

Optimal, reliability-based code calibration

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Reliability based code calibration is considered in this paper. It is described how the results of FORM based reliability analysis may be related to the partial safety factors and characteristic values. The code calibration problem is presented in a decision theoretical form and it is discussed how acceptable levels of failure probability (or target reliabilities) may be established. Furthermore suggested values for acceptable annual failure probabilities are given for the ultimate and the serviceability limit states. Finally the paper describes a procedure for the practical implementation of reliability based code calibration of LRFD based design codes.

Key words: code calibration, stochastic models, target reliabilities, partial safety factors, reliability analysis, decision theory.

1. Introduction

Design codes are normally based on design equations from which the reliability verification of a given design may be performed by a simple comparison of resistances and loads and/or load effects. Since loads and resistances are subject to uncertainties, design values for resistances and load effects are introduced in the design equations to ensure that the design is associated with a satisfactory level of reliability. Design values for resistances are introduced as a characteristic value of the resistance divided by a partial safety factor and design values for load effects are introduced as characteristic values multiplied by a partial safety factor. Furthermore, in order to take into account the effect of simultaneously occurring variable load effects so-called load combination factors are multiplied on one or more of the variable loads.

During the last decades different approaches for establishing design values for resistances and loads have been applied in different countries. However,

almost all design codes use the Partial Safety Factor (PSF) method also named the Load and Resistance Factor Design format (LRFD). Different versions of the LRFD format exist see e.g. CIRIA [1], CEB [2] and [3], Eurocodes [4], AHSTO [5] and OHBDC [6], but they are essentially based on the same principles.

The development of structural reliability methods during the last decades has provided a more rational basis for the design of structures in the sense that these methods facilitate a consistent basis for comparison between the reliability of well tested structural design and the reliability of new types of structures. Therefore the methods of structural reliability have been applied increasingly in connection with the development of new design codes over the last decades.

By means of structural reliability methods the safety formats of the design codes, i.e. the design equations, characteristic values and partial safety factors may be chosen such that the level of reliability of all structures designed according to the design codes is approximately homogeneous and independent of the choice of material and the loading. This process including the choice of the desired level of reliability or target reliability is denoted code calibration. Reliability based code calibration has been formulated by several researchers, see e.g. Ravindra and Galambos [7], Ellingwood et al. [8] and Rosenblueth and Esteva [9] and has also been implemented in several codes, see e.g. OHBDC [6], NBCC [10] and more recent the Eurocodes [4]. The present paper gives an overview of the methods applied in reliability based code calibration, see also Faber and Sørensen [11].

2. Partial safety factors

In code based design formats such as the Eurocodes [4], design equations are used for the verification of the capacity of different types of structural components and for different modes of failure. The typical format for the verification of a structural component is given as design equations such as:

$$G = R_d - S_d = \frac{zR_c}{\gamma_m} - (\gamma_{G_a}G_c + \gamma_QQ_C) \geq 0 \quad (2.1)$$

where:

R_d – design value for load bearing capacity,

S_d – design value for load,

R_C – characteristic value for the resistance (typically the 5% quantile in the distribution function for the strength),

z – design variable (e.g. the cross sectional area),

- G_C – characteristic value for the permanent load (typically the 50% quantile in the distribution function for the permanent load),
- Q_C – characteristic value for the variable load (typically the 98% quantile in the distribution function for the annual maximum variable load),
- γ_m – partial safety factor for the resistance,
- γ_G – partial safety factor for the permanent load,
- γ_Q – partial safety factor for the variable load.

In the codes different partial safety factors are specified for different materials and for different types of loads. Furthermore, when more than one variable load is acting load combination factors are multiplied on one or more of the variable load components to take into account the fact that it is unlikely that all variable loads are acting with extreme values at the same time.

The partial safety factors together with the characteristic values are introduced in order to ensure a certain minimum reliability level for the structural components designed according to the code. As different materials have different uncertainties associated with their material parameters the partial safety factors are in general different for the different materials. The principle is illustrated in Figs. 1 and 2.

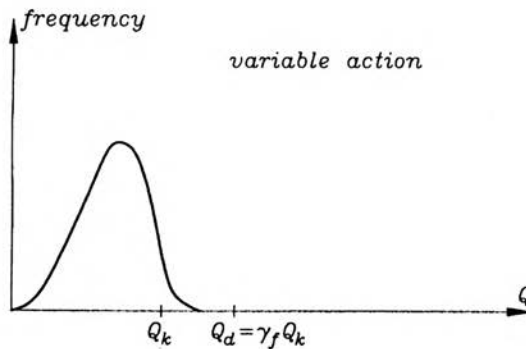


FIGURE 1. Illustration of the relation between design values, characteristic values and partial safety factors for variable loads.

In accordance with a given design equation, e.g. (2.1) a reliability analysis may be made with a limit state function of the same form as the design equation but where the characteristic values for the resistance and load variables are now replaced by basic random variables, i.e.,

$$g = zR - (G + Q) = 0. \quad (2.2)$$

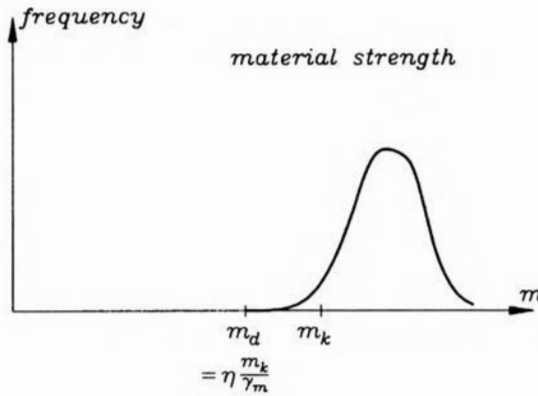


FIGURE 2. Illustration of the relation between design values, characteristic values and partial safety factors for a material strength parameter m .

For given probabilistic models for the stochastic variables R, G and Q and with a given requirement to the maximum allowable failure probability the value of the design variable z can be determined. Such a design could be interpreted as being an optimal design because it exactly fulfils the given requirements to structural reliability.

Having determined the optimal design z we may also calculate the corresponding design point in the original space, i.e. \mathbf{x}_d for the basic random variables, denoted \mathbf{X} . This point may be interpreted as the most likely failure point, i.e. the most likely combination of the outcomes of the basic random variables leading to failure. Partial safety factors may be determined from the design point for the resistance variables as

$$\gamma_m = \frac{x_c}{x_d} \quad (2.3)$$

and for load variables as

$$\gamma_Q = \frac{x_d}{x_c} \quad (2.4)$$

where x_d is the design point for the considered design variable and x_c the corresponding characteristic value.

For time-variant reliability problems a similar procedure can be used to determine partial safety factors.

3. The code calibration decision problem

In this section it is described how the code calibration problem can be formulated as a decision problem. Two levels of code calibration can be formu-

lated, namely calibration of target reliabilities (or probabilities of failure) and direct calibration of the partial safety factors. Calibration/determination of target reliabilities are considered in section 4. Here it is described how partial safety factors using a decision theoretical approach can be calibrated. A general formulation based on decision theoretical concepts is obtained when the total expected cost-benefits for a given class of structures are maximized with the partial safety factors as decision variables, see e.g. Sørensen et al. [12]:

$$\max_{\gamma} W(\gamma) = \sum_{j=1}^L w_j \left[B_j - C_{Ij}(\gamma) - C_{Rj}(\gamma) - C_{Fj} P_{Fj}(\gamma) \right] \quad (3.1)$$

$$\text{s.t. } \gamma_i^l \leq \gamma_i \leq \gamma_i^u, \quad i = 1, \dots, m,$$

where $\gamma = (\gamma_1, \dots, \gamma_m)$ are the m partial safety factors to be calibrated. Load combination factors will in general also be calibrated/optimized, therefore $\gamma = (\gamma_1, \dots, \gamma_m)$ can be assumed also to contain those load combination factors to be calibrated. $\gamma_1^l, \dots, \gamma_m^l$ and $\gamma_1^u, \dots, \gamma_m^u$ are lower and upper bounds on the partial safety factors. L is the number of different failure modes/limit states used to cover the application area considered. w_j is a factor indicating the relative frequency of failure mode j . B_j is the expected benefits (in general for the society, but in some cases the benefits can be related to the owner of the structures considered), C_{Ij} is the initial (or construction) costs, C_{Rj} is the repair/maintenance costs during the design life time and C_{Fj} is the cost of failure. C_{Fj} is assumed to be independent of the partial safety factors. P_{Fj} is the probability of failure for failure mode j if the structure is designed using given partial safety factors.

The formulation in (3.1) is based on single failure modes and corresponds to the single failure mode checking format used in structural codes of practice. A similar systems approach can be formulated where the probability of failure of the system can be determined assuming system failure if one of the single failure modes fails (series system model) and where systems related costs are introduced. However, the corresponding deterministic systems reliability measures (robustness measures) are difficult to identify and are generally not used in structural codes. In the following the single failure mode checking format is assumed to be used.

The limit state functions related to the failure modes considered are written as follows:

$$g_j(\mathbf{x}, \mathbf{p}_j, \mathbf{z}) = 0 \quad (3.2)$$

where \mathbf{p}_j is a vector with deterministic parameters and $\mathbf{z} = (z_1, \dots, z_N)$ are the design variables. The application area for the code is described by the set I of L different vectors \mathbf{p}_j , $j = 1, \dots, L$. The set I may, e.g., contain different

geometrical forms of the structure, different parameters for the stochastic variables and different statistical models for the stochastic variables.

The deterministic design equation related to the limit state equation in (3.2) is written as follows:

$$G_j(\mathbf{x}_c, \mathbf{p}_j, \mathbf{z}, \gamma) \geq 0 \quad (3.3)$$

where $\mathbf{x}_c = (x_{c1}, \dots, x_{cn})$ are characteristic values and $\gamma = (\gamma_1, \dots, \gamma_m)$ are the partial safety factors.

$C_{Ij}(\gamma)$, $C_{Rj}(\gamma)$ and $P_{Fj}(\gamma)$ can be determined on the basis of the solution of the following deterministic optimization problem where the optimal design \mathbf{z} is determined using the design equations and given partial safety factors:

$$\begin{aligned} & \min_{\gamma} C_{Ij}(\mathbf{z}) \\ & \text{s.t. } G_j(\mathbf{x}^c, \mathbf{p}_j, \mathbf{z}, \gamma) \geq 0, \\ & z_i^l \leq z_i \leq z_i^u, \quad i = 1, \dots, N. \end{aligned} \quad (3.4)$$

The objective function in (3.4) is the construction costs, and the constraints are related to the design equations. Using the limit state equation in (3.2) the probability of failure of the structure P_{Fj} and the expected repair/maintenance costs C_{Rj} to be used in (3.1) are determined at the optimum design point \mathbf{z}^* . In cases where more than one failure mode is used to design a structure included in the code calibration, the relevant design equations all have to be satisfied for the optimal design \mathbf{z}^* . The objective function in (3.4) can be extended also to include the repair/maintenance costs and the benefits.

It is noted that when the partial safety factors are determined from (3.1) they will in general not be independent. In the simplest case with only a resistance partial safety factor and a load partial safety factor only the product of the two partial safety factors is determined.

4. Optimality and target reliabilities

The reliability of a structure is normally estimated on the basis of a given set of probabilistic models for loads and resistances that may have limited relevance for the actual reliability of the structure. This is the case when the probabilistic modelling forming the basis of the reliability analysis is highly influenced by subjectivity. Then the estimated reliability should be interpreted as being a measure for comparison only. In these cases it is thus not immediately possible to judge whether the estimated reliability is

sufficiently high without first establishing a more formalized reference for comparison.

Such a reference may be established by the definition of an optimal or best practice structure. The idea behind the “best practice” reference is that if the structure considered has been designed according to the “best practice” then the reliability of the structure is “optimal” according to agreed conventions for the target reliability. Typical values for the corresponding target annual failure probability are in the range of 10^{-5} to 10^{-7} depending on the type of structure and the characteristics of the considered failure mode. Using this approach the target reliability is determined as the reliability of the “best practice” design as assessed with the given probabilistic model.

The determination of the “best practice” design can be performed in different ways. The simplest approach is to use the existing codes of practice for design as a basis for the identification of “best practice” design. Alternatively the “best practice design” may be determined by experts.

In case where the probabilistic modelling is not based on subjective assessments the most rational approach is to establish the optimal design on the basis of the economic decision theory. By considering the expected total benefit $E[B]$ associated with the considered structure

$$\begin{aligned} E[B] &= I(1 - P_F(C_D)) - C_D - C_F P_F(C_D) \\ &= I - C_D - (I + C_F)P_F(C_D), \end{aligned} \quad (4.1)$$

where I is the expected benefit from the structure, C_F is the cost consequence in case of failure, C_D is the cost of some risk reducing measure, e.g. an increase of a dimension, and where the probability of failure is a function of the costs invested in the risk reduction we have that the optimal investment in risk reducing measures may be determined from the following optimality criterion

$$\frac{\partial E[B]}{\partial C_D} = -1 - (I + C_F) \frac{\partial P_F(C_D)}{\partial C_D} = 0 \quad (4.2)$$

from which the cost efficient level of risk reducing measures may be determined. Having determined these we may investigate the feasibility of the considered structure taking into account that the total expected benefit of the structure has to be larger than zero.

Without going into the details it is noted that it is possible, based on recent research work by Nathwani and Lind [13] and Rackwitz [14] to establish optimal values for risk reduction costs when also the consequences of loss of human lives are considered by means of the Life Quality Index. The Life Quality Index, L is a compound social indicator defined as

$$L = g^w e^{1-w} \quad (4.3)$$

where g is the gross domestic product per year per person, e is the life expectancy at birth and w is the proportion of life spent in economic activity. In developed countries it may be assumed that $w=1/8$. g lies in the interval of \$US 2600-14000 ranging from poor to well developed countries. The life expectancy at birth e being 56 years in poorly developed countries, 67 years in medium developed countries and 73 years in highly developed countries, see e.g. Skjong and Ronold [15].

In Tables 1 and 2 target failure probabilities and corresponding target reliability indexes are given for ultimate limit states and serviceability limit states, respectively based on the recommendations of JCSS [16]. Note that the values given correspond to a one year reference period and the stochastic models recommended in JCSS [16].

TABLE 1. Tentative target reliability indices β (and associated target failure probabilities) related to a one-year reference period and ultimate limit states.

Relative cost of safety measure	High	Normal	Low
Minor consequences of failure	$\beta = 3.1$ ($P_F \approx 10^{-3}$)	$\beta = 3.7$ ($P_F \approx 10^{-4}$)	$\beta = 4.2$ ($P_F \approx 10^{-5}$)
Moderate consequences of failure	$\beta = 3.3$ ($P_F \approx 5 \cdot 10^{-4}$)	$\beta = 4.2$ ($P_F \approx 10^{-5}$)	$\beta = 4.4$ ($P_F \approx 10^{-5}$)
Large consequences of failure	$\beta = 3.7$ ($P_F \approx 10^{-4}$)	$\beta = 4.4$ ($P_F \approx 5 \cdot 10^{-5}$)	$\beta = 4.7$ ($P_F \approx 10^{-6}$)

TABLE 2. Tentative target reliability indices (and associated probabilities) related to a one-year reference period and irreversible serviceability limit states.

Relative cost of safety measure	High	Normal	Low
Target index (irreversible SLS)	$\beta = 1.3$ ($P_F \approx 10^{-1}$)	$\beta = 1.7$ ($P_F \approx 5 \cdot 10^{-2}$)	$\beta = 2.3$ ($P_F \approx 10^{-2}$)

5. A practical code calibration procedure

Code calibration can be performed by judgement, fitting, optimization or a combination of these, see Madsen et al. [17] and Thoft-Christensen & Baker [18]. Calibration by judgement has been the main method until 10-20 years ago. Fitting of partial safety factors in codes is used when a new code format is introduced and the parameters in this code are determined, e.g., such that the same level of safety is obtained as in the old code or calibrated

to a target reliability level. In practical code optimization the following steps are generally performed:

1. *Definition of the scope of the code,*
2. *Definition of the code objective,*
3. *Definition of code format,*
4. *Identification of typical failure modes and of stochastic model,*
5. *Definition of a measure of closeness,*
6. *Determination of the optimal partial safety factors for the chosen code format,*
7. *Verification.*

Ad 1.

The class of structures and the type of relevant failure modes to be considered are defined.

Ad 2.

The code objective may be defined using target reliability indices or target probability of failures. Those can, for example, be based on the target reliabilities indicated in recommendations (e.g. JCSS [16], Eurocodes [4] or ISO [24]), see Sec. 4 or on reliabilities obtained by reliability analyses of structures designed by old, well-proven and accepted structural codes of practice. It is important to note that the target reliabilities are linked closely to the stochastic models used for the uncertain variables and the applied limit states.

Ad 3.

The code format includes:

- how many partial safety factors and load combination factors to be used,
- should load partial safety factors be material independent,
- should material partial safety factors be load type independent,
- how to use the partial safety factors in the design equations and
- rules for load combinations.

In general for practical use the partial safety factors should be as few and general as possible. On the other hand a large number of partial safety factors is needed to obtain economically and safe structures for a wide range of different types of structures.

Ad 4.

Within the class of structures considered typical failure modes are identified. Limit state equations and design equations are formulated and stochastic models for the parameters in the limit state equations are selected. Also the frequency at which each type of safety check is performed is determined.

The stochastic model for the uncertain parameters should be selected very careful. Guidelines for the selection can be found in JCSS [16]. Also in the Eurocodes [4] and ISO [19] some guidelines can be found. In general the following main recommendations can be made.

Strength/resistance parameters are often modeled by Lognormal distributions. This avoids the possibility of negative realizations. In some cases it can be relevant also to consider a Weibull distribution for a material parameter. This is especially the case if the strength is governed by brittleness, size effects and material defects. The coefficient of variation varies with the material type considered. Typical values are 5% for steel and reinforcement, 15% for the concrete compression strength and 15-20% for the bending strength of structural timber. The characteristic value is generally chosen as the 5% quantile.

Variable loads (imposed and environmental) can be modeled in different ways, see JCSS [16]. The simplest model is to use a stochastic variable modelling the largest load within the reference period (often one year). This variable is typically modeled by an extreme distribution such as the Gumbel distribution. The coefficient of variation is typically in the range of 20-40% and the characteristic value is chosen as the 98% quantile in the distribution function for the annual maximum load.

Permanent loads are typically modeled by a Normal distribution since it can be considered as obtained from many different contributions. The coefficient of variation is typically 5-10% and the characteristic value is chosen as the 50% quantile.

Model uncertainties are in many cases modeled by Lognormal distributions if the they are introduced as multiplicative stochastic variables and by Normal distributions if the they are modeled by additive stochastic variables. Typical values for the coefficient of variation are 3-15% but should be chosen very carefully. The characteristic value is generally chosen as the 50% quantile.

Ad 5.

The partial safety factors γ are calibrated such that the reliability indices corresponding to L different vectors \mathbf{p}_j are as close as possible to a target probability of failure P_F^t or equivalently a target reliability in-

dex $\beta_t = -\Phi^{-1}(P_F^t)$. This can be formulated by the following optimization problem:

$$\min_{\gamma} W(\gamma) = \sum_{j=1}^L w_j (\beta_j(\gamma) - \beta_t)^2 \quad (5.1)$$

where $w_j, j = 1, \dots, L$ are factors ($\sum_{j=1}^L w_j = 1$ indicating the relative frequency of appearance/importance of the different design situations. Instead of using the reliability indices in (5.1) to measure the deviation from the target, for example the probabilities of failure can be used:

$$\min_{\gamma} W'(\gamma) = \sum_{j=1}^L w_j (P_{Fj}(\gamma) - P_F^t)^2 \quad (5.2)$$

where P_F^t is the target probability of failure in the reference period considered. Also, a nonlinear objective function giving relatively more weight to reliability indices smaller than the target compared to those larger than the target can be used.

The above formulations can easily be extended to include a lower bound on the reliability or probability of failure for each failure mode.

Ad 6.

The optimal partial safety factors are obtained by numerical solution of the optimization problem in step 5. The reliability index $\beta_j(\gamma)$ for combination j given the partial safety factors γ is obtained as follows. First, for given partial safety factors γ the optimal design is determined.

If the number of design variables is $N = 1$ then the design \mathbf{z}^* can be determined from the **design equation**, see (3.3),

$$G_j(\mathbf{x}_c, \mathbf{p}_j, \mathbf{z}, \gamma) \geq 0. \quad (5.3)$$

If the number of design variables is $N > 1$ then a design optimization problem can be formulated as follows:

$$\begin{aligned} \min C(\mathbf{z}) \\ \text{s.t. } c_i(\mathbf{x}_c, \mathbf{p}_j, \mathbf{z}, \gamma) &= 0, \quad i = 1, \dots, m_e, \\ c_i(\mathbf{x}_c, \mathbf{p}_j, \mathbf{z}, \gamma) &\geq 0, \quad i = m_e + 1, \dots, m, \\ z_i^l &\leq z_i \leq z_i^u, \quad i = 1, \dots, N. \end{aligned} \quad (5.4)$$

C is the objective function and $c_i, i = 1, \dots, m$ are the constraints. The objective function C is often chosen as the weight of the structure. The m_e

equality constraints in (5.4) can be used to model design requirements (e.g. constraints on the geometrical quantities) and to relate the load on the structure to the response (e.g. finite element equations). Often equality constraints can be avoided because the structural analysis is incorporated directly in the formulation of the inequality constraints. The inequality constraints in (5.4) ensure that response characteristics such as displacements and stresses do not exceed codified critical values as expressed by the design (5.3). The inequality constraints may also include general design requirements for the design variables. The lower and upper bounds, z_i^l and z_i^u , to z_i in (5.4) are simple bounds. Generally, the optimization problem (5.4) is non-linear and non-convex.

Next, the reliability index $\beta_j(\gamma)$ is estimated by FORM/SORM or simulation on the basis of the limit state equations (3.2) using the optimal design \mathbf{z}^* from (5.3) or (5.4).

Ad 7.

As discussed above a first guess of the partial safety factors is obtained by solving these optimization problems. Next, the final partial safety factors are determined taking into account current engineering judgment and tradition.

Examples of reliability-based code calibration can be found in Nowak [20], Sørensen et al. [21] and SAKO [22].

6. Example

The following representative limit state function is considered:

$$g = zRX_R - ((1 - \alpha)G + \alpha Q) \quad (6.1)$$

where:

R – strength,

X_R – model uncertainty,

z – design variable,

G – permanent load,

Q – variable load,

α – factor between 0 and 1, representing the relative fraction of variable load.

Three different materials with strengths R_1 , R_2 and R_3 and two different variable loads Q_1 and Q_2 are considered. The stochastic model is shown in Table 3. The table also shows the quantile values used to determine the characteristic value.

TABLE 3. Stochastic model.

Variable		Distribution type	COV	Quantile value
Permanent load	G	N	0.10	50%
Variable load (environmental load)	Q_1	G	0.40	98%
Variable last (imposed load)	Q_2	G	0.20	98%
Strength material 1 (steel)	R_1	LN	0.05	5%
Strength material 2 (concrete)	R_2	LN	0.15	5%
Strength material 3 (timber)	R_3	LN	0.20	5%
Model uncertainty	X_R	N	0.03 (steel) 0.05 (concrete) 0.05 (timber)	50%

The design variable $z = \max(z_1, z_2)$ is determined from the following two design equations corresponding to the equations (6.10a) and (6.10b) in Eurocodes, Basis of Structural Design, [4]:

$$\text{LC 1: } \frac{z_1 R_c}{\gamma_R} - \left((1 - \alpha) \gamma_G G_c + \alpha Q_c \right) = 0, \quad (6.2)$$

$$\text{LC 2: } \frac{z_2 R_c}{\gamma_R} - \left((1 - \alpha) \xi_G \gamma_G G_c + \alpha \psi \gamma_Q Q_c \right) = 0, \quad (6.3)$$

where: index c indicates characteristic value and

γ_G – partial safety factor for permanent load in LC 1 (variable load dominating),

γ_Q – partial safety factor for variable load in LC 1 (variable load dominating),

ψ – factor for variable load in LC 2 (permanent load dominating),

ξ_G – factor for permanent load in LC 2 (permanent load dominating),

γ_R ($= \gamma_{R_1}, \gamma_{R_2}$ and γ_{R_3}) – partial safety factor for strength.

The application area is defined as follows:

- 3 materials,
- 2 variable loads,
- 9 different α values: 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8 and 0.9.

Thus the application area consists of $L = 3 \cdot 2 \cdot 9 = 54$ different failure modes. It is assumed that $w_j = 1$, $j = 1, \dots, L$. It is chosen that $\gamma_G = 1$ and $\psi = 0.6$. In Table 4 optimal partial safety factors are shown for the case where the code optimization problem in (5.1) is considered and the target reliability index is 3.8, 4.3 and 4.8. It is seen partial safety factor for the environmental load becomes rather high for $\beta_t = 4.8$ reflecting that the uncertainty for Q_1 is relatively high and dominating.

TABLE 4. Calibrated partial safety factors.

	$\beta_t = 3.8$	$\beta_t = 4.3$	$\beta_t = 4.8$
γ_G	1	1	1
γ_Q environmental load	1.39	1.60	1.83
γ_Q imposed load	1.24	1.38	1.53
ξ_G	1.14	1.16	1.18
γ_{R_1} steel	1.11	1.13	1.16
γ_{R_2} concrete	1.21	1.27	1.33
γ_{R_3} timber	1.27	1.35	1.45

7. Conclusion

Using decision theoretical principles and methods from structural reliability it is shown how rational and cost efficient design code partial safety factors can be determined consistent with the load and resistance uncertainties. The important items to be considered in consistent, reliability-based code calibration are an international understanding on:

1. Selection of stochastic models for the uncertain parameters (distribution types and quantification of coefficient of variations).
2. Choice of target reliability levels, which have to be related to the stochastic model, recommended reliability levels and/or reliability levels in existing well proven structural codes. The optimal reliability levels could be based on cost optimal, decision theoretical considerations.
3. Code formats, including load combination rules.
4. Selection of structures/failure modes covering the application area of the code.

These items need general recommendations especially for national code committees.

In order to benefit from reliability based code calibration it is very important that consistency exists between all steps in the probabilistic representation of uncertainties and limit state functions for the application domain of the code in terms of types of structures, load conditions and materials. This may give problems in practice when, e.g., for reasons of tradition some partial safety factors are selected by choice in accordance with previous codes and afterwards combined with partial safety factors derived on the basis of reliability analysis.

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