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Reliability Level of the Second Generation of the Eurocodes – Status and Potential

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ABSTRACT: The EN Eurocodes are a collection of 10 European Standards, EN 1990 - EN 1999 (CEN, 2002), providing a common basis for the design of load bearing structures. The next generation of the Eurocodes, the second generation, will be published by 2026 and the corresponding revision process is about to be concluded. The Eurocodes prescribe a semi-probabilistic partial factor safety concept with the intention that design solutions that follow this concept comply with the prescribed reliability criterion. In the present contribution this intended compliance is assessed in particular for the 3 alternative specifications of the safety concept that has to be selected by the Eurocode member states. In order to do so, a representative domain of design equations is formulated that captures load bearing structures with 9 different combinations of structural failure modes and materials, as well as a wide range of permanent and time-variable loads. The necessary probabilistic representation is adapted from the recently issued Reliability Background of the Eurocodes. An example calibration of the partial load factors is performed and results and corresponding implications are discussed. An outlook on future required research is given.

1. INTRODUCTION

In FprEN 1990:2022 (E) three different equations for the combination of actions for persistent and transient (fundamental) design situations are suggested. There, these equations are referred to as 8.12, 8.13 and 8.14. National standardisation committees have to prioritise one of these equations and in order to do so, more insights on the structural reliability that is implied by these three options is considered informative.

The design situations considered in this paper comprise one permanent load and one variable load and for such situations the three different equations can be reformulated as design equations where a design solution p_{ij} (for material i and variable load j) is identified as:

According (8.12):

$$p_{ij} = \frac{\gamma_{M,i}}{\theta_{Ri,k} r_{i,k}} \left\{ (1-a_Q) \left[a_G \gamma_{GS} g_{S,k} + (1-a_G) \gamma_{GP} g_{P,k} \right] + a_Q \theta_{Qj,k} \gamma_Q q_{j,k} \right\}$$

According (8.13):

$$p_{ij} = max \left\{ \begin{array}{c} \frac{\gamma_{M,i}}{\theta_{Ri,k} r_{i,k}} \left\{ (1-a_Q) \left[a_G \gamma_{GS} g_{S,k} + (1-a_G) \gamma_{GP} g_{P,k} \right] + a_Q \theta_{Qj,k} \psi_{0,j} \gamma_Q q_{j,k} \right\} \\ \frac{\gamma_{M,i}}{\theta_{Ri,k} r_{i,k}} \left\{ (1-a_Q) \xi \left[a_G \gamma_{GS} g_{S,k} + (1-a_G) \gamma_{GP} g_{P,k} \right] + a_Q \theta_{Qj,k} \gamma_Q q_{j,k} \right\} \end{array} \right\}$$

According (8.14):

$$p_{ij} = max \left\{ \begin{array}{c} \frac{\gamma_{M,i}}{\theta_{Ri,k} r_{i,k}} (1-a_Q) \left[a_G \gamma_{GS} g_{S,k} + (1-a_G) \gamma_{GP} g_{P,k} \right] \\ \frac{\gamma_{M,i}}{\theta_{Ri,k} r_{i,k}} \left\{ (1-a_Q) \xi \left[a_G \gamma_{GS} g_{S,k} + (1-a_G) \gamma_{GP} g_{P,k} \right] + a_Q \theta_{Qj,k} \gamma_Q q_{j,k} \right\} \end{array} \right\}$$

Notes:

- for illustration, in equation (8.12) above three different parts are indicated: resistance (red), permanent load (blue) and variable load (green). Equations (8.13) and (8.14) have a comparable structure.
- the marginal difference to (8.12) in terms of the factors $\psi_{0,j}$ and ξ underlined in orange.
- the second sub-equations in (8.13) and (8.14) are identical. The first sub-equation in (8.14) neglects variable loads.

Decoding the nomenclature in (8.12) – (8.14):

p_{ij} is the design solution for material i and variable load j (e.g. for a tension rod, this would be the cross-section area),

$\gamma_{M,i}$ is the material partial factor for material i,

$\theta_{Ri,k}$ is the characteristic value of the resistance model uncertainty of material i (often equal to 1, bias or hidden safety might be introduced here),

$r_{i,k}$ is the characteristic value of the strength of material i,

a_Q is a value between 0 and 1 that indicates the importance of variable loads. (0: no variable load, 1: only variable load and no permanent load),

a_G is a value between 0 and 1 that indicates the importance of self weight. (0: no self weight, 1: only self weight and no permanent installations),

$g_{S,k}$ is the characteristic value of the structural self weight,

γ_{GS} is the partial factor for the structural self weight,

$g_{P,k}$ is the characteristic value of the permanent load from elements other than structural,

γ_{GP} is the partial factor for the permanent load from elements other than structural,

γ_Q is the partial factor for the variable load,

$\theta_{Qj,k}$ is the characteristic value of the variable Q_j load model uncertainty,

$q_{j,k}$ is the characteristic value of the variable load j,

$\psi_{0,j}$ is the load combination factor for variable load j,

ξ is a reduction factor for permanent actions.

Limit state function:

For all three different ways to represent the design equations, one corresponding limit state function can be identified:

$$g(\mathbf{X}, p_{ij}) = p_{ij}\Theta_{R,i}R_i - \Theta_S((1-a_Q)(a_G \cdot G_{S,i} + (1-a_G)G_P) - a_Q\Theta_{Q,j}Q_j) \tag{1}$$

With:

p_{ij} is the design solution for material i and variable load j as identified by equation (8.12), (8.13) or (8.14),

$\Theta_{R,i}$ is the resistance model uncertainty of material i (represented as random variable),

R_i is the strength of material i (represented as random variable),

G_P is the permanent load from elements other than structural (represented as random variable),

G_S is the permanent load from self weight of the structural component (represented as random variable),

$\Theta_{Q,j}$ is the variable load j model uncertainty (represented as random variable),

Q_j is the variable load j (represented as random variable).

2. Approach

In order to benchmark the effect of the use of equations (8.12), (8.13) or (8.14) on the reliability level:

1. the design solutions, $p_{i,j}$, after equations (8.12), (8.13) or (8.14) are computed,
2. the corresponding reliability index is computed according to the limit state equation (1).

A domain D of 810 relevant design equations is defined representing the main structural materials, failure modes and loads induced by wind and snow, sustained and intermittent imposed loads as well as permanent actions. The importance of the different design equations is accounted for by weighting factors w_k. The assessment is conditional to many assumptions. For those it is referred to the results of CEN/TC 250/SC 10 Ad-Hoc Group on the Reliability Background in the Eurocodes (AHG).

The weighted average reliability level of the Eurocodes is computed as:

$$\beta_{w-average}(\gamma_{EC}) = \sum_{k \in D} w_k \beta_k(\gamma_{EC}) \tag{2}$$

3. Input / Assumptions

On the resistance side, 9 different failure modes have been chosen to represent the domain of failure modes and materials for which the equations (8.12), (8.13) or (8.14) apply. In Table 1 the representation of model uncertainties $\Theta_{R,i}$ is specified.

Table 1: Model Uncertainties of load bearing capacities, $\Theta_{R,i}$.

Variable	Distribution	Mean	C.o.V.
Steel bend.	lognormal	1.150	0.050
Concrete comp.	lognormal	0.970	0.140
Concrete bend.	lognormal	1.030	0.070
Glulam bend.	lognormal	1.000	0.100
Timber bend.	lognormal	1.000	0.100
Masonry	lognormal	1.160	0.180
Aluminium bend.	lognormal	1.270	0.150
Pile found.	lognormal	1.000	0.200
Shallow found.	lognormal	1.000	0.150

Comment: Mean values different from 1.00 indicate a model bias. I.e. a mean value of 1.15 means that the real capacity is expected to be 15% larger in average than predicted by the model.

The partial safety factors for resistance, $\gamma_{M,i}$ are given in Table 2.

Table 2: Partial factors for resistance.

Variable	$\gamma_{M,i}$
Steel yield strength	1.000
Concrete compression strength	1.500
Rebar yield strength	1.150
Glulam bending strength	1.250
Timber strength	1.300
Masonry	1.500
Aluminum	1.100
Soil	1.400
CPT	1.500

Comment: The values are taken from the existing Eurocodes and are fixed throughout the analysis.

Material resistances are represented as normalised random variables, which means that the mean values are scaled down to unity, see Table 3. The absolute magnitude of the different material resistances is not relevant here.

Table 3: Material Resistances, R_i.

Variable	Distribution	Mean	C.o.V.	char.value	fractile
Steel yield strength	lognormal	1.000	0.050	0.830	-
Concrete compression strength	lognormal	1.000	0.100	0.844	0.050
Rebar yield strength	lognormal	1.000	0.045	0.928	0.050
Glulam bending strength	lognormal	1.000	0.150	0.774	0.050
Timber strength	lognormal	1.000	0.200	0.708	0.050
Masonry	lognormal	1.000	0.160	0.760	0.050
Aluminum	lognormal	1.000	0.050	0.850	-
Soil	lognormal	1.000	0.150	0.774	0.050
CPT	lognormal	1.000	0.120	0.816	0.050

Comment: The characteristic values for steel and aluminium are not defined by specified fractile values, but as nominal values according EN1993-1-1, Annex E (CEN, 2005). The assigned characteristic value of steel corresponds to the 0.01% -fractile, the one for aluminium to the 0.06% -fractile (compared to the 5% -fractile for all other material resistances).

Table 4: Load Variables

Variable	Distribution	Mean	C.o.V.	char.value	fractile
Load Effect MU Frames	lognormal	1.000	0.100	1.000	-
Self weight steel	normal	1.000	0.025	1.000	-
Self weight concrete	normal	1.000	0.050	0.980	-
Self weight glulam	normal	1.000	0.100	0.950	-
Self weight timber	normal	1.000	0.100	0.950	-
Self weight masonry	normal	1.000	0.070	1.000	0.500
Self weight aluminum	normal	1.000	0.040	1.000	0.500
Self weight soil	normal	1.000	0.050	1.000	0.500
Permanent load small V	normal	1.000	0.100	1.000	0.500
Permanent load large V	normal	1.000	0.200	1.329	0.950
Wind MU	lognormal	0.970	0.260	-	-
Wind 50a-extreme	gumbel	1.000	0.140	1.084	0.980
Snow MU	lognormal	0.810	0.260	-	-
Snow 50a-extreme	gumbel	1.000	0.200	0.821	0.980
Imposed MU	lognormal	1.000	0.100	-	-
Imposed 50a-extreme	gumbel	1.000	0.260	1.350	0.990

Comment: The characteristic values of the variable actions are determined such that it corresponds to the specified fractile of the product of the yearly extreme value distribution, Q_i^{1a} and the corresponding model uncertainty, $\theta_{Q,i}$. The "independent re-occurrence period" for snow and wind is assumed to be 1 year, however, if 5 years would be assumed for imposed load a 99%-fractile would correspond to 1.38. The tabulated value of 1.35 was conventionally assigned.

Load variables, self weight G_S, permanent load G_P, variable loads as snow, wind and imposed load, Q_j are also represented as normalised random variables. Variable loads are represented with the distribution of their yearly extreme values (5 year extreme value in case of imposed loads). The corresponding model uncertainties are also specified in Table 4.
In order to represent a range of generalised design equations that are characterised by different relative contributions of permanent and variable loads, the parameters a_Q and a_G are varied within the specified ranges in Table 5. Here, also the relative importance (frequency) of the different failure modes is specified in terms of weighting factors w_k.

Table 5: Load Contributions and weight per design case.

Variable	a_q	a_g	w_k
Steel bend.	0.3 – 0.8	0.6 – 1.0	0.170
Concrete comp.	0.1 – 0.7	0.6 – 1.0	0.12
Concrete bend.	0.1 – 0.7	0.6 – 1.0	0.4
Glulam bend.	0.2 – 0.8	0.6 – 1.0	0.035
Timber bend.	0.2 – 0.8	0.6 – 1.0	0.017
Masonry	0.1 – 0.7	0.6 – 1.0	0.12
Aluminium bend.	0.3 – 0.8	0.6 – 1.0	0.017
Pile found.	0.1 – 0.5	0.6 – 1.0	0.06
Shallow found.	0.1 – 0.5	0.6 – 1.0	0.06

Comment: A uniform distribution between the limits is assumed. The sensitivity of the results to other assumed distributions is addressed later in the document.

4. STATUS QUO

The average reliability indices for the following partial load factors are computed:

$$\begin{aligned}
\text{Permanent action:} \quad & \gamma_G = 1.35 \\
\text{Wind, Snow, Imposed:} \quad & \gamma_Q = 1.5 \\
\text{Combination Factor } G\text{:} \quad & \xi = 0.85 \\
\text{Combination Factor } Q_{imp}\text{:} \quad & \psi_{imp} = 0.7 \\
\text{Combination Factor } Q_s\text{:} \quad & \psi_s = 0.5 \\
\text{Combination Factor } Q_w\text{:} \quad & \psi_w = 0.6
\end{aligned}$$

Table 6: Weighted average and crude average 50 year reliability index β per design equation.

	(8.12)	(8.13)	(8.14)
Weighted average	3.74	3.51	3.48
Crude average	3.65	3.43	3.41

The crude average is computed as the average of all considered cases. The weighted average (Eq. 2) incorporates the importance of the different cases. E.g. a similar number of classes is considered for concrete in compression and aluminium in bending, however, concrete structures are more frequent and therefore weighted larger.
In Figure 1 it can be seen that by applying design equation 8.12 the highest average reliability index is obtained, which is also close to the reliability target $\beta = 3.8$ for a 50 years reference period. Applying design equations 8.13 and 8.14 leads to lower reliability indices in average. However, the scatter in reliability index is smaller when following design equations 8.13 and 8.14, i.e. applying these design equations leads to a more consistent reliability level among different design situations.
Consistency among different load combinations is illustrated in Figure 2. Here it is seen that applying design equation (8.14) leads to design solutions with the most consistent reliability.

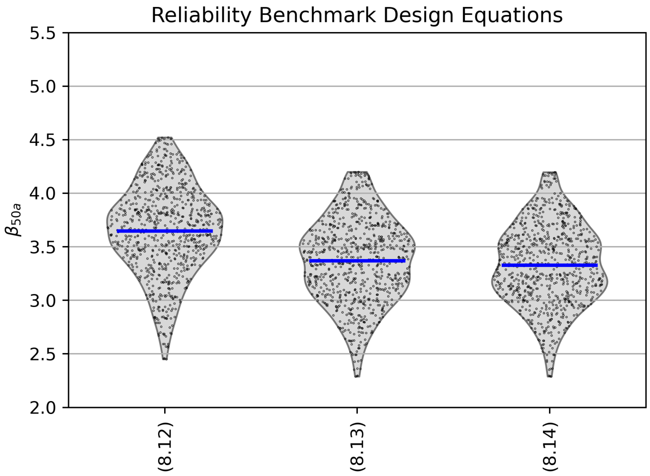

Figure 1: Beta indices with existing partial factors according to design equations 8.12., 8.13 and 8.14. The corresponding standard deviations are 0.46, 0.39 and 0.37 correspondingly.

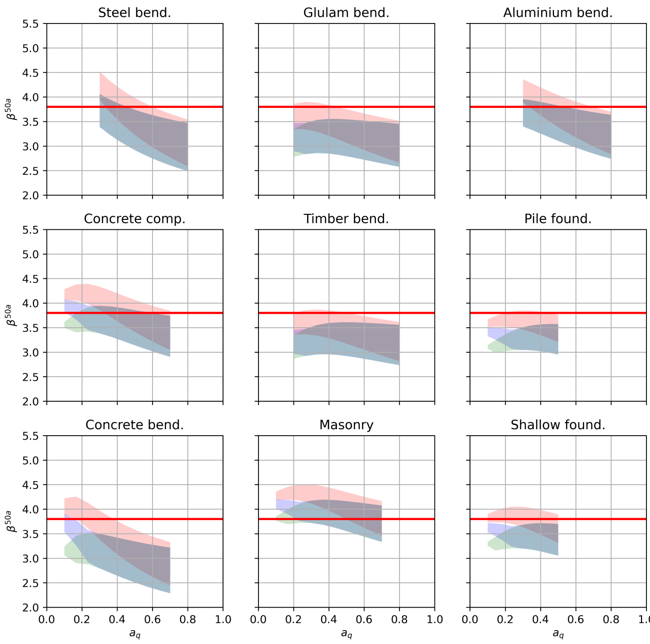

Figure 2: Beta indices with existing partial factors according to design equation 8.12. (red), 8.13. (blue) and 8.14. (green). a_q indicates the contribution of variable load, i.e. large value - large contribution. The target reliability is indicated with a red bar.

5. POTENTIAL FOR CALIBRATION

Systematic approaches to code calibration have been addressed e.g. in Cornell (1969), Ravindra et al. (1978), Thoft-Christensen P. (1982), Madsen H.O. (2006), Ditlevsen O. (2007) and Baravalle and Köhler (2017). Faber and Sørensen (2003) introduced a 7-step approach for reliability based code calibration, which was the basis for the corresponding standardisation in ISO2394 (2015).

If the partial factors of an existing code are calibrated similar principles should be followed. However, the existence of a semi-probabilistic design framework that is practically applied for years requires some adaption, so, the results presented herein are considered as preliminary and indicative. The current safety format of the Eurocodes is characterised by a high degree of generalisation. Variable loads, for example, are all factorised by the same partial factor γ_Q even though the underlying uncertainties in representing the variable loads are rather different. If partial factors would be differentiated and calibrated in order to minimise the variability of the resulting reliability indices, design solutions that are more consistent with the reliability criterion could be identified.

An alternative set of partial load factors $\gamma_S = (\gamma_G, \gamma_{Q,wind}, \gamma_{Q,snow}, \gamma_{Q,imposed})$ is specified based on the following objective function, where the sum of squared deviations from a target reliability level is minimized.

$$M(\gamma_S) = \sum_{k \in D} w_k \left(\beta_k(\gamma_S) - \beta_{target}\right)^2 \qquad (3)$$

Correspondingly, the following partial factors have been calibrated:

Permanent:	$\gamma_G = 1.19$
Variable Wind:	$\gamma_{Q,wind} = 1.94$
Variable Snow:	$\gamma_{Q,snow} = 2.35$
Variable Imposed:	$\gamma_{Q,imposed} = 1.60$

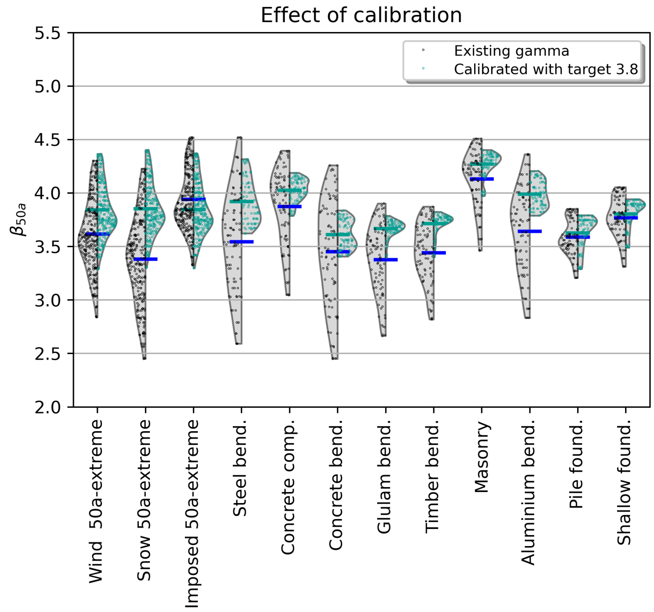

Figure 3: Beta indices with existing partial factors according to design equation 8.12. compared to the resulting beta indices that correspond to the set of calibrated partial load factors.

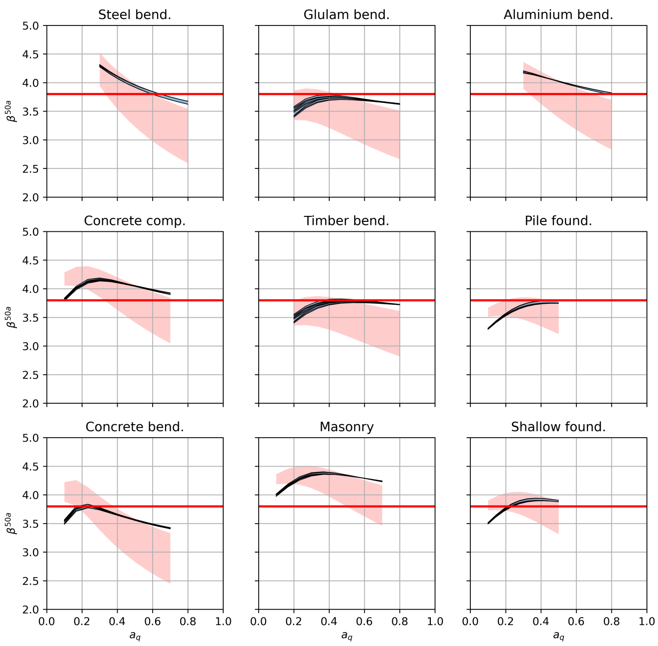

Figure 4: Beta indices with existing partial factors according to design equation 8.12. (red) compared to the resulting beta indices that correspond to the set of calibrated partial load factors, displayed for different load contributions a_q.

The effect on the variability of the reliability index and the consistency among different load contributions (a_q) is indicated in Figure 3 and (4). Due to the calibration, the standard deviation is dropping from $st.dev._{EC}$ =0.46 to $st.dev._{cal}$ =0.24. It can also be seen that the variability for the calibrated case is dominated by in-between material variability, i.e. the variability within materials are even much lower than indicated by $st.dev._{cal}$ =0.24.

6. CONCLUSIONS AND OUTLOOK

Applying the design equations 8.12, 8.13 or 8.14 has significant implication on the obtained reliability level. Applying 8.12 leads to the highest average reliability levels (3.74), which is also the closest to the prescribed target level of 3.8 (all values corresponding to a 50 year reference period). The average values corresponding to design equations 8.13 (3.51) and 8.14 (3.48) fall significantly below the prescribed target. However, all indicated absolute values for the reliability index are conditional to the underlying assumptions and the found deviation between the average reliability indices and the reliability target should be considered with adequate moderation. The first generation of the Eurocodes is applied for roughly 20 years now and either design equation 8.12, 8.13 or 8.14 are applied in different countries. Up to now, no evidence on higher failure rates as a result of the application of these safety formats has been reported. Thus, structural performance seems to be perceived acceptable.

Another interesting property that is more robust against the particular model assumption is the resulting variability of the reliability indices among different design situation. This indicates on how the generalisation of the safety format affects the efficiency of the design code. Applying design equation 8.12 leads to a rather high variability, i.e. β-values between 2.4 and 4.5 are obtained, the standard deviation among the considered cases is 0.46. This means that the simplicity of the safety format leads to a very high variation in reliability level and therewith to a rather sub-optimal utilisation of resources. Given the important role of the building sector for material consumption and the broad application of the Eurocode safety concept, this lack of efficiency is seen as a major obstacle towards a more sustainable development. Applying design equations 8.13 and 8.14 leads to slightly lower variability in the reliability index.

The potential of the calibration of the partial factors is indicated for a case where individual safety factors for different types of loads are considered. This is seen as a rather moderate increase of the complexity of the safety format. However, the effect on the variability of the reliability of the obtained design solutions is significant, i.e. the standard deviation dropped from 0.46 to 0.24. This indicates the huge potential a possible calibration of the safety concept would have.

To date the built environment is developed and maintained by broadly following structural design standards, which did evolve continuously over time and contain safety concepts that support daily structural engineering decision making based on simple calculus. The major objective that has been followed in their development, was the provision of sufficient safety, and the observed relative low failure rates do proof success in this regard. However, as conceived, structural design codes do not allow for the optimal allocation of the limited financial and environmental resources into structural performance. An improvement of this situation can be achieved by a reliability-based calibration of the standardised decision rules. As shown in the paper, such a calibration would reduce the significant scatter of the reliability level associated with different design situations, while maintaining the present average safety level.

7. ACKNOWLEDGEMENTS

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