both the one year reference period and the fifty year reference period are assessed and compared. For the future developments of EN1990, several suggestions are made to obtain a more uniform reliability level over different load-ratios between self-weight, permanent loads, and variable loads. Most important recommendations for EN1990 are: (i) to switch to the one year reference period for the reliability based design and assessment of structures and structural elements; (ii) to lower the partial factor for self-weight for material types with low variability; (iii) to include a slightly larger partial factor for wind and snow loads. It was demonstrated that neither of these suggestions would require any change or recalibration of the material dependent Eurocodes.

Keywords: Structural reliability, partial factors, safety format, probabilistic framework, Eurocodes

1 Introduction

*1.1 **Problem statement***

To ensure structural safety, structures need to fulfil minimum reliability requirements (target reliabilities). In the present Eurocode EN1990:2002, target reliabilities are prescribed as a function of consequence class (CC), reference period and limit state. For structures corresponding to the ultimate limit state (ULS) and CC2 (buildings with medium consequences for the loss of human life and considerable economic, social, or environmental consequences), the prescribed target reliabilities are $\beta_t = 3.8$ for the fifty year reference period and $\beta_t = 4.7$ for the one year reference period; the latter obtained from the first assuming mutual independence between the annual failure events (Vrouwenvelder, 2002).

To assess the reliability of a structure, EN1990:2002 allows for the application of full-probabilistic and semi-probabilistic methods (i.e. the partial factor method). In principle, both methods should be equivalent and lead to (approximately) the same reliability level. To ensure this, the partial factors are ideally obtained by a full-probabilistic calibration, accounting for a wide range of design situations (material types, failure modes, load combinations). In practice however, the partial factors are often obtained by a combination of historical developments, expert judgement, and calibrations to previous design methods; not by means of a dedicated full-probabilistic calibration. As a result, many design situations are deemed to apply partial factors that may differ from the 'optimal' values, resulting in a (wide) scatter of reliability levels over the distinct design situations (Gulvanessian, 2002). It is therefore questionable whether a design according to the partial factor method indeed results in the same reliability level as a design according to the full-probabilistic method.

At this moment, CEN (SC10-WG1) is pursuing the revision of EN1990 (expected release: 2020). As part of this revision, the currently prescribed partial factors and target reliabilities in EN1990:2002 are debated and taken under scrutiny. Questions that gain most attention are:

- What should the future target reliabilities be?
- Which reference period should the target reliabilities have (one year/fifty year)?
- How can a more uniform safety level be obtained?

As a basis of this discussion, knowledge on the prevailing safety level is needed. Several investigations have been performed to assess the reliability levels obtained by Eurocode partial factor design. The studies can be split in two categories: *direct studies*, which directly calculate the reliability level obtained by Eurocode partial factor design (e.g. Holický and Sýkora, 2009; Holický and Retief, 2005; Meinen et. al., 2017) and *indirect studies*, which calibrate the required partial factors to obtain specified target reliabilities (e.g. Sørensen, 2001; Sørensen and Hansen, 2015; Steenbergen et. al., 2012; Rózsás et. al. 2014). The direct studies generally focus on a subset of potential design situations only, where the indirect studies only provide (indirect) insight in the reliability levels by a comparison of the calibrated and prescribed partial factors. Moreover, most studies focus on either the one year reference period or the fifty year reference period, and the comparison between the two reference periods is generally not addressed.

*1.2 **Objective***

Currently there is a lack of insight in the safety levels obtained by Eurocode partial factor design. This hampers the provision of new target reliabilities, since they should (to a certain extent) be based on the prevailing safety level. Main objective of this study is to provide insight in the reliability levels obtained by current Eurocode EN1990:2002 partial factor design and to discuss these reliability levels in the light of current and future target reliabilities. To achieve this, a large number of typical design situations (ULS, CC2) are designed using the EN1990:2002 partial factor format and assessed on their structural reliability. The design situations cover different material types, failure modes, loading types and load ratios. Both the one year reference period and the fifty year reference period are assessed. The obtained reliability levels are compared with the currently prescribed target reliabilities. Additionally, the obtained reliability levels for the one year reference and the fifty year reference period are compared, and it is discussed which reference period is more suitable for future target reliabilities. As an illustration, the impact of an alternative partial factor format is investigated. Based on the results, recommendations for future codification actions are provided.

1.3 *Scope and limitations*

When assessing the reliability levels entailed by the Eurocodes, one should in principle adopt its (designated) probabilistic framework (see subsection 3.5). As was also recognized by (Caspeele et. al., 2015), such a designated framework is currently lacking for EN1990:2002. For the purpose of this study, we therefore established a general probabilistic framework which is assumed to adequately describe the typical design situations in scope of the Eurocodes (see subsection 3.5 and appendix A). Even though the results of the reliability calculations depend on the exact details of this framework, it is expected that the main conclusions of this study hold regardless of minor changes.

1.4 *Reading guide*

This paper is organized as follows. Section 2 describes the design situations which are taken into account in this study. Section 3 provides the details of the conducted reliability analyses. Section 4 entails a discussion on the interpretation of the currently prescribed target reliabilities. Section 5 presents the results of the conducted reliability analyses. Section 6 provides a discussion on the current and future reliability levels, and provides recommendations for a new safety format. Section 7 provides the conclusions and recommendations of this study. Appendix A is an extension to section 3 and presents the probabilistic framework adopted in this study. Appendix B is an extension of section 5, and provides a graphical overview of the results of all reliability analyses conducted in this study.

2 Design situations

To obtain a representative overview of the safety level embodied in the Eurocode (CC2, ULS), it is necessary to analyse a large number of design situations comprising various structural types, construction materials, load combinations and structural dimensions. For this study a selection is made of design situations which we deem the most important. Inspiration was gained from related studies conducted by Vrouwenvelder and Siemes (1987) and Sørensen (2001).

An overview of the selected design situations is provided in Table 1. A design situation regards the combination of a reference period, material type, failure mode, variable loading type, and the load ratios.

Two reference periods are considered: the one year reference period and the fifty year reference period. The reliability index calculated for the one year reference period corresponds to the failure probability in the first year of the lifetime of the structure.

Three material types are considered: structural steel, reinforced concrete and a generalized material type *m* with high variability in its resistance model and material characteristics (e.g. structural timber).

Two types of structural members (or failure modes) are considered: a simply supported beam loaded in bending and a (short) column loaded in axial compression (no buckling failure).

Two permanent loading types are distinguished: the self-weight of the structure G_{SW} and the imposed permanent loads G_P. Three variable loading types are considered: imposed loads Q_I , wind loads Q_W and snow loads Q_S . The permanent loads are combined with a single variable loading type only, i.e. combinations of variable loading types are not taken into account. All loads are assumed to act unfavourable on the structural member; favourable actions (such as the favourable effects of prestressing) are not taken into account.

*Table 1: Summary of design situations taken into account in this study. * Material type m is assessed for the bending failure mode only.*

ref. period	x	mat. type	x	fail. mode	x	var. load	x	χ_G	x	χ_Q
1 year		steel		bending		imposed		0-1		0-1
50 year		reinf. concrete		compression		wind		fine grid		fine grid
		mat. type *m**				snow				

A structure consists of a large number of structural elements, each of which subjected to different combinations of permanent and variable loads. To account for this, several load ratios between the permanent and variable loads are considered. The ratio between the variable load and the total loads is specified by the parameter χ_Q , such as defined by eq. (1). The ratio between the self-weight and the total permanent load is specified by the parameter χ_G , such as defined by eq. (2). To ease the implementation of the reliability calculations, the load ratios are specified on the level of the mean values (indicated by overbars). In case of the variable loading types, the mean-value corresponds either to the one year average or the fifty year average.

$$\chi_Q = \frac{\bar{Q}}{\bar{G}_{\mathrm{sw}} + \bar{G}_{\mathrm{P}} + \bar{Q}} \tag{1}$$

$$\chi_G = \frac{\bar{G}_{\mathrm{sw}}}{\bar{G}_{\mathrm{sw}} + \bar{G}_{\mathrm{P}}} \tag{2}$$

Theoretical values of χ_Q and χ_G lie within the range $[0, 1]$.

- In case $\chi_Q = 1$ the load consists of variable loads only. Design situations close to this load ratio are e.g. fastenings of façade members.
- In case $\chi_Q = 0$ the load consists of permanent loads only. Design situations close to this load ratio are e.g. structural members close to the foundation of the structure.
- In case $\chi_Q = 0$ and $\chi_G = 1$ the load consists of self-weight only. Design situations close to this load ratio are e.g. gravity-based structural elements.
- In case $\chi_Q = 0$ and $\chi_G = 0$ the load consists of imposed permanent loads only. Design situations close to this load ratio are e.g. structural elements in (steel) storage racks.

Within a consequence class, the reliability levels should preferably be uniformly distributed over the load ratios. This means that those structural elements loaded by permanent loads only ($\chi_Q = 0$) should have reliability levels close to structural elements loaded by variable loads only ($\chi_Q = 1$) and all load ratios in between. The same holds for structural elements loaded by self-weight ($\chi_G = 1$) or imposed permanent loads ($\chi_G = 0$) only.

Depending on the material type, not all load ratios are observed equally often in practice. For example steel members are more likely to have low χ_G values due to their beneficial self-weight versus strength ratio. In the same vein, concrete members are more likely to have relatively high χ_G values. When assessing the overall reliability level embodied in the Eurocode, this relative occurrence (i.e. the probability) of load ratios needs to be taken into account explicitly. Several references have attempted to specify relevant ranges of the variable load ratio χ_Q , of which an overview is provided in Figure 1. The ranges are however difficult to compare; some references provide ranges on the level of the mean values, others on the level of characteristic values, some have a specific variable loading type in mind, others have a 'general' variable loading type in mind.

Structural members come in all materials, dimensions and load combinations. To make sure we do not overlook certain structural members, we choose to analyse all load ratios $\chi_Q, \chi_G \in [0,1]$, representing all possible structural elements within a structure. To acknowledge the fact that not all load ratios are equally important in practice, we indicate the relevant variable load ranges as a function of material type (values chosen by expert judgement). For steel and material type *m* we take the relevant variable load range $\chi_{Q,\text{rel}} \in [0.2, 0.8]$. For concrete we take the relevant variable range $\chi_{Q,\text{rel}} \in [0.15, 0.7]$.

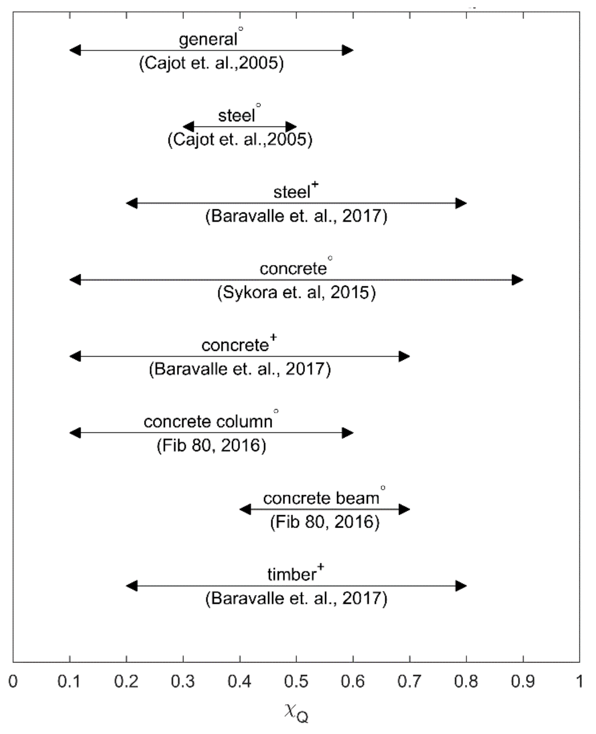

Figure 1: Variable load ratios such as recommended in the literature. Ranges marked with a cross (+) are defined by characteristic values. Ranges marked with a circle (o) are defined by mean values.

3 Reliability analysis

3.1 *Overview*

Starting point of the reliability analysis is a selected design situation describing all input parameters relevant for the design (see section 2). The design situation is expressed in terms of a limit state function which identifies the safe domain from the failure domain (see subsection 3.2). First step in the analysis is the design of the structural member according to the semi-probabilistic partial factor method as prescribed by the Eurocodes (see subsection 3.3). Subsequently the uncertainties in the parameters are taken into account by probabilistic models (see appendix A). The structural reliability and sensitivity factors are then calculated using the FORM reliability method (see subsection 3.4). Some brief notes on the chosen probabilistic framework are provided in subsection 3.5.

3.2 *Limit state function*

The generic limit state function (LSF) applied in this study is defined by:

$$z(\vec{X}) = R - E \tag{3}$$

where $z(.)$ represents the limit state function, $\vec{X}$ the vector of basic random variables, R the resistance of the structural member, and E the load effects exerted on the structural member.

The structural resistance is further specified by:

$$R = a_{\text{opt}}\mathrm{r}(\vec{X}_R)\theta_R \tag{4}$$

where $\mathrm{r}(.)$ represents the physical resistance model, $\vec{X}_R$ the basic random variables affecting the structural resistance (e.g. material properties), a_{opt} the design parameter obtained by Eurocode partial factor design (e.g. a cross-sectional area, see also subsection 3.3), and θ_R the model uncertainty factor accounting for the uncertainties in the physical resistance model.

The load effects are further specified by:

$$E = \theta_E\left(\left(1-\chi_Q\right)\left(\chi_G G_{\text{sw}} + (1-\chi_G)G_{\text{P}}\right) + \chi_Q Q_j\right) \tag{5}$$

where G_{SW} represents the self-weight of the structural member, G_{P} the imposed permanent loads, Q_j variable loading type *j* (either imposed, wind or snow), χ_Q the load ratio between the variable loads and the total loads (see eq. 1), χ_G the load ratio between the self-weight and the total permanent loads (see eq. 2), and θ_E is the model uncertainty factor accounting for the uncertainties in the load effect model.

The variable load is further specified by:

$$Q_j = q_j(\vec{X}_{Q,j})\theta_{Q,j} \tag{6}$$

where $q_j(.)$ represents the physical model for variable loading type *j*, $\vec{X}_{Q,j}$ the vector of basic random variables affecting variable loading type *j*, and $\theta_{Q,j}$ the model uncertainty factor accounting for the uncertainties in the load model for variable loading type *j*.

3.3 *Design of the structural member*

3.3.1 Safety format

For a sufficiently safe structure, the design value of the structural resistance needs to be larger than, or equal to, the design value of the load effects. In case of unity-design, the fundamental equation becomes:

$$R_{\mathrm{d}} = E_{\mathrm{d}} \tag{7}$$

Where R_{d} represents the design value of the structural resistance and E_{d} represents the design value of the load effects.

The design value of the structural resistance is defined by:

$$R_{\mathrm{d}} = a \cdot \mathrm{r}\left(\left\{\frac{X_{\mathrm{k},i}}{\gamma_{\mathrm{M},i}}\right\}\right), \qquad i \geq 1 \tag{8}$$

where $\mathrm{r}(.)$ represents the physical resistance model, $X_{\mathrm{k},i}$ the characteristic value of material property *i* (e.g. ultimate strength), $\gamma_{\mathrm{M},i}$ the partial factor for the material property *i*, and *a* the design parameter. R_{d} can also directly be obtained from experiments without specific definition of characteristic values and partial factors.

The design value of the load effects E_d is obtained by the fundamental load combinations provided by EN1990:2002 article 6.4.3.2. formulas 6.10a and 6.10b. For persistent and transient design situations, and for STR (strength of construction material is governing) and GEO (strength of soil or rock governs) limit states, the fundamental load combinations for unfavourable actions are obtained by:

$$E_{\text{d,a}} = (1-\chi_Q)(\chi_G \gamma_G G_{\text{sw,k}} + (1-\chi_G)\gamma_G G_{\text{P,k}}) + \chi_Q \gamma_Q \Psi_{0,j} Q_{j,\text{k}} \tag{9}$$

$$E_{\text{d,b}} = (1-\chi_Q)(\chi_G \xi \gamma_G G_{\text{sw,k}} + (1-\chi_G)\xi \gamma_G G_{\text{P,k}}) + \chi_Q \gamma_Q Q_{j,\text{k}} \tag{10}$$

$$E_\text{d} = \max\left(E_{\text{d,a}}, E_{\text{d,b}}\right) \tag{11}$$

where $G_{\text{sw,k}}$ is the characteristic value of the self-weight, $G_{\text{P,k}}$ the characteristic value of the permanent actions, $Q_{j,\text{k}}$ the characteristic value of variable loading type *j*, γ_G the partial factor for permanent loads, γ_Q the partial factor for variable actions, ξ the reduction factor for unfavourable permanent actions, and $\Psi_{0,j}$ the load combination factor for variable loading type *j*. The partial factors and load combination factors are prescribed by the Eurocodes (see Table 2).

3.3.2 Design procedure

The design of the structural member goes as follows. First the design value of the load effects is determined using equations (9) to (11). Subsequently the value of E_d is substituted in the fundamental equation (7), as well as the physical model for the design value of the structural resistance equation (8). The optimal value of the design parameter a_opt is obtained by solving the resulting equation for the design parameter *a*.

It is remarked that, in this study, the design of the structural member is established by the optimization of the geometrical properties only (e.g. moment of inertia, or effective cross-section); the material properties (e.g. characteristic values of yield strength, ultimate strength, modulus of elasticity) are assumed to be chosen on beforehand.

Table 2: Safety format for CC2, ULS such as adopted in this study

symbol	description	value	remark
$\gamma_{M,s}$	partial factor for structural steel, also accounting for model uncertainties and dimensional variations	1.00	EN1993-1-1:2005
$\gamma_{M,c}$	partial factor for concrete, also accounting for model uncertainties and dimensional variations	1.50	EN1992-1-1:2004
$\gamma_{M,rs}$	partial factor for reinforcement steel, also accounting for model uncertainties and dimensional variations	1.15	EN1992-1-1:2004
$\gamma_{M,m}$	partial factor for material type *m* with high resistance uncertainty, also accounting for model uncertainties and dimensional variations	1.30	typical value for material with high variability, based on expert judgement
γ_G	partial factor for permanent actions	1.35	EN1990:2002
γ_Q	partial factor for variable actions	1.50	EN1990:2002
ξ	reduction factor for unfavourable permanent actions	0.85	EN1990:2002
$\Psi_{0,I}$	load combination factor for imposed loads	0.70	EN1990:2002 category B: office areas
$\Psi_{0,W}$	load combination factor for wind loads	0.60	EN1990:2002
$\Psi_{0,S}$	load combination factor for snow loads	0.70	EN1990:2002

3.4 Reliability analysis

After the design of the structural member is completed, the uncertainties in the parameters are taken into account by probabilistic models (see appendix A). Subsequently the failure probability is calculated, which is defined as the probability that the LSF is below zero:

$$P_{\mathrm{f}} = \mathrm{P}\left(z(\vec{X}) \leq 0\right) \tag{12}$$

The reliability index is then calculated by:

$$\beta = -\Phi^{-1}(P_{\mathrm{f}}) \tag{12}$$

Where $\Phi(.)$ stands for the standard normal distribution.

Several methods exist for the evaluation of the limit state function. For the purpose of this study we adopt the First Order Reliability Method (FORM) as it is relatively fast and additionally provides insight in the sensitivity factors of the basic random variables involved in the analysis. Sensitivity factors (or α-values) describe the sensitivity of the calculated structural reliability as a function of the uncertainties in random variables. To study the assumptions inherent to the FORM reliability method (among others the linearization of the limit state function at the design point) a selected number of design situations was also assessed using Monte Carlo simulations. The reliability calculations are performed with the TNO owned reliability program Prob2B (2018).

*3.5 **Probabilistic framework***

The total of physical and probabilistic models in a reliability analysis is referred to as the probabilistic framework. Different reliability studies apply different probabilistic frameworks and therefore the calculated reliability levels will generally be at odds. For this reason, calculated reliability levels are often referred to as *notational probabilities*; probabilities with only a formal meaning.

When assessing the structural reliability entailed by the Eurocodes, one should therefore also adopt the probabilistic framework accordingly. However, such a formal framework is currently lacking (see also Caspeele et. al., 2015). Based on recommendations found in literature, expert judgement and measurement data analysed by the authors, a general probabilistic framework is proposed for the assessment of reliability levels in the context of the Eurocodes. For a complete overview and substantiation of the applied probabilistic models it is referred to appendix A. Each of the random variables is defined by (a) the distribution type; (b) the distribution moments (mean, coefficient of variation) and; (c) the characteristic fractile defining the probability of non-exceedance of the characteristic (or specified) value such as prescribed by the Eurocode.

Some remarks with respect to the adopted probabilistic framework:

- *Normalized random variables*: For an easy parametric analysis on the load ratios the concept of normalized random variables is adopted. Thereby for each of the random variables the mean value is set equal to one and the standard deviation is set equal to the coefficient of variation (CoV). This approach does not affect the outcomes of the reliability analysis.

- *Structural resistance*: The structural resistance is modelled by physical models as prescribed by the Eurocodes. This in contrast to most calibration studies, where the structural resistance is modelled as a single random variable only (see e.g. Sørensen, 2001; Meinen et. al., 2017).
- *Model uncertainties*: For each of the physical models applied in this study it is explicitly accounted for model uncertainties. It is distinguished between model uncertainties for the structural resistance, the loads, and the load effects.
- *Uncertainties geometric properties*: It is not explicitly accounted for uncertainties in the geometrical properties of the structural members, as these are assumed to be small as compared to the uncertainties in the other basic random variables.
- *Case-dependency*: For wind and snow loads, the probabilistic models strongly depend on the geographical location of the structure. In those situations, it was aimed at having a general probabilistic description corresponding to the 'average' design situation.
- *Time-dependencies*: Time-dependent effects such as climate change or material degradation are not taken into account in the probabilistic framework.

4 Interpretation of target reliabilities

In the following sections we will discuss the reliability levels obtained by Eurocode EN1990:2002 factor design in the light of current and future target reliabilities. For a fair discussion, we first need an unambiguous interpretation of these target reliabilities, which is often a topic of discussion in the scientific community.

One of the discussions involves the question whether the prescribed target reliabilities should be considered as *minimum* or *average* target values. On the one hand EN1990:2002 refers to the target reliabilities as "recommended minimum values" and states that "a design using EN1990 […] is considered to […] lead to a structure with a β value greater than 3.8 for a 50 year reference period". In this case the target reliability is considered as a minimum value to which each individual structure should be assessed against. On the other hand, the '*Designers Guide to EN1990*' (Gulvanessian et. al., 2002) states that "a certain proportion of construction works may have a β value less than 3.8 in 50 years" and that "many engineers consider that this value should be a target value for the calibration of […] partial factors." In that case the target reliability is more interpreted as an 'average' target value, whereby some scatter around this average value is allowed (yet the magnitude of this allowed scatter is not mentioned).

It is readily reasoned that in fact both a minimum *and* an average target reliability are important in practice. As a reference, ISO 2394:2015 explicitly prescribes average reliability levels based on economic optimization (depending on the cost of safety measures) *and* minimum reliability levels based on life-safety considerations. Moreover, an indication of the maximum allowed scatter should be provided (e.g. in terms of the maximum range or standard deviation); not only to prevent undesirably low reliability levels within a certain consequence class, but also to ensure that structures are not designed much safer (and more expensive) than originally aimed at. In the following sections, we will therefore compare the obtained reliability levels with both the minimum ($\beta_{t,min}$), maximum ($\beta_{t,max}$), and average ($\beta_{t,av}$) target reliabilities. Since EN1990:2002 does not specify these values explicitly, the following was reasoned. Table 3 provides the target reliabilities for the ULS such as recommended by EN1990:2002 as function of consequence class and reference period. It is observed that the target reliabilities for each consecutive consequence classes differ $\Delta\beta = 0.5$ in the prescribed reliability index. In case the prescribed target values are interpreted as average target reliabilities ($\beta_{t,av}$), it could be reasoned that each consequence class covers structural members with reliability levels between $\beta_{t,av} \pm 0.25$. Accordingly, the maximum and minimum reliability levels within a consequence class are $\beta_{t,max} = \beta_{t,av} + 0.25$ and $\beta_{t,min} = \beta_{t,av} - 0.25$ respectively. Structures outside these boundaries will automatically fall in the superior or the subordinate consequence class.

A second discussion regards the appropriateness of the currently prescribed target reliability in EN1990:2002 for the one year reference period ($\beta_{t,1yr} = 4.7$), which is derived from the fifty year target reliability assuming mutually independent annual failure events (Vrouwenvelder, 2002). For many design situations however, this assumption is not realistic, and leads to a conservative estimation of the annual target reliability. This especially holds for design situations which are governed by uncertainties in the time-independent random variables (e.g. those related to the permanent loads or structural resistance). For these reasons, alternative, (lower) yearly target reliabilities have been proposed in the literature; e.g. Vrouwenvelder (2002) states for CC2 that “the Eurocode target could better be interpreted as corresponding to $\beta_t = 4.5$ for one year” and the Danish National Annex of EN1990:2002 adapted the recommended value to $\beta_{t,1yr} = 4.3$.

The *Probabilistic Model Code* (JCSS, 2001) recommends a yearly target reliability of $\beta_{t,1yr} = 4.2$, which was also found to be largely compatible with observed failure rates and with outcomes of cost-benefit analyses (Vrouwenvelder, 2002).

It is observed that the annual target reliability proposed by EN1990:2002 ($\beta_{t,1yr} = 4.7$) is clearly on the conservative side compared to these alternative codes and standards. Combined with the fact that the Eurocode partial factors are generally derived for the fifty year reference period, this target value will therefore most likely not be achieved for many design situations in scope of the Eurocodes (and therefore in scope of this study). For a meaningful comparison, we will therefore compare the outcomes of the reliability analysis with the value of $\beta_{t,1yr} = 4.2$ instead of the recommended value $\beta_{t,1yr} = 4.7$. The results from section 5 also show that this lower value is indeed more realistic.

Table 3: Recommended target reliabilities for ULS as a function of consequence class and reference period (EN1990:2002)

Consequence class	1 year reference period	50 year reference period
CC1	5.2	4.3
CC2	4.7	3.8
CC3	4.2	3.3

5 Results of the reliability analyses

5.1 Introduction

The reliability calculations are conducted for each of the design situations in scope of this study. In total over 3000 design situations have been investigated, resulting in an equal amount of reliability levels and an even larger number of sensitivity factors. It is impossible to analyse all design situations individually. As an illustration, this paper discusses the reliability results and sensitivity factors obtained for a steel member in bending (see subsection 5.3) and concrete member in bending (see subsection 5.4) in detail; the results of the other design situations are presented as a graphical summary (see subsection 5.5). For the details on the results of the other design situations it is referred to appendix B. Subsection 5.6 provides a summary of the most important findings.

5.2 *Measures of interest*

5.2.1 *Reliability levels*

With respect to the reliability levels, the following measures of interest are investigated (for the definition of the load ratios see section 2):

- the average reliability levels obtained by unweighted averaging over the relevant load ratios (both χ_Q and χ_G), referred to as β_{av};
- the overall maximum and minimum reliability levels obtained by taking the maximum or minimum values over all load ratios, referred to as $\beta_{\max}$, $\beta_{\min}$ respectively;
- the scatter in reliability levels around the average reliability levels.

5.2.2 *Sensitivity factors*

With respect to the sensitivity factors, focus lies on the equivalent sensitivity factors for the structural resistance ($\alpha_{R,\text{equiv}}$) and load effects ($\alpha_{E,\text{equiv}}$). The equivalent sensitivity factors describe the *combined effects* of the uncertainties in the random variables on the resistance or loading side respectively, and are determined by:

$$\alpha_{\text{equiv}} = \sqrt{\sum_{i=1}^{n} \alpha_i^2} \tag{14}$$

Where α_i is the sensitivity factor corresponding to basic random variable X_i in the resistance or load effect model and n is the total number of random variables in either the resistance or load effect model. For brevity the subscript 'equiv' is omitted in the rest of this paper.

Main interest goes to the *maximum obtained sensitivity factors* rather than the average values or minima. This is because higher sensitivity factors relate to more conservative design values and are therefore more interesting from a standardization point of view. It is however remarked that the obtained (maximum) sensitivity factors cannot directly be compared with the (ISO-) standardized values such as mentioned in EN1990:2002, since the obtained values in this study do not necessarily lead to the same target reliability.

5.3 *Steel member in bending*

5.3.1 Reliability levels

Figure 2 shows the calculated reliability levels for a steel member in bending, loaded by self-weight, permanent loads and imposed loads (left), wind loads (middle) or snow loads (right) as a function of the variable load ratio χ_Q (horizontal axes) and permanent load ratio χ_G (distinguished by line type). The reliability levels for the one and fifty year reference period are presented in red and blue respectively. The relevant variable load range is indicated by the horizontal double arrow (see section 2). The average reliability levels are displayed by horizontal, solid lines. The figure shows the results for the permanent load ratios $\chi_G = 0$ and $\chi_G = 1$ only, the reliability levels obtained for the other χ_G -values were within these boundaries.

Load ratios

As addressed in section 2, the reliability levels should preferably be uniformly distributed over the distinct load ratios. From a bird-eye perspective however, it can be found that for both reference periods and for all three variable loading types the calculated reliability levels vary significantly over the distinct load ratios; for the fifty year reference period more than for the one year reference period. For variable load ratios below $\chi_Q < 0.2$, the reliability levels obtained for self-weight only ($\chi_G = 1$, dotted line) lie systematically above those obtained for imposed permanent loads only ($\chi_G = 0$, dashed line). This is attributed to the fact that the CoV of the self-weight for steel lies below the CoV of the imposed permanent loads, while the distribution type, characteristic fractile and prescribed partial factor are identical (see appendix A). For variable load ratios above $\chi_Q > 0.2$, the effect of the permanent load ratio χ_G becomes negligible and the effect of the variable load ratio χ_Q becomes more apparent; the reliability systematically decreases with increasing variable load ratio χ_Q . This behaviour is especially observed in case of snow loads. For design situations covering permanent loads only ($\chi_Q = 0$), the obtained reliability levels are identical for the one year reference period and the fifty year reference period. This makes sense, since the permanent loads are time (and reference period) independent.

Average reliability levels

The average reliability levels are indicated by horizontal, solid lines. They refer to the relevant load ratios only. Irrespective of the reference period, the highest average reliability levels are found for imposed loads ($\beta_{av,50yr} \approx 3.6$, $\beta_{av,1yr} \approx 4.0$) followed by wind loads ($\beta_{av,50yr} \approx 2.8$, $\beta_{av,1yr} \approx 3.7$) , followed by snow loads ($\beta_{av,50yr} \approx 2.0$, $\beta_{av,1yr} \approx 3.0$). For all variable loading types, the scatter in reliability levels around the average is larger for the fifty year reference period than for the one year reference period. This especially holds for snow loads.

Fifty year reference period

For imposed loads and the fifty year reference period, the Eurocode partial factor format leads to an average reliability level close to the target value of $\beta_{t,av,50yr} = 3.8$. The scatter around the average is relatively small (the majority of the design situations lies within $\beta_{av,50yr} \pm 0.25$). This means that for steel structures designed according to the Eurocodes and loaded by self-weight, permanent loads and imposed loads, the target $\beta_{t,av,50yr} = 3.8$ is (by approximation) satisfied as being an average target. This does not hold for steel structures under wind or snow load, where the average reliability levels lie below the average target value and where moreover the scatter around the average value is larger (a large part of the design situations lies outside $\beta_{av,50yr} \pm 0.25$).

One year reference period

For imposed loads and the one year reference period, the Eurocode partial factor format leads to an average reliability level that is approximately equal to the value of $\beta_{t,av,1yr}$ = 4.2 (slightly lower). For wind and snow loads, the average reliability levels lie relatively far below the defined target value. For each of the variable loading types, the scatter around the average value is relatively small (the majority of the design situations lies within $\beta_{av,1yr} \pm 0.25$).

Overall maximum and minimum reliability levels

For both reference periods and all three variable loading types, the overall maximum reliability levels correspond to design situations governed by self-weight (χ_G = 1, χ_Q < 0.2). For the one year reference period, the overall minimum reliability levels are found either for design situations with imposed permanent loads only (imposed loads, wind loads) or for design situations with variable load only (snow loads). For the fifty year reference period the overall minimum reliability levels correspond to design situations with variable loads only (χ_Q = 1).

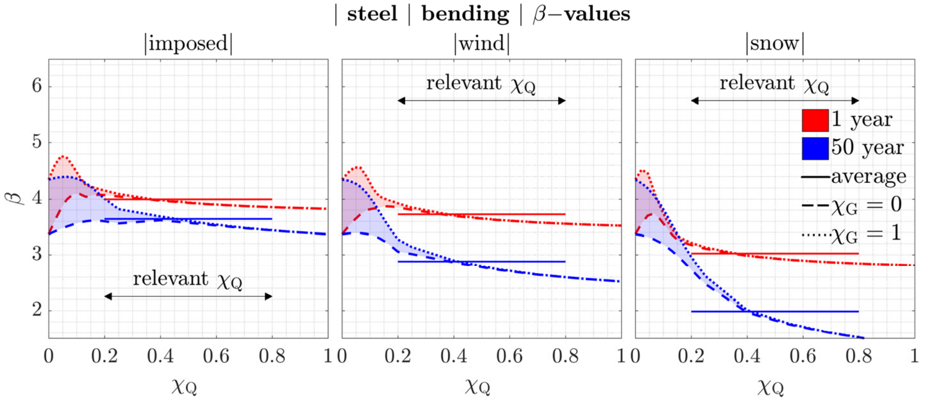

Figure 2: Calculated reliability levels for a steel member in bending for the one and fifty year reference period

5.3.2 Sensitivity factors

Figure 3 and Figure 4 show the equivalent sensitivity factors for the structural resistance and load effects for the steel member in bending. The factors are strongly dependent on the variable load ratio (χ_Q), and to a lesser extent on the permanent load ratio (χ_G), of which the effects become negligible for variable load ratios above $\chi_Q > 0.2$.

Structural resistance

The sensitivity factors for the structural resistance decrease with increasing variable load ratio χ_Q. This is rather trivial, since an increasing variable load ratio χ_Q means a relative increase in the (uncertainties in the) variable loads, and therefore a relative decrease of uncertainties in the structural resistance. The sensitivity factors for the one-year reference period lie systematically below those for the fifty-year reference period.

Load effects

The equivalent sensitivity factors for the load effects increase systematically with increasing variable load ratio load ratio χ_Q. Also this is rather trivial. The values found for the fifty-year reference period lie systematically below those for the one-year reference period, yet the differences are small.

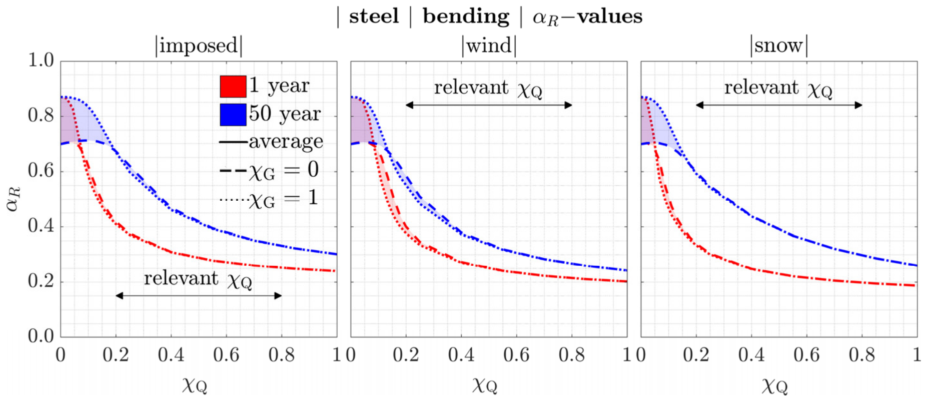

Figure 3: Equivalent sensitivity factors of the structural resistance for a steel member in bending for the one and fifty year reference period

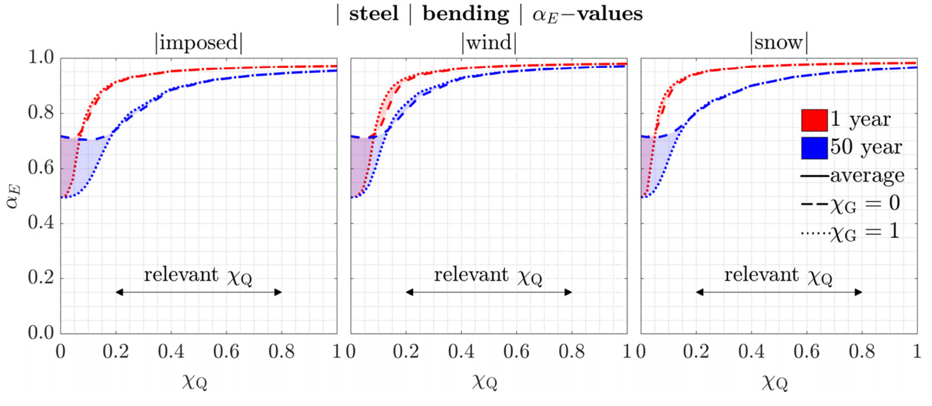

Figure 4: Equivalent sensitivity factors of the load effects for a steel member in bending for the one and fifty year reference period

5.4 *Concrete member in bending*

5.4.1 Reliability levels

Figure 5 shows the obtained reliability levels for a concrete member in bending. The behaviour is similar to that of the steel member in bending; the reliability levels strongly depend on the design situation (load ratios, variable loading type); the design situations with self-weight only lie systematically above those with imposed permanent loads only; for variable load ratios above $\chi_Q > 0.2$ the reliability levels systematically decrease with increasing variable load ratio; and the scatter in reliability levels around the average is larger for the fifty year reference period than for the one year reference period. Again, the highest average reliability levels are found for imposed loads ($\beta_{av,1yr} \approx 4.4$, $\beta_{av,50yr} \approx 4.2$), followed by wind loads ($\beta_{av,1yr} \approx 4.1$, $\beta_{av,50yr} \approx 3.3$), followed by snow loads ($\beta_{av,1yr} \approx 3.3$, $\beta_{av,50yr} \approx 2.5$).

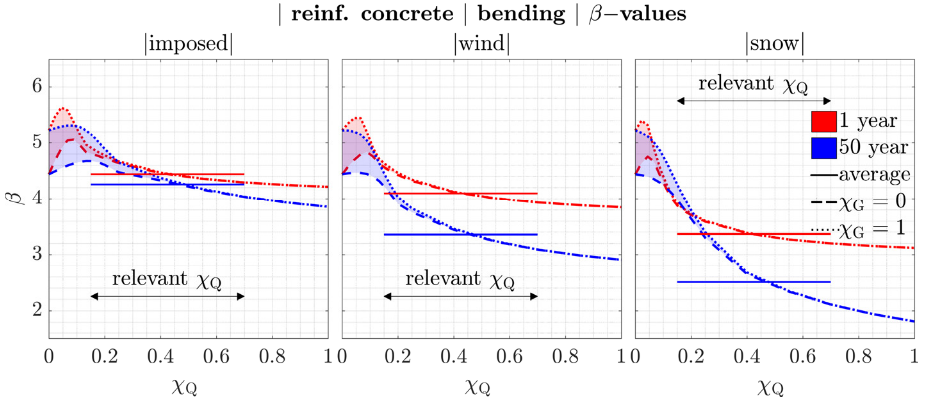

Figure 5: Calculated reliability levels for a reinforced concrete member in bending for the one and fifty year reference period

Fifty year reference period

In case of the fifty year reference period and for imposed loads, both the average reliability levels and the minimum reliability levels lie above the target average of $\beta_{t,av,50yr} = 3.8$. This means that for concrete structures designed according to the Eurocodes and loaded by self-weight, permanent loads and imposed loads, (by approximation) the target $\beta_{t,av,50yr} = 3.8$ is satisfied as being a minimum target. In case of wind and snow loads however, the average reliability levels lie below the target average. In case of imposed loads, the scatter around the average value is relatively small, and a large part of the relevant design situations lies within the boundaries of $\beta_{av,50yr} \pm 0.25$. In case of wind and snow loads the scatter around the average value is larger, and a large part of the design situations lies outside of boundaries of $\beta_{av,50yr} \pm 0.25$.

One year reference period

In case of the one year reference period and for imposed loads, the average reliability level lies slightly above the target average of $\beta_{t,av,1yr} = 4.2$. In case of wind and snow loads, the average reliability levels lie below the target value. For all three variable loading types the scatter in reliability levels over the relevant load ratios is small and a large part of the design situations lies within $\beta_{av,1yr} \pm 0.25$.

5.4.2 Sensitivity factors

Figure 6 and Figure 7 show the equivalent sensitivity factors for the structural resistance and the load effects for the reinforced concrete member in bending. A similar behaviour is observed as for the steel member in bending; both qualitatively and quantitatively. The findings are therefore not further discussed.

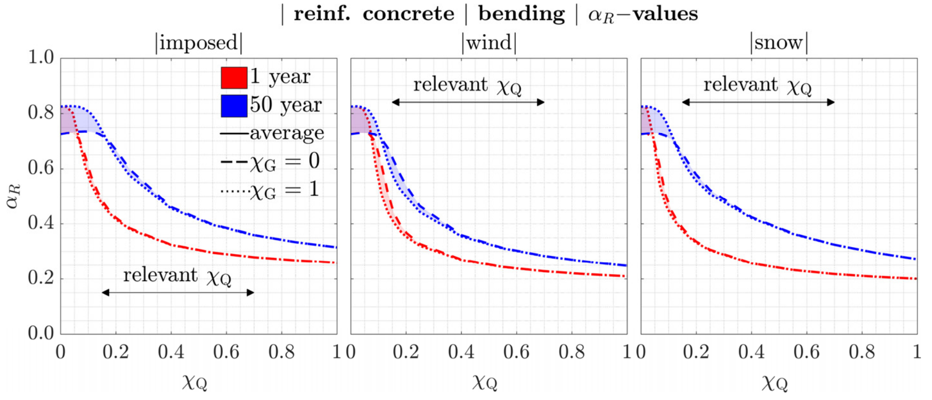

Figure 6: Equivalent sensitivity factors for a concrete member in bending for the one and fifty year reference period for the structural resistance

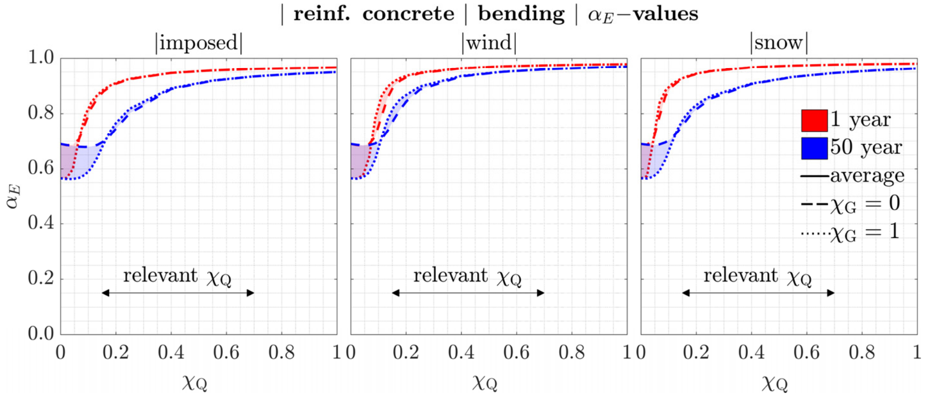

Figure 7: Equivalent sensitivity factors for a concrete member in bending for the one and fifty year reference period for the load effects

5.5 *Other design situations*

5.5.1 *Reliability levels*

A similar analysis is conducted for all other design situations, of which a graphical summary is provided in Figure 8. The color-coding is identical to that from the previous figures. The capital letter I stands for imposed loads, W for wind loads and S for snow loads. For each combination of material type and variable load ratio, the range of reliability levels over *all load ratios* is indicated by a thin vertical line. The range of reliability levels over the *relevant load ratios* is presented by a thick vertical line. The average reliability levels (over the relevant load ratios) are represented by a thick horizontal white stripe. As a comparison, the average target reliabilities are displayed as horizontal, dotted lines. The load ratios corresponding to the overall maximum and minimum reliability levels are marked by symbols. In case the variable load ratio is below $\chi_Q < 0.2$, this is marked with an open circle. In case of variable loads only ($\chi_Q = 1$), this is marked with an open diamond. In case of imposed permanent loads as permanent loads only ($\chi_G = 0$) this is marked by a cross. In case of self-weight as permanent loads only ($\chi_G = 1$) this is marked by an asterisk.

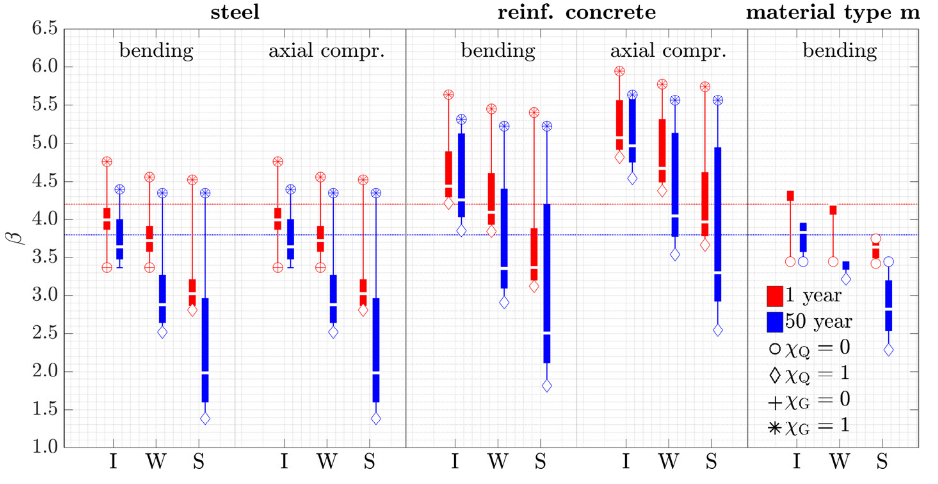

Figure 8: Range of reliability levels obtained for the one year reference period and the fifty year reference period as a function of the design situation

General

The calculated reliability levels differ strongly for the distinct design situations. For the fifty year reference period, reliability levels are found between $\beta_{50yr} \approx 1.4$ (steel, bending, snow) and $\beta_{50yr} \approx 5.5$ (reinforced concrete, axial compression, imposed). For the one year reference period, reliability levels are found between $\beta_{1yr} \approx 2.7$ (steel, bending, snow) and $\beta_{1yr} \approx 5.9$ (reinforced concrete, axial compression, imposed). The results for the steel member in bending are identical to those of steel member in axial compression. This is attributed to the fact that both the physical and probabilistic models of these design situations are equivalent (see appendix A). The scatter in reliability levels over the relevant variable load ratios is significantly larger for the fifty year reference period than for the one year reference period.

Average reliability levels (and comparison with average target reliabilities)

Figure 8 shows a considerable difference in average reliability levels for the distinct design situations. Regardless of the material type, failure mode or reference period, the highest average reliability levels are obtained for design situations with imposed loads (I), followed by wind loads (W), followed by snow loads (S). The average reliability levels for the concrete member lie systematically above those for the steel member, regardless of failure mode, variable loading type or reference period. The same holds for the comparison between the steel member and the member with high variability (material type *m*).

- *Fifty year reference period*

  In case of the fifty year reference period and for imposed loads, the average reliability levels lie close to or above the target value of $\beta_{t,av,50yr} = 3.8$, regardless of material type and failure mode. Also for wind loads some combinations of material type and failure mode result in average reliability levels above the target average. In case of snow loads however, the average reliability levels lie systematically below the average target values, regardless of the material type or failure mode. For almost all combinations of material type, failure mode and variable loading type the scatter in reliability levels (over the relevant load variable ratios) is relatively large, and a large number of the design situations results in reliability levels outside $\beta_{av,50yr} \pm 0.25$.

- *One year reference period*

  In case of the one year reference period and for imposed loads, the average reliability levels lie close to or above the target value of $\beta_{t,av,1yr} = 4.2$ regardless of material type or failure mode. In case of wind loads, this only holds for reinforced concrete and material type *m*. In case of snow loads, the average reliability levels lie systematically below the target value, regardless of material type and failure mode. For all design situations the scatter in reliability levels around the average is relatively small, and a large part of the relevant design situations lies within $\beta_{av,1yr} \pm 0.25$. The scatter for the reinforced concrete member is found to be larger than for the member conducted from steel or material type *m*.

Overall maximum and minimum reliability levels

For both steel and reinforced concrete the maximum reliability levels are found for design situations where self-weight is governing ($\chi_G = 1, \chi_Q < 0.2$). In case of the one year reference period the minimum reliability levels are found either for situations where imposed permanent loads are governing ($\chi_G = 0, \chi_Q < 0.2$), or where variable loads are governing ($\chi_Q = 1$). In case of the fifty year reference period and for wind and snow loads, the overall minimum reliability levels are found for design situations with variable loads only ($\chi_Q = 1$).

5.5.2 *Sensitivity factors*

Figure 9 and Figure 10 present graphical summaries of the obtained equivalent sensitivity factors for the structural resistance and load effects respectively. The layout of the figures is identical to that of Figure 8. In contrast to Figure 8, rather regular patterns are observed.

Structural resistance

- *All variable load ratios*

  Considering all variable load ratios (thin vertical lines), the maximum sensitivity factors for steel and concrete correspond to design situations with self-weight only ($\chi_Q = 0, \chi_G = 1$). Also for material type *m* the maximum sensitivity factors correspond to design situations with permanent loads only ($\chi_Q = 0$), however the ratio between self-weight and imposed permanent loads slightly deviates from $\chi_G = 1$ (see appendix B). Since the maximum values are obtained for design situations with permanent loads only, the maximum values for the one year reference period and fifty year reference period are identical. For the steel member, the maximum sensitivity factor is $\alpha_{R,\max} \approx 0.85$, both for bending and axial compression. For the reinforced concrete member, the maximum sensitivity factor is $\alpha_{R,\max} \approx 0.82$ for bending and $\alpha_{R,\max} \approx 0.92$ for axial compression. For material type *m* in bending the maximum sensitivity factor is $\alpha_{R,\max} \approx 0.94$.

- *Relevant variable load ratios*

  Considering the relevant variable load ratios only (thick vertical lines), the maximum sensitivity factors are significantly lower than those corresponding to all load ratios. In case of the one year reference period, the maximum sensitivity factor for steel is $\alpha_{R,\max} \approx 0.40$ (bending, imposed), for concrete $\alpha_{R,\max} \approx 0.65$ (axial compression, imposed) and for material type *m* $\alpha_{R,\max} \approx 0.88$ (bending, wind). In case of the fifty year reference period, the maximum value for steel is $\alpha_{R,\max} \approx 0.75$ (bending, imposed), for concrete $\alpha_{R,\max} \approx 0.82$ (axial compression, imposed) and for material type *m* $\alpha_{R,\max} \approx 0.88$ (bending, imposed). The range of sensitivity factors over the relevant load ratios is significantly smaller for the one year reference period than for the fifty year reference period. This is interesting from a standardization point of view (see also section 6.2).

Load effects

Figure 9 shows that the maximum sensitivity factors for the load effects are relatively high as compared to those for the structural resistance. The maximum values over all variable load ratios are found to be (approximately) equal to those for the relevant load ratios only, which is attributed to the fact that for variable load ratios above $\chi_Q > 0.5$ the graph of α_E is almost horizontal (see e.g. Figure 4 and Figure 7). Also, the values for the one year reference period and the fifty year reference period lie close to each other (fifty year reference period slightly below). For both steel and concrete, the maximum sensitivity factors for the load effects are above $\alpha_{E,\max} > 0.95$. In case of material type *m* the maximum value is $\alpha_{E,\max} \approx 0.80$. Again, the range of sensitivity factors over the relevant load ratios is smaller for the one year reference period than for the fifty year reference period.

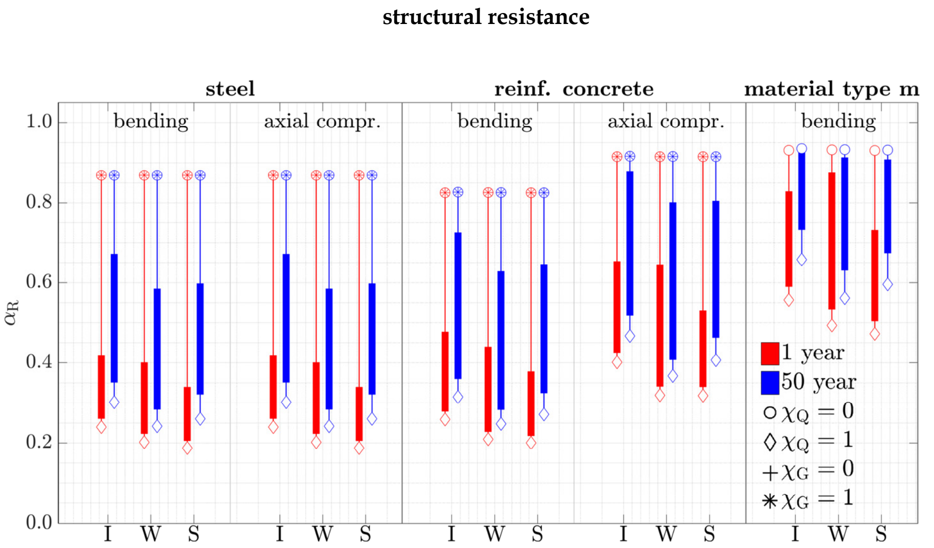

Figure 9: Range of equivalent sensitivity factors for the structural resistance for the one year reference period and the fifty year reference period as a function of the design situation

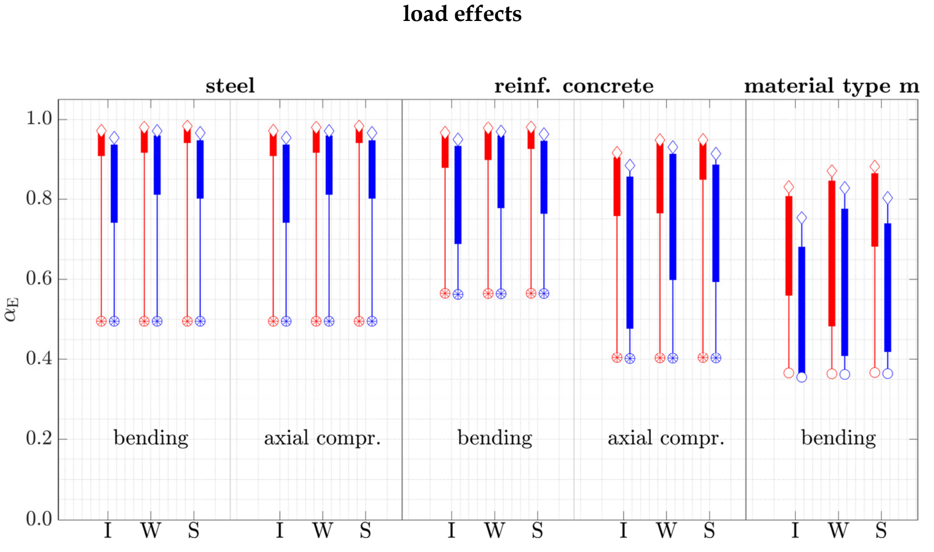

Figure 10: Range of equivalent sensitivity factors for the load effects for the one year reference period and the fifty year reference period as a function of the design situation

5.6 *Summary*

With respect to the reliability levels the following was found:

1. *General*: The obtained reliability levels are strongly dependent on the material type, failure mode, variable loading type and load ratios.
2. *Imposed loads*: For concrete structures loaded by both permanent and imposed variable loads, the target reliabilities are satisfied as a minimum; both for the one year reference period and the fifty year reference period. In case of steel and material type *m*, the target reliability is (approximately) satisfied on the average for this situation.
3. *Comparison variable loading types*: The average reliability levels found for imposed variable loads lie systematically above those for wind loads, followed by snow loads, irrespective of material type and failure mode.
4. *Comparison permanent loading types*: For both steel and reinforced concrete, the design situations with self-weight as dominant permanent loading type lie systematically above those with imposed permanent loads as dominant permanent loading type.
5. *Minimum reliability levels*: The overall minimum reliability levels are generally found for design situations covering variable loads only. In case of steel and reinforced concrete, the reliability levels systematically decrease with increasing variable load ratio (for $\chi_Q > 0.2$).
6. *Average reliability levels*:
    a. *Fifty year reference period*: In case of imposed loads, the average reliability levels lie close to or above the fifty year target value of $\beta_{t,av50}$ = 3.8, regardless of material type or failure mode. In case of wind loads, this holds for reinforced concrete in axial compression only. In case of snow loads, the average reliability levels lie systematically below the average target values, regardless of the material type or failure mode.
    b. *One year reference period*: In case of imposed loads, the average reliability levels lie close to or above the yearly target value of $\beta_{t,av1}$ = 4.2. In case of wind loads, this holds for reinforced concrete and material type *m* only. In case of snow loads, the average reliability levels lie systematically below the target value, regardless of material type or failure mode.
7. *Scatter around average reliability levels*: The scatter around the average reliability levels over the relevant load ratios is systematically smaller for the one year reference period

than for the fifty year reference period. For many design situations the scatter in reliability levels over the variable load ratios is larger than $\beta_{av} \pm 0.25$.

With respect to the equivalent sensitivity factors, the most important findings are:

8. *Variable load ratio*: The sensitivity factors for the structural resistance systematically decrease with increasing variable load ratio, whereas the sensitivity factors for the load effects systematically increase with increasing variable load ratio.
9. *Permanent load ratio*: For variable load ratios above $\chi_Q > 0.2$, the effects of the permanent load ratio becomes negligible. This holds for both the structural resistance and the load effects.
10. *Reference periods*: In case of the structural resistance, the sensitivity factors for the one year reference period lie systematically below those for the fifty year reference period. In case of the load effects, the sensitivity factors for the one year reference period lie systematically above those for the fifty year reference period. Considering the relevant variable load ratios only, the range of sensitivity factors for the one year reference period is smaller than for the fifty year reference period.

6 Discussion

The previous section sketched an overview of the reliability levels entailed by current Eurocode (EN1990:2002) partial factor design. It was observed that the reliability levels were strongly dependent on the considered design situation (points 1-4). Additionally it was observed that for both the one year reference period and the fifty year reference period the (average) target reliabilities were approximately met for imposed loads, where for wind and snow loads many design situations resulted in reliability levels below the target values (point 6). Moreover, for many material types and failure modes the scatter in reliability levels over the different load ratios was found to be large (point 7).

To deal with the differences between the obtained reliability levels and the prescribed reliability levels, the code-maker has in fact three options, which will be discussed below:

1. To **accept** the differences between the currently prescribed (average) target reliabilities in EN1990:2002 and the (average) reliability levels obtained by current Eurocode (EN1990:2002) partial factor design.

2. To **match** the future EN1990:2020 (average) target reliabilities with the (average) reliability levels such as obtained by current Eurocode (EN1990:2002) partial factor design.
3. To **recalibrate** the partial factors to the prescribed (average) target reliabilities.

From a consistency point of view, the first option is undesirable. As was addressed in the introduction, EN1990:2002 allows for the application of both full probabilistic and semi-probabilistic design methods (i.e. the partial factor method). In principle, both methods should be equivalent and (approximately) lead to the same reliability levels. However, the results of this study showed that current Eurocode (EN1990:2002) partial factor design in many cases results in reliability levels above or below the target value. When the same members would be designed using full-probabilistic design methods, they would be assessed against the prescribed target reliability, which (in those cases) lies higher or lower than the values obtained by the partial factor design. This establishes an unfair and undesirable competition between the semi- and full-probabilistic design methods.

With respect to the second option, the following should be kept in mind. The results in this study showed that systematic differences in (average) reliability levels exist between different material types, failure modes and variable loading types (see Figure 8). When matching the future (average) target reliabilities with the currently obtained (average) values, this inevitably results in different target reliabilities for different design situations.

The matching of the (average) reliability levels with the future (average) target values does not yet solve the problem of the large scatter in reliability values over the different load ratios. This leaves the third option; recalibration of the partial factors. Often this option is feared, since this will affect the current (established) design procedures. Especially the recalibration of the material dependent Eurocodes is considered undesirable. The results from section 5 however provide guidance on how to address this problem, without much impact on the current design standards. On the one hand, the problem can be addressed by a slight recalibration of partial factors on the loading side (see subsection 6.1), and on the other hand, the problem can be addressed by a smarter choice for the reference period (see sections 6.2 and 6.3). It should however be remarked that these suggestions do not deal with the fact that the (average) reliability levels will remain different for e.g. wind, snow and imposed variable loads. From an economic optimization point of view, a differentiation in target reliabilities seems to be inevitable, since the optimal target value will depend (among others) on the relative cost of safety measures (which will differ for the different variable loading types). ISO2394:2015 therefore provides target reliabilities as a function of the cost of safety measures. We recommend this approach for future code making.

6.1 *Adaptations of partial factors on the loading side*

A slight adaptation of the currently prescribed partial factors could result in a more uniform safety level over the different load ratios. A total recalibration of partial factors is out scope of this study, however, the reliability results found in the previous section provide valuable insight on this matter.

- *Partial factor for variable loads*
  For many combinations of material type, failure mode and variable loading type, the minimum reliability levels corresponded to design situations with variable loads only (see Figure 8, minima marked with a diamond). This gives the impression that the partial factor for variable loads is relatively low as compared to the partial factor for permanent loads. Moreover, the average reliability levels for imposed loads were systematically higher than those for wind loads, followed by snow loads. This gives the impression that the partial factor for wind and snow loads are relatively low as compared to the one for imposed loads. It is therefore suggested to increase the partial factor for variable loads as compared to permanent loads, while separating the partial factor for the three variable loading types. Thereby the partial factor for snow loads should be highest, followed by wind loads, followed by imposed loads.

- *Partial factor for permanent loads*
  For both steel and reinforced concrete, the design situations with self-weight as dominant permanent loading type lie systematically above those with imposed permanent loads as dominant permanent loading type. It is therefore proposed to separate the partial factor for permanent loads into a partial factor for self-weight and a partial factor for imposed permanent loads. In case of material types with a low variability (CoV < 0.05) the partial factor for self-weight can be taken lower than the partial factor for imposed permanent loads.

As an illustration, Table 4 shows a new safety format incorporating the changes mentioned above (values based on expert judgement). Figure 11 and Figure 12 show the resulting reliability levels for the steel member in bending and the concrete member in bending respectively. Indeed the new safety format results in a more uniform reliability level over the distinct load ratios, without a significant increase or decrease in the average safety level (see also Figure 2 and Figure 5). From that perspective, the proposed changes are an improvement compared to the current EN1990:2002.

It should be remarked that, in theory, also the characteristic values of the loads could be adapted to enforce a more uniform reliability level. Such adaptations are however not always desirable from a conceptual point of view. For example, characteristic values of climatic actions (wind speed, snow load) are typically chosen such that they correspond to a return period equal to the design lifetime of the structure. When changing the characteristic value to a higher (or lower) value, the return period will deviate from the design lifetime. Moreover, the adaptation of characteristic values would require the additional recalibration of the partial factors for serviceability limit states (SLS), which is undesirable as well. This option is therefore not further investigated.

Table 4: Investigated adapted safety-format

symbol	description	Eurocodes (see Table 2)	adaptation
$\gamma_{G,\mathrm{P}}$	Partial factor for imposed permanent actions	1.35	1.35
$\gamma_{G,\mathrm{sw,s}}$	Partial factor for self-weight (steel)	1.35	1.20
$\gamma_{G,\mathrm{sw,rc}}$	Partial factor for self-weight (reinf. concrete)	1.35	1.20
$\gamma_{G,\mathrm{sw},m}$	Partial factor for self-weight (material type *m*)	1.35	1.20
$\gamma_{Q,\mathrm{I}}$	Partial factor for variable actions (imposed)	1.50	1.50
$\gamma_{Q,\mathrm{W}}$	Partial factor for variable actions (wind)	1.50	1.60
$\gamma_{Q,\mathrm{S}}$	Partial factor for variable actions (snow)	1.50	1.90

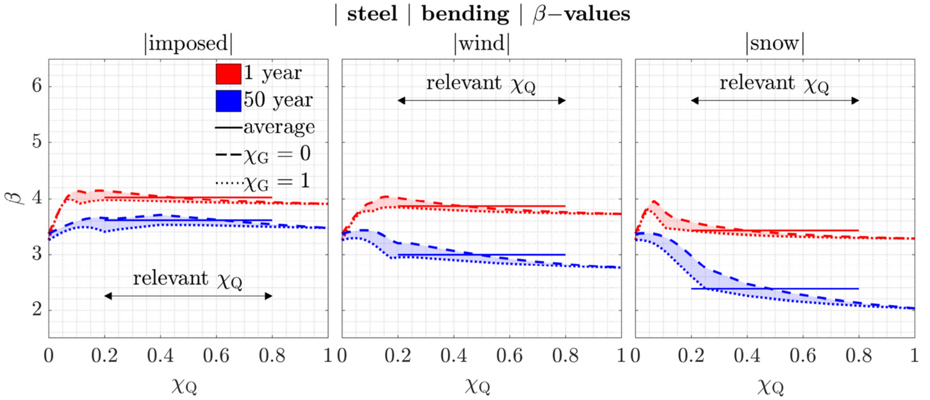

Figure 11: Calculated reliability levels for a steel member in bending, using the adapted partial factor format

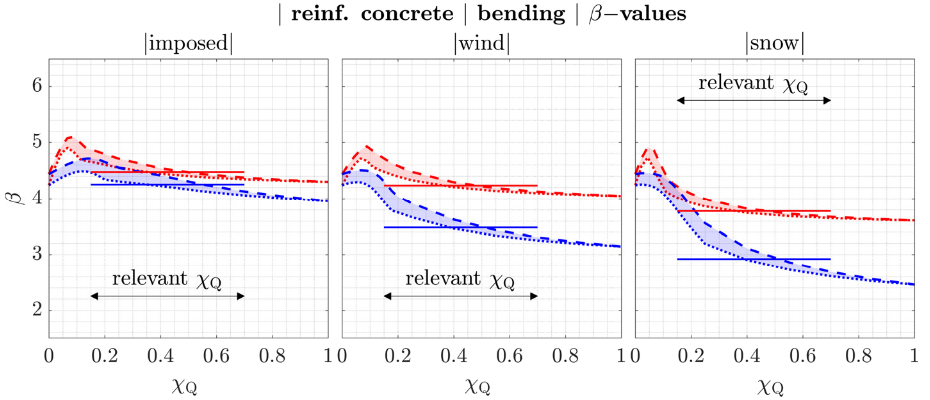

Figure 12: Calculated reliability levels for a reinforced concrete member in bending, using the adapted partial factor format

6.2 *Reference periods*

It is often debated which reference period is more suitable for the design and assessment of (existing) structures; the unit period of one year or the 'lifetime' period of fifty years. A comprehensive discussion on this topic is out scope of this study, yet, the results presented in section 5 provide valuable insights on this matter. Regardless of material type, failure mode or variable loading type, the scatter in reliability levels over the relevant load ratios was found to be significantly smaller for the one year reference period than for the fifty year reference period (see Figure 8). When a single target reliability needs to be chosen to 'summarize' the prevailing safety level, therefore, the one year reference period seems more appropriate. This choice would also immediately 'reduce' the apparent scatter in reliability levels over the different load ratios. In the same vein, the scatter in sensitivity factors over the relevant load ratios was found to be smaller for the one year reference period than for the fifty year reference period (see Figure 9 and Figure 10). In case a single 'standardized' sensitivity factor should be chosen, the application of the one year reference period would eventually result in less scatter in the obtained reliability levels.

6.3 *Influence of reference period on sensitivity factors* α

Many material dependent codes are calibrated on the basis of the fifty year reference period only. It is often feared that the change in reference period would therefore require an exhaustive recalibration of these codes. The results in this study showed that this fear is not needed, which is explained as follows. According to EN1990:2002 article C7 (3), the design value of the structural resistance R_{d} is defined such that the probability of having a more unfavourable value is equal to:

$$\mathrm{P}(R \le R_{\mathrm{d}}) = \Phi(-\alpha_R \beta_{\mathrm{t}}) \tag{16}$$

where $\Phi(.)$ is the standard normal distribution, α_R the standardized sensitivity factor for the structural resistance (equal to 0.8), and β_{t} is the target reliability (equal to 3.8 for the 50 year reference period). This means that the current design values of the structural resistance are calibrated such that they (approximately) correspond to $P(R \le R_{\mathrm{d}}) = \Phi(-\alpha_{R,50\mathrm{yr}}\, \beta_{\mathrm{t},50\mathrm{yr}}) = \Phi(-0.8 \cdot 3.8)$.

Figure 13 and Figure 14 show the values for $\alpha_{R,50\mathrm{yr}}\, \beta_{50\mathrm{yr}}$ and $\alpha_{R,1\mathrm{yr}}\, \beta_{1\mathrm{yr}}$ for the steel and reinforced concrete in bending (note that these figures can be obtained by the combination of the figures presented in section 5). The figures show that for the vast majority of design situations $\alpha_{R,1\mathrm{yr}}\, \beta_{1\mathrm{yr}} \le \alpha_{R,50\mathrm{yr}}\, \beta_{50\mathrm{yr}}$ and if not, the difference is small. A similar behaviour was observed for all other design situations in scope of this study (see appendix B). This means that the application of the design rule (16) with the currently obtained values for $\alpha_{R,1\mathrm{yr}} / \alpha_{R,50\mathrm{yr}}$ and $\beta_{1\mathrm{yr}} / \beta_{50\mathrm{yr}}$ would generally result in $R_{\mathrm{d},50\mathrm{yr}} \le R_{\mathrm{d},1\mathrm{yr}}$. In other words, a design according to the fifty year reference period generally leads to a more conservative design than a design according to the one year reference period. The proposed choice for the one year reference period is therefore demonstrated to require no recalibration of the material oriented building code parts.

Applying this reasoning to the current safety format (and assuming that $\beta_{t,1} = 4.2$), this would mean that the standardized sensitivity factor should become $\alpha_{R,1\mathrm{yr}} = 0.8 \cdot 3.8 / 4.2 \approx 0.7$.

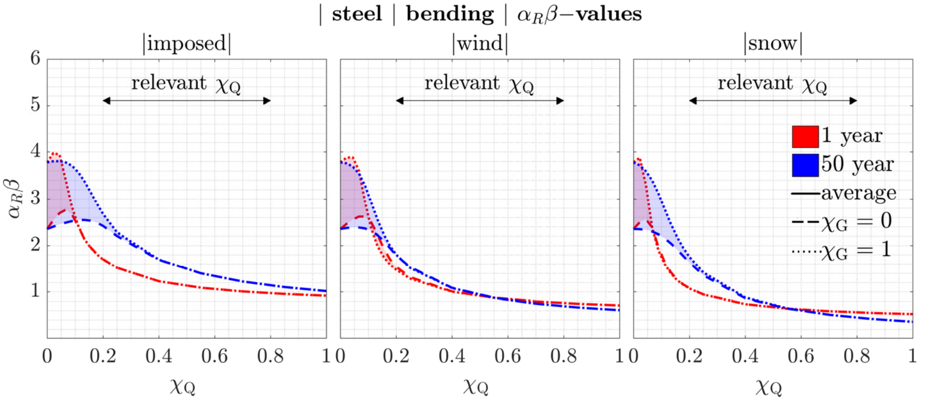

Figure 13: Steel; $\alpha_R\beta$ -values for the 1 year reference period (red) are for almost all relevant load ratios smaller than for the 50 year reference period (blue). If not, the difference is negligibly small

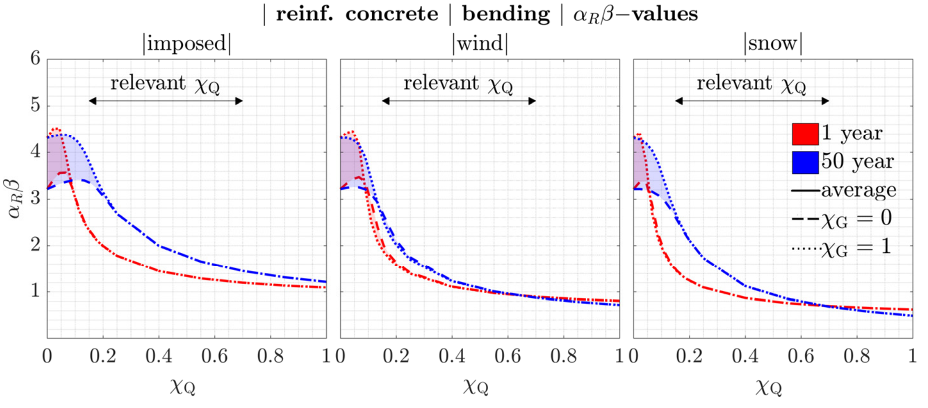

Figure 14: Concrete; aRβ-values for the 1 year reference period (red) are for all relevant load ratios smaller than for the 50 year reference period (blue)

7 Conclusions and recommendations

Main objective of this study was to provide insight in the reliability levels obtained by current Eurocode EN1990:2002 partial factor design (CC2, ULS), and to discuss these reliability levels in the light of current and future target reliabilities. To achieve this, a large number (> 3000) of typical design situations were designed using the EN1990:2002 partial factor format, and assessed on their structural reliability.

The results showed systematic differences between the reliability levels obtained for different material types, failure modes, variable loading types, and load ratios. For both steel and reinforced concrete, the reliability levels obtained for design situations governed by self-weight were found to be systematically higher than those obtained for design situations governed by imposed permanent loads. For both steel and reinforced concrete, the reliability levels obtained for design situations governed by variable loads were found to be systematically (and significantly) lower than those obtained for design situations governed by permanent loads. For wind and snow loads, many design situations resulted in reliability levels below the target values. In case of imposed loads, the average reliability levels were found to be close to the target values.

Based on the results, several options for future codification actions were discussed. This has led to the following recommendations:

- From a consistency point of view, it is not recommended to accept the current differences between the obtained reliability levels and the prescribed reliability levels, as this would lead to an unfair an undesirable competition between the semi-probabilistic and full-probabilistic design methods.
- It is recommended not to set a single (fixed) target reliability for all design situations in scope of the Eurocodes; both from the prevailing safety level point of view and from an economic optimization point of view.
- A more uniform reliability level over the different load ratios is needed and conceivable. Two possible actions are suggested, which are expected to lead to a more uniform reliability level, without a significant change in the average safety level:
    1. A slight adaptation of the current partial factor format is recommended, in which separate partial factors are assigned to the distinct permanent loading types (self-weight, imposed permanent loads) and the distinct variable loading types (imposed, wind, snow). For material types with low variability (CoV<0.05), the

partial factor for self-weight can be taken lower than for imposed permanent loads. The partial factors for variable loads are increased as compared to the current values, with the highest value for snow loads, followed by wind loads, followed by imposed loads.

2. A change from the fifty year reference period to the one year reference period is recommended, as this would lead to an immediate decrease in the scatter of reliability levels over the different load ratios.

It is was demonstrated that neither of these recommendations would require any change or recalibration of the material dependent Eurocodes.

8 Acknowledgements

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Appendix A Probabilistic framework

This appendix describes the probabilistic framework applied in this study. It presents the adopted physical models and the probabilistic descriptions of the random variables.
The physical models correspond to those recommended by the Eurocodes. The probabilistic descriptions of the random variables are obtained from recommendations in the literature, from expert knowledge, or from measurement data analysed by the authors.

A.1 ***Resistance models***

A.1.1 Structural steel

<u>Bending moment resistance</u>

The design value of the bending moment resistance of a steel member is obtained by:

$$R_{\mathrm{d}} = W_{\mathrm{el}} \frac{f_{\mathrm{y,k}}}{\gamma_{\mathrm{M,s}}} \tag{A.1}$$

where $f_{\mathrm{y,k}}$ is the characteristic value of the yield strength, $\gamma_{\mathrm{M,s}}$ the partial factor for structural steel (see Table 2), W_{el} the elastic section modulus of the steel profile (design parameter).

The corresponding probabilistic model is:

$$R = \theta_{R,\mathrm{s,bend}} W_{\mathrm{el,opt}} f_{\mathrm{y}} \tag{A.2}$$

where $\theta_{R,\mathrm{s,bend}}$ represents the model uncertainty of the resistance model for a steel member in bending (random variable), f_{y} the yield strength of the structural steel (random variable) and $W_{\mathrm{el,opt}}$ the optimal value of the elastic section modulus obtained by substitution of equation (A.1) in equation (7) and solving for W_{el}. For the probabilistic descriptions of the random variables it is referred to Table A.1.

<u>Axial compression resistance</u>

The design value of the axial compression of a steel member is obtained by:

$$R_{\mathrm{d}} = A \frac{f_{\mathrm{y,k}}}{\gamma_{\mathrm{M,s}}} \tag{A.3}$$

where $\gamma_{M,s}$ represents the partial factor for structural steel (see Table 2), $f_{y,k}$ the characteristic value of the yield strength and A the cross section of the steel profile (design parameter).

The corresponding probabilistic model is:

$$R = \theta_{R,s,comp} A_{opt} f_y \quad (A.4)$$

where $\theta_{R,s,comp}$ represents the model uncertainty of the resistance model for a steel member in axial compression (random variable), f_y represents the yield strength of the structural steel (random variable) and A_{opt} the optimal value of the cross-sectional area of the steel member obtained by substitution of equation (A.4) in equation (7) and solving for A. For the probabilistic descriptions of the random variables it is referred to Table A.1.

Table A.1: Probabilistic descriptions of (random) variables of resistance models for steel

X	description	dist.	mean	CoV	frac.	remark
W_{el}	Elastic section modulus of steel member in bending	DET	$W_{el,opt}$	-	-	design parameter
A	Cross-sectional area of steel member in axial compression	DET	A_{opt}	-	-	design parameter
f_y	Yield strength of structural steel	LOG	1.00	0.08	0.016 (1)	Cajot et. al. (2005)
$\theta_{R,s,bend}$	Resistance model uncertainty for steel members in bending	LOG	1.00	0.05	-	*Probabilistic Model Code* (JCSS, 2001)
$\theta_{R,s,comp}$	Resistance model uncertainty for steel members in axial compression	LOG	1.00	0.05	-	expert knowledge

(1) *Cajot et. al. (2005) specify the mean value of the yield strength by* $\mu(f_y) = f_{y,k} + k_s \sigma(f_y)$*, where* k_s *represents the quality control factor. In this study* $k_s = 2$ *is chosen, which corresponds to no regular quality control, and leads to a fractile value of 0.016.*

A.1.2 Reinforced concrete

Bending moment resistance

In case the concrete member is loaded in bending only (i.e. no axial loads) and has a ductile failure mechanisms (i.e. failure due to yielding of the reinforcement bars), the design value of the structural resistance is determined by:

$$R_{\mathrm{d}} = b\rho d \frac{f_{\mathrm{y,k}}}{\gamma_{\mathrm{M,rs}}} \left(d - 0.5\rho d \frac{f_{\mathrm{y,k}}}{\gamma_{\mathrm{M,rs}}} \frac{\gamma_{\mathrm{M,c}}}{f_{\mathrm{c,k}}} \right) \tag{A.5}$$

where b is the width of the reinforced concrete member, ρ is the reinforcement ratio for the bending moment, d is the effective depth of the reinforced concrete member, $f_{\mathrm{y,k}}$ is the characteristic value of the yield strength of the reinforcement steel, $f_{\mathrm{c,k}}$ is the characteristic value of the concrete compression strength, $\gamma_{\mathrm{M,c}}$ is the material factor for the concrete compressive strength and $\gamma_{\mathrm{M,rs}}$ is the material factor for the reinforcement steel (see Table 2).

In case the characteristic material parameters $f_{\mathrm{y,k}}$ and $f_{\mathrm{c,k}}$ are chosen on beforehand, the design of the structural member amounts the determination of appropriate values for b, d, and ρ (see section 3.3.2). This means that we have three design parameters rather than one. However, in the set-up of the calibration study we assume a single design parameter only. Moreover we assume that the structural resistance is proportional to that parameter. For this reason we choose the parameter b as the design parameter, while for the other parameters we adopt fixed (deterministic) values. Thereby the effective depth is fixed to $d = 5b$, and the reinforcement ratio is fixed to $\rho = 0.5\%$. The design parameter b_{opt} will be then obtained by substitution of (A.5) in (7) and solving for b.

The corresponding probabilistic model is:

$$R = \theta_{R,\mathrm{rc,bend}} b_{\mathrm{opt}} \rho d f_{\mathrm{y}} \left(d - 0.5\rho d \frac{f_{\mathrm{y}}}{f_{\mathrm{c}}} \right) \tag{A.6}$$

where $\theta_{R,\mathrm{rc,bend}}$ is the model uncertainty in the resistance model of reinforced concrete members in bending. For the probabilistic descriptions of the random variables it is referred to Table A.2.

Axial compression resistance

The design-value of a concrete member loaded by axial compression (not sensitive to buckling) is determined by:

$$R_{\mathrm{d}} = A\left(\frac{f_{\mathrm{c,k}}}{\gamma_{\mathrm{M,c}}} + \rho\frac{f_{\mathrm{y,rs,k}}}{\gamma_{\mathrm{M,rs}}}\right) \tag{A.7}$$

where A is the cross-sectional area of the concrete member, ρ is the reinforcement ratio for axial compression, $f_{\mathrm{c,k}}$ is the characteristic value of the concrete compressive strength, $f_{\mathrm{y,rs,k}}$ is the characteristic value of the yield strength of the reinforcement steel, $\gamma_{\mathrm{M,rs}}$ is the material factor for reinforcement steel and $\gamma_{\mathrm{M,c}}$ is the material factor for concrete.

In case the material parameters $f_{\mathrm{c,k}}$ and $f_{\mathrm{y,rs,k}}$ are chosen on beforehand, the design of the structural member amounts the determination of appropriate values for A and ρ, this means that we have two design parameters rather than one. However, in the set-up of the calibration study we assume a single design parameter only. Moreover we assume that the structural resistance is proportional to the design parameter. For this reason we choose A as the design parameter, while for ρ we adopt a fixed, deterministic value of $\rho = 0.5\%$. The design parameter A_{opt} is obtained by substitution of (A.7) in (7) and solving for A .

The corresponding probabilistic model for the resistance for axial compression of a concrete member is given by:

$$R = \theta_{R,\mathrm{rc,comp}} \mathrm{A}_{\mathrm{opt}}\left(f_{\mathrm{c}} + \rho f_{\mathrm{y,rs}}\right) \tag{A.8}$$

where $\theta_{R,\mathrm{rc,comp}}$ is the model uncertainty for the resistance model of reinforced concrete in axial compression (random variable), f_{c} the concrete compression strength (random variable), $f_{\mathrm{y,rs}}$ the yield strength of the reinforcement steel (random variable). The probabilistic models of the parameters are provided in Table A.2.

Table A.2: Probabilistic descriptions of (random) variables in resistance models for reinforced concrete

X	description	dist.	mean	CoV	frac.	remark
b	Width of concrete member in bending	DET	b_{opt}	-	-	design parameter
d	Effective depth of concrete member in bending	DET	$5\, b_{\mathrm{opt}}$	-	-	fixed value based on expert knowledge
ρ	Reinforcement ratio in bending / axial compression	DET	0.005	-	-	fixed value based on expert knowledge
f_{c}	Concrete compressive strength	LOG	1.00	0.15	0.05	expert knowledge
$f_{\mathrm{y,rs}}$	Yield strength of reinforcement steel	LOG	1.00	0.07	0.05	expert knowledge
$\theta_{R,\mathrm{rc,bend}}$	Resistance model uncertainty for reinforced concrete members in bending	LOG	1.075	0.075	-	Sýkora et. al. (2015)
$\theta_{R,\mathrm{rc,comp}}$	Resistance model uncertainty for reinforced concrete members in axial compression	LOG	1.00	0.05	-	Sýkora et. al. (2015)

A.1.3 Material type m

The design value of the bending moment resistance of structural material type m is obtained by:

$$R_{\mathrm{d}} = W_{\mathrm{el}} \frac{f_{\mathrm{u,k}}}{\gamma_{\mathrm{M},m}} \tag{A.9}$$

where $f_{\mathrm{u,k}}$ is the characteristic value of the ultimate strength, $\gamma_{\mathrm{M},m}$ is the material factor for structural material m (see Table 2) and W_{el} the elastic section modulus of the member.

The corresponding probabilistic model is:

$$R = \theta_{R,m,\text{bend}} W_{\text{el,opt}} f_{\text{u}} \tag{A.10}$$

where $\theta_{R,m,\text{bend}}$ is the model uncertainty for the structural resistance of material type *m* (random variable), f_{u} is the ultimate strength of material type *m* (random variable) and $W_{\text{el,opt}}$ is the elastic section modulus of the member (design parameter), obtained by substitution of (A.9) in (7) and solving for W_{el}. The probabilistic descriptions of the (random) variables are provided in Table A.3.

Table A.3: Probabilistic descriptions of (random) variables in resistance model for material type m

X	description	dist.	Mean	CoV	frac.	remark
W_{el}	Elastic section modulus material type *m*	DET	$W_{\text{el,opt}}$	-	-	design parameter
f_{u}	Bending strength material type *m*	LOG	1.00	0.25	0.05	expert knowledge (1)
$\theta_{R,m,\text{bend}}$	Resistance model uncertainty for members of material type *m* in bending	LOG	1.00	0.08	-	expert knowledge (2)

(1) *Value corresponds to (glued laminated) timber in the Probabilistic Model Code (JCSS, 2001).*

(2) *No references were found. Value is taken slightly larger than for reinforced concrete in bending.*

A.2 ***Loads and load effects***

A.2.1 Permanent loads

Both the self-weight (G_{sw}) and the imposed permanent loads (G_{P}) are modelled by a single random variable. For the probabilistic descriptions see Table A.4.

Table A.4: Probabilistic descriptions of random variables describing the permanent loads

symbol	description	dist.	Mean	CoV	fractile	remark
$G_{\mathrm{sw,s}}$	Self-weight structural steel	NOR	1.00	0.02 [(1)]	0.50	expert knowledge
$G_{\mathrm{sw,rc}}$	Self-weight reinforced concrete	NOR	1.00	0.05 [(2)]	0.50	expert knowledge
$G_{\mathrm{sw},m}$	Self-weight material type *m*	NOR	1.00	0.10	0.50	expert knowledge
G_{P}	Induced permanent load	NOR	1.00	0.10	0.50	expert knowledge

[(1)] *The Probabilistic Model Code (JCSS, 2001) recommends the value CoV < 0.01 for structural steel. To account for the (small) deviations in the structural dimensions we slightly increase this recommended value.*

[(2)] *The Probabilistic Model Code (JCSS, 2001) recommends the value CoV = 0.04 for ordinary concrete without reinforcement and with stable moisture content. To account for the (small) deviations in the dimensions we slightly increase this recommended value.*

A.2.2 Imposed loads

The design value of the imposed loads is obtained by:

$$Q_{\mathrm{I,d}} = q_{\mathrm{equ,k}}\, A\, \gamma_Q \tag{A.11}$$

Where $q_{\mathrm{equ,k}}$ is the characteristic value of the time-dependent part of the equivalent uniform imposed load, A the geometrical properties of the element and γ_Q the partial factor for variable loads (see Table 2).

The corresponding probabilistic model is:

$$Q_{\mathrm{I}} = \theta_{Q,\mathrm{I}}\, q_{\mathrm{equ}}\, A \tag{A.12}$$

where $\theta_{Q,\mathrm{I}}$ is the model uncertainty for the action model for imposed loads (random variable), q_{equ} is the time-dependent part of the equivalent uniform imposed load (random variable) and A represent the geometrical properties of the element (deterministic value). The probabilistic descriptions of the (random) variables are provided in Table A.5.

Table A.5: Probabilistic descriptions of the random variables in the imposed load model

symbol	description	dist.	Mean	CoV	frac.	remark
$q_{\mathrm{equ,5yr}}$	Time-dependent part of the imposed loads for a reference time of 5 years	GUM	1.00	0.53	0.995	*Probabilistic Model Code* (JCSS, 2001) (1)
$q_{\mathrm{equ,50yr}}$	Time-dependent part of the imposed loads for a reference time of 50 years	GUM	1.00	0.22	0.953	extrapolation and normalization $q_{\mathrm{equ,5yr}}$ (2)
$\theta_{Q,\mathrm{I}}$	Time-independent model uncertainty factor for imposed loads	LOG	1.00	0.10	-	expert knowledge

(1) *Based on the proposed random field model in the Probabilistic Model Code (JCSS,2001), Baravalle et. al. (2017) sampled a time-history of the equivalent uniform imposed load for office loads (Cat B) and fitted a Gumbel distribution on the upper tail. The fitted distribution resulted in a mean value of 839 N/m² and a CoV = 0.53. The characteristic value of 2.5kN/m² for office loads (EN1991-1-1:2002, Cat B) then corresponds to the characteristic fractile of 0.995.*

(2) *Obtained by extrapolation and normalization of the distribution of the 5-yearly extremes to the fifty yearly extremes assuming independent and identically distributed random variables. The characteristic value is taken equal as for the 5-year reference period.*

A.2.3 Wind loads

The design value of the wind loads on the structure is obtained by EN1991-1-4:2005:

$$Q_{\mathrm{W,d}} = q_{\mathrm{ref,k}}\, c_{\mathrm{e,k}}\, c_{\mathrm{p,k}}\, c_{\mathrm{s,k}}\, c_{\mathrm{d,k}}\, A\, \gamma_Q \tag{A.13}$$

Where $q_{\mathrm{ref,k}}$ is the characteristic value of the N-yearly extreme hourly-mean wind velocity pressure at reference height and reference terrain roughness, $c_{\mathrm{e,k}}$ is the characteristic value of the exposure factor, $c_{\mathrm{p,k}}$ is the characteristic value of the hourly extreme pressure coefficient, $c_{\mathrm{s,k}}$ is the characteristic value of the size factor, $c_{\mathrm{d,k}}$ the characteristic value of the dynamic factor, A the dimensions of the loaded element and γ_Q the partial factor for variable loads defined by EN1990:2002 (see Table 2).

The corresponding probabilistic model is:

$$Q_W = \theta_{Q,W}\, q_{ref}\, c_e\, c_{pe}\, c_s\, c_d\, A \tag{0.26}$$

Where $\theta_{Q,W}$ is the model uncertainty for the physical wind load model (random variable), q_{ref} the *N*-yearly extreme hourly-mean wind velocity pressure at reference height and reference terrain roughness (random variable), c_e the exposure factor (random variable), c_p the hourly extreme pressure coefficient (random variable), c_s is the size factor (random variable), c_d the dynamic factor (random variable), A the dimensions of the loaded element (deterministic). The probabilistic descriptions of the (random) variables are provided in Table A.6.

Table A.6: Probabilistic description of (random) variables in the wind load model

X	description	dist.	Mean	CoV	frac.	remark
$q_{ref,1}$	Yearly extreme hourly mean wind velocity pressure	GUM	1.00	0.27	0.98	expert knowledge (1)
$q_{ref,50}$	50-yearly extreme hourly mean wind velocity pressure	GUM	1.00	0.15	0.37	extrapolation and normalization $q_{ref,1}$ (2)
c_e	Exposure factor correcting for height and terrain roughness	LOG	1.00	0.15 (3)	0.94 (4)	*Probabilistic Model Code* (JCSS,2001)
c_p	Hourly extreme pressure coefficient	GUM	1.00	0.20 (5)	0.78 (6)	
c_s	Size factor	DET	1.00	-	-	
c_d	Dynamic amplification factor	LOG	1.00	0.15	0.50	*Probabilistic Model Code* (JCSS, 2001)
$\theta_{Q,W}$	Time-independent model uncertainty factor for wind loads	LOG	1.00	0.10	-	expert knowledge

(1) *Should in principle be based on meteorological data at site. Meinen et. al. (2015) analysed 64 years of meteorological data at Schiphol Airport, the Netherlands and found a CoV=0.27. This value is assumed to be typical for the European wind climate. Its remarked that EN1991-1-4:2005 assumes a slightly lower value, being CoV = 0.23 (value can be derived by taking the values of K=0.2 and n=0.5, see EN1991-1-4:2005 p. 19).*

[2] *Obtained by extrapolation and normalization of the distribution of the yearly extremes to the fifty yearly extremes assuming independent and identically distributed random variables. The characteristic value is taken equal as for the one year reference period.*

[3] *The Probabilistic Model Code (JCSS, 2001) recommends lognormal distribution with a CoV between 0.1-0.2. In this study we adopt the in-between value of CoV=0.15.*

[4] *The Probabilistic Model Code (JCSS, 2001) recommends a mean-to-characteristic value of 0.8. Provided a lognormal distribution with a CoV=0.15 it can be derived that the characteristic fractile lies at 0.94.*

[5] *Based on a large number of wind-tunnel investigations conducted at the TNO boundary layer wind tunnel, (Meinen et. al., 2015) found CoVs between 0.1 and 0.4. In this study we adopt the in-between value of 0.2.*

[6] *Geurts et. al. (2001) suggest that the values for the peak external pressure coefficients (cpe,10) such as presented in ENV1991-2-4:1995 were (indirectly) obtained using the analysis technique of Cook and Mayne*
(1980), which entails an (envelope) characteristic fractile value of 0.78 (see also Kasperski, 2003). In the generation of EN1991-1-4, many of the these values were reaccepted and are therefore also assumed to correspond to the (envelope) characteristic fractile of 0.78.

A.2.4 Snow loads

The design value of the snow loads is obtained by EN1991-1-3:2003:

$$Q_{S,d} = s_{k,k}\, C_{e,k}\, C_{t,k}\, \mu_{i,k}\, A\, \gamma_Q \tag{A.14}$$

Where $s_{k,k}$ is the characteristic value of the *N*-yearly extreme snow load on ground, $C_{e,k}$ is the characteristic value of the exposure coefficient, $C_{t,k}$ is the characteristic value of the heat coefficient, μ_i is the characteristic value of the load shape coefficient, A the geometrical properties of the loaded element and γ_Q the partial factor for variable loads defined by EN1990:2002 (see Table 2).

The corresponding probabilistic model is:

$$Q_S = \theta_{Q,S}\, s_k\, C_e\, C_t\, \mu_i\, A \tag{A.15}$$

Where $\theta_{Q,S}$ is the model uncertainty factor for the snow load model (random variable), s_k is the *N*-yearly extreme snow load on ground (random variable), C_e the exposure coefficient (random variable), C_t the heat coefficient (random variable), μ_i the load shape coefficient (random variable), A the geometrical properties of the loaded element (deterministic value). The probabilistic models of the parameters are provided in Table A.7.

Table A.7: Probabilistic descriptions of (random) variables in the snow load model

Symbol	description	dist.	Mean	CoV	frac.	Remark
$s_{k,1yr}$	Yearly extreme snow load at ground	GUM	1.00	0.60 (1)	0.98	expert knowledge
$s_{k,50yr}$	50-yearly extreme snow load at ground	GUM	1.00	0.21	0.37	extrapolation and normalization $s_{k,1vr}$ (2)
$\mu_i C_e$	Load shape coefficient for a uniform snow load covering a whole roof area multiplied by the exposure coefficient	NOR	1.00	0.30 (3)	0.50	Cajot et. al. (2005)
C_t	Heat coefficient	DET	1.00	-	-	Cajot et. al. (2005)
$\theta_{Q,S}$	Time-independent model uncertainty factor for snow loads	LOG	1.00	0.10	-	expert knowledge

(1) *Should in principle be based on meteorological data at site. fib (2016) provides indicative values for the Czech republic being CoV = 0.6 – 0.7 (lowlands) and 0.4 – 0.6 (mountains). In this study we adopt the (lower) value of 0.60.*

(2) *Obtained by extrapolation and normalization of the distribution of the yearly extremes to the fifty yearly extremes assuming independent and identically distributed random variables. The characteristic value is taken equal as for the one year reference period.*

(3) *Sanpaolesi et. al. (1999) investigated the influence of different roof types, wind exposure and geographical locations on the load shape coefficient (μ_i). They found coefficients of variations between CoV ≈ 0.1 (flat roofs, sheltered) and CoV ≈ 1.2 (gables roofs, wind swept).*

A.2.5 Load effects

The uncertainties associated with the calculation of the load effects are specified for the distinct failure modes. It is assumed that the load effect uncertainties are independent of the structural material.

Table A.8: Model uncertainties load effects

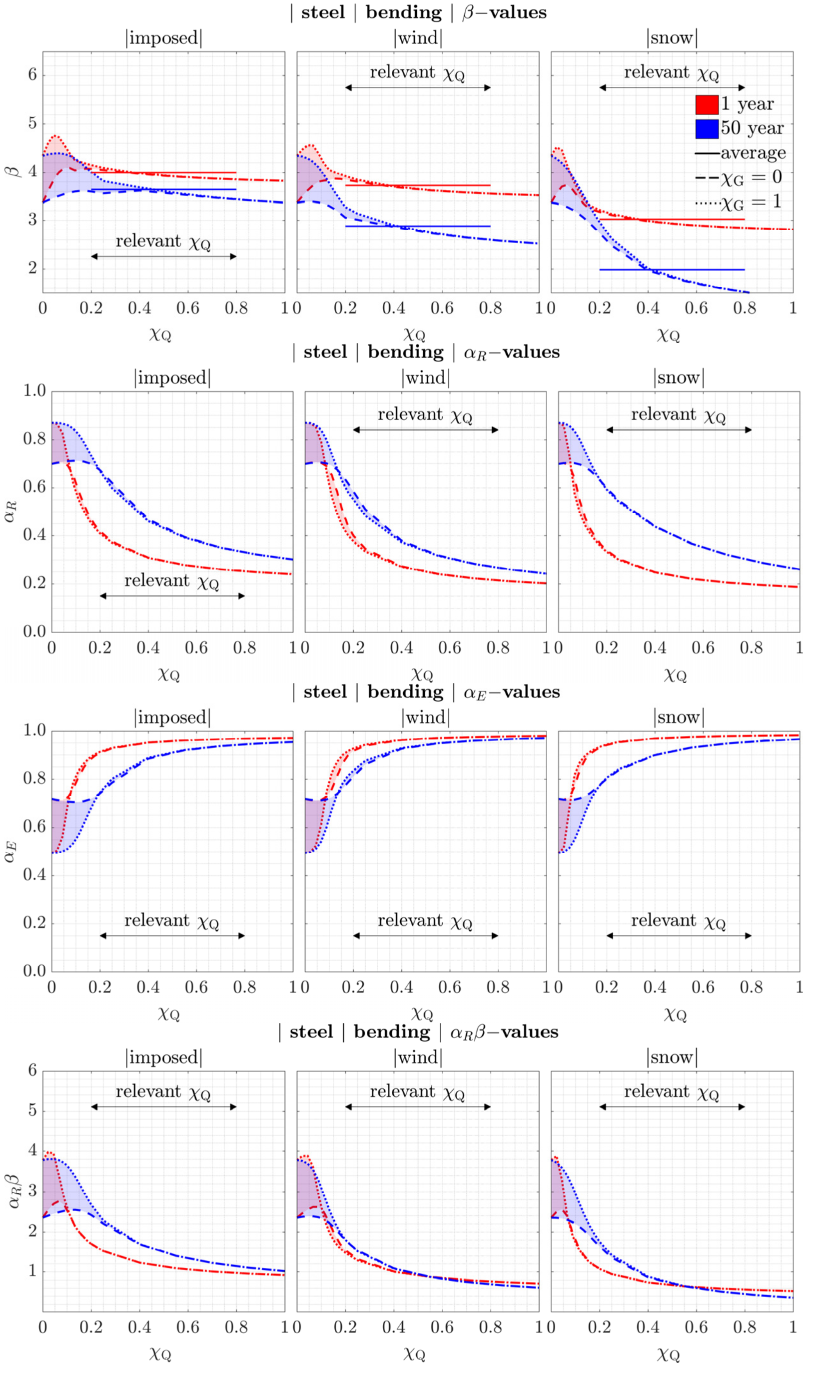
symbol	description	dist.	mean	CoV	frac.	remark
$\theta_{E,\text{bend}}$	Model uncertainty for load effects in bending	LOG	1.00	0.05 (1)	-	expert knowledge
$\theta_{E,\text{comp}}$	Model uncertainty factor for load effects in axial compression	LOG	1.00	0.05	-	*Probabilistic Model Code* (JCSS, 2001)

(1) *The Probabilistic Model Code (JCSS, 2001) recommends CoV = 0.1 for moments in frames.*

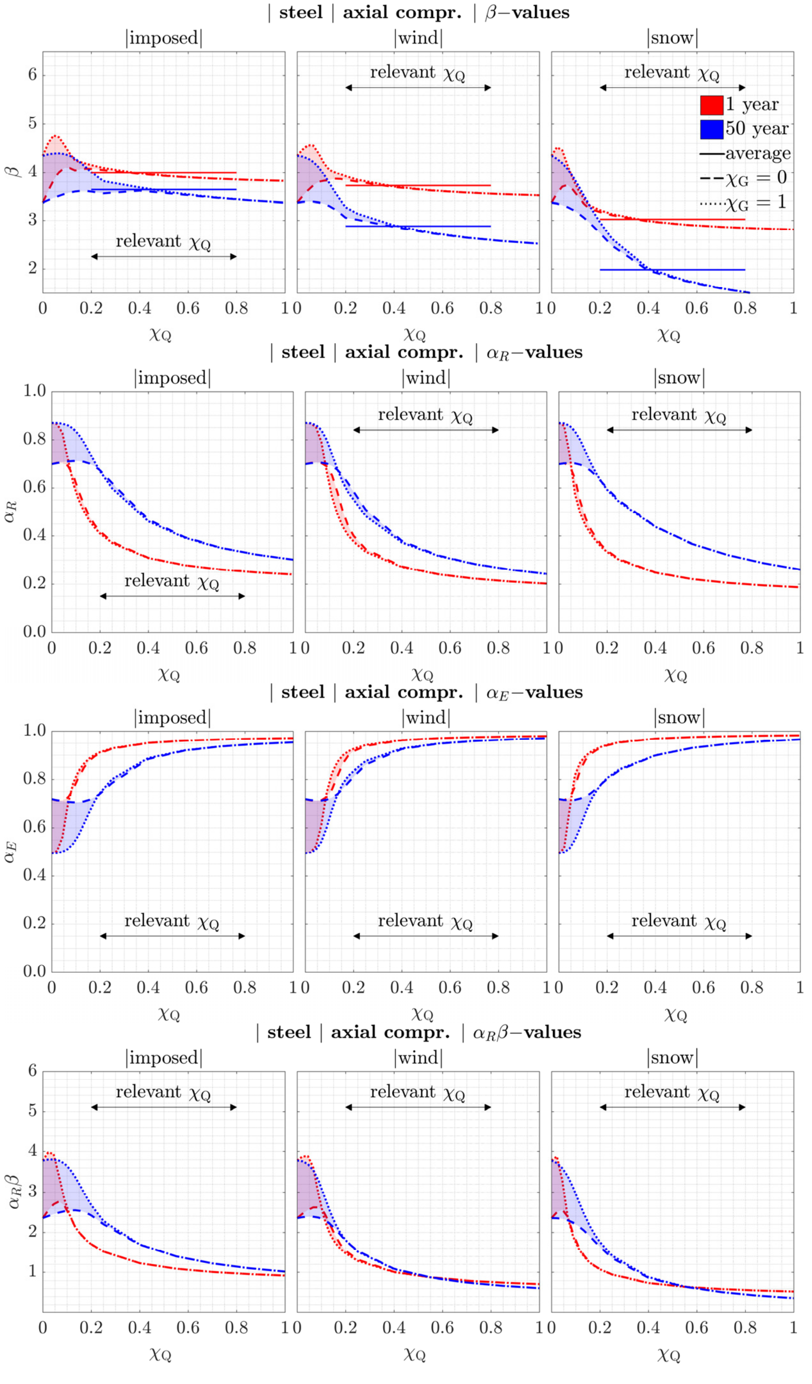
Appendix B Graphical overview of results

This appendix gives an overview of all computation results.

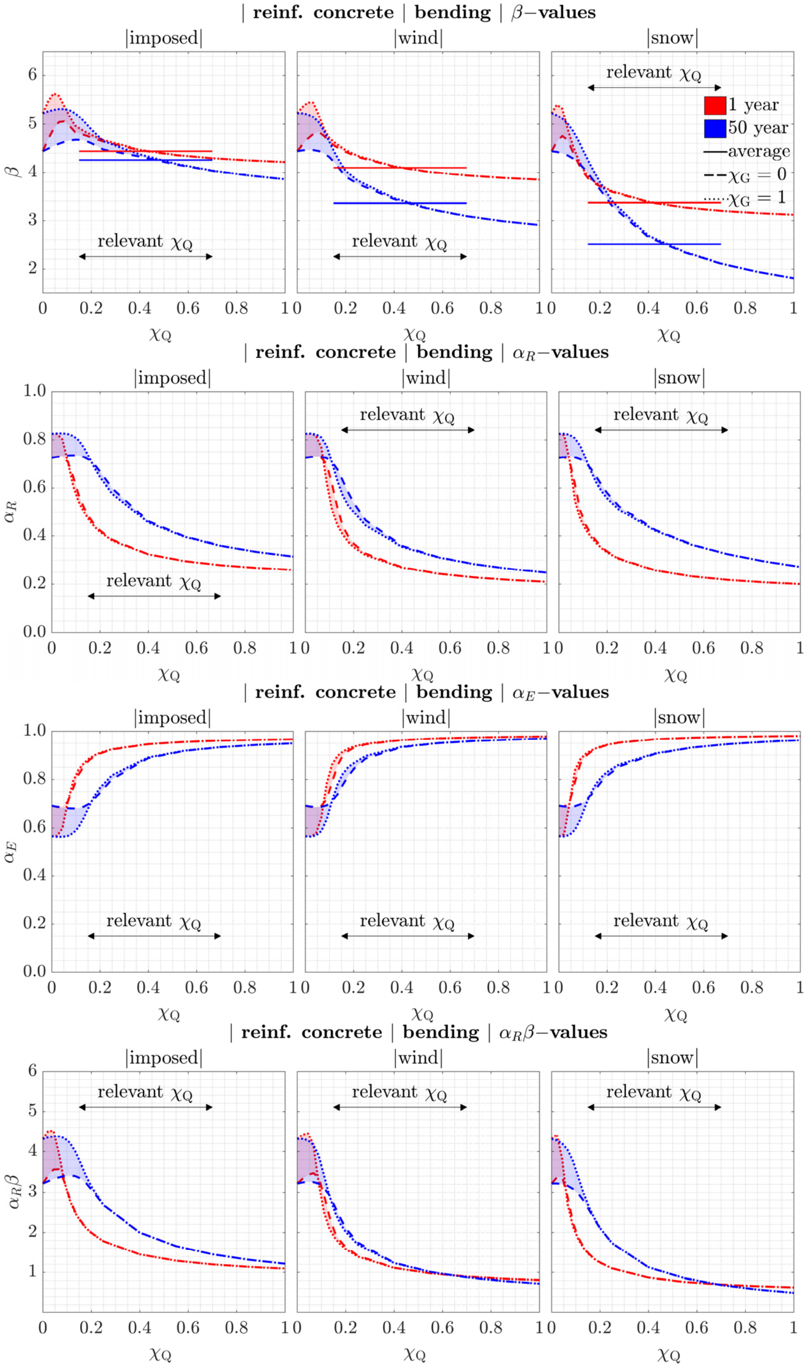
B.1 Steel in bending

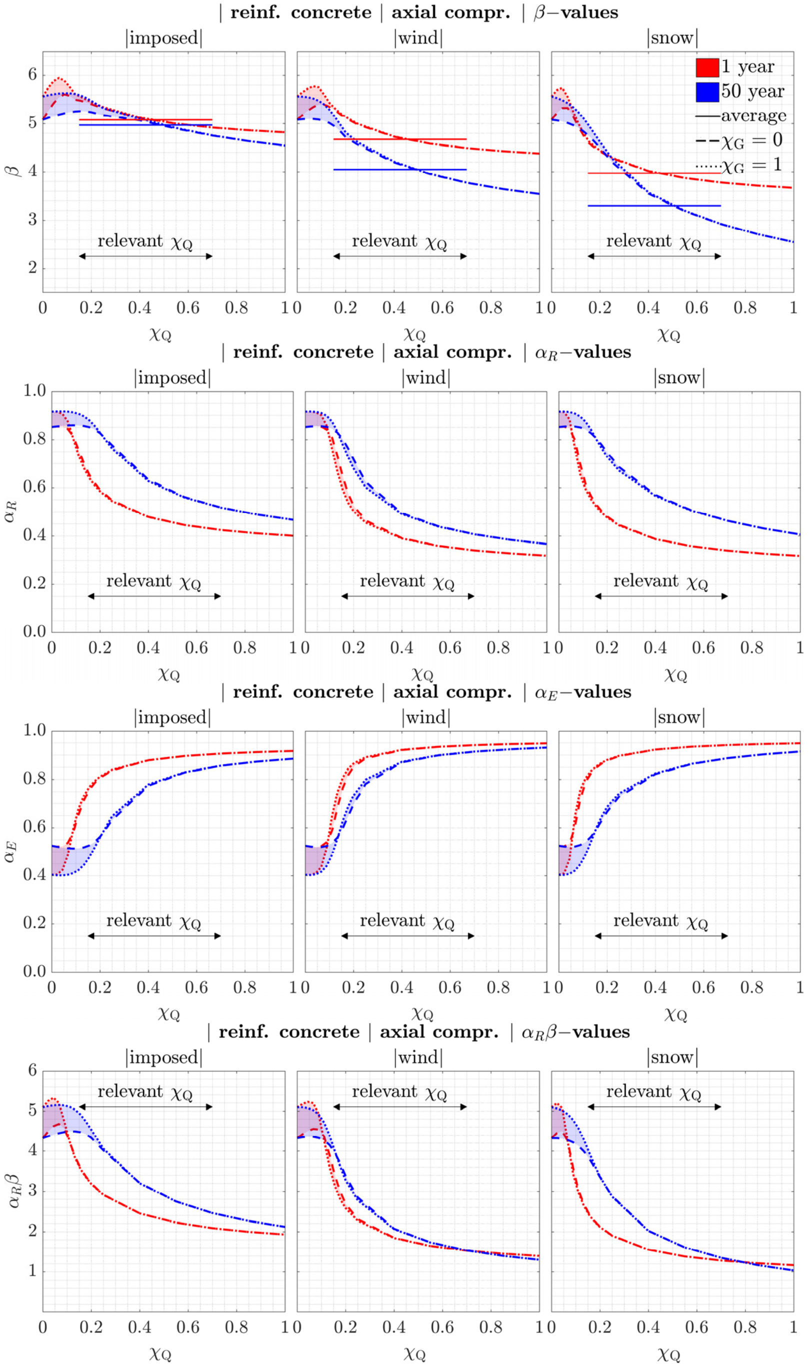


B.2 Steel in axial compression



B.3 Reinforced concrete in bending




| reinf. concrete | axial compr. | β−values
|imposed|
|wind|
|snow|
1 year
50 year
average
$\chi_G = 0$
$\chi_G = 1$
relevant χ_Q
β
χ_Q
| reinf. concrete | axial compr. | α_R−values
α_R
| reinf. concrete | axial compr. | α_E−values
α_E
| reinf. concrete | axial compr. | $\alpha_R\beta$−values
$\alpha_R\beta$

B.5 Material type m in bending