Lifetime and annual structural reliability implicitly required by the Eurocodes

Nivel de fiabilidad de por vida útil y anual, implícitamente requerido por los Eurocódigos

Peter Tanner [a], Ramon Hingorani [b], Carlos Lara [c]

[a] PhD Civil Engineering, Instituto de Ciencias de la Construcción Eduardo Torroja (IETcc-CSIC), Madrid, tannerp@ietcc.csic.es

[b] PhD Civil Engineering, SINTEF, Trondheim, ramon.hingorani@sintef.no

[c] PhD Civil Engineering, Instituto de Ciencias de la Construcción Eduardo Torroja (IETcc-CSIC), Madrid, carloslara@ietcc.csic.es

ABSTRACT

This article addresses the reliability level implicitly required in the Eurocodes. Probabilistic models that reflect the state of uncertainty associated with the basic variables relevant for structural design are derived and used to establish the lifetime reliability level inherent in the Eurocodes for a large portfolio of design situations. While the obtained average reliability indices generally exceed the nominal target value, the results show a large scatter and non-negligible percentages of design situations do not comply with this requirement. In addition, annual reliabilities are established, which confirm the conservatism of the corresponding Eurocode requirement.

RESUMEN

En este artículo se aborda el nivel de fiabilidad implícitamente requerido en los Eurocódigos. Para un periodo de referencia de 50 años se derivan modelos probabilistas para las variables básicas, relevantes a efectos del dimensionado estructural que, considerando un gran número de situaciones de proyecto, se utilizan para establecer el nivel de fiabilidad inherente a las reglas de los Eurocódigos. Aunque los índices de fiabilidad medio generalmente exceden el valor nominal requerido, la dispersión de los resultados es grande, con porcentajes apreciables de casos que no cumplen con este requisito. Además, se establecen fiabilidades anuales que confirman el conservadurismo del requisito correspondiente del Eurocódigo.

KEYWORDS: uncertainties, probabilistic models, target reliability, Eurocodes.

PALABRAS CLAVE: incertidumbres, modelos probabilistas, fiabilidad requerida, Eurocódigos.

1. Introduction

In structural engineering, new developments are traditionally based on careful extrapolations of design rules from the past that have led to satisfactory results over a sufficiently long period of time so that the corresponding performance levels are generally accepted. This applies not least to the level of safety inherent to the present design codes and standards. Indeed, it is a common approach to establish the fundamentals for making decisions regarding the acceptance of failure probabilities and ultimately the acceptance of risks related to new structures on the basis of risks experienced and apparently accepted in the past [1]. Regarding the Eurocode [2], for instance, the design rules provided are derived on the basis of a "calibration to a long experience of building tradition", along with the appreciation that risks associated with structural failure are nowadays reasonably low [3].

While the overall level of risks associated with civil engineering structures designed to current Eurocode practices appears to be generally acceptable, previous studies by the authors have shown that such risks diverge widely [4 - 6], pointing to non-optimal design decisions and, hence, to inefficient allocation of societal resources for the purpose of risk reduction. In part, the significant scatter observed may be attributed to the necessary approximations and simplifications introduced in the codified design rules to facilitate their straightforward application in daily practice [3, 7]. However, there is still a large potential for a more consistent formulation of these rules.

One of the principal matters of concern is the missing transparency in the relationship between the established nominal reliability requirements in codes such as the Eurocode, in terms of target values for reliability indices β_{EC}, and the set of partial factors provided in the same codes for practical design and verification purposes. Although this set may, with some exceptions (e.g. structural design dominated by wind or snow loads) be considered as being broadly in line with the well-known target reliability index $\beta_{EC,50} = 3.8$ [8, 9] (reference period $T_{ref} = 50$ years, Consequence Class CC2 [2]), several authors manifest doubts on their precise connection. It is observed [10], for instance, that EN 1990:2002 [2] defines the same β_{EC} as minimum reliability requirement, on one hand, and on the other, as target values for calibration procedures, what would suggest their interpretation as average values. Indeed, a clear distinction should be made between the two. As an inherent feature of any calibration process, regardless of the numerical value used as the reliability target for the calibration of partial factors, a certain fraction of structures correctly designed using those factors will achieve a reliability below the corresponding target. In this sense, the Eurocode background guidance admits that a certain proportion of construction works designed according to the partial factor format may not comply with the $\beta_{EC,50} = 3.8$ requirement [3], without, however, providing quantitative estimates. In contrast to earlier results obtained by the Joint Nordic Group for Structural Matters [7], more recent studies [11, 12] suggest that this proportion is generally large. The cited studies report 50-year average reliability levels associated with Eurocode design that are generally below $\beta_{EC,50} = 3.8$. However, provided that any statement on reliability is dependent on the modelling detail (e.g. choice of portfolio of structural members and limit states) and the representation of uncertainties in the analysis [10], calls for a careful comparison of these results, which up to date has not been reported in the literature.

Another relevant matter is the choice of an appropriate reference period T_{ref} for the target reliability. In parallel to $\beta_{EC,50}$, the Eurocode specifies a set of annual requirements $\beta_{EC,1}$ based on the conservative assumption of mutually independent failure events in subsequent years. For CC2, this assumption results in an equivalency between $\beta_{EC,50} = 3.8$ and $\beta_{EC,1} = 4.7$. However, in many situations, failure events are unavoidably correlated to a certain degree, mainly due to the contribution of permanent loads and resistance variables [13]. For this reason, lower annual Eurocode target values than $\beta_{EC,1} = 4.7$ have been suggested in the past [14] and the Technical Report on the Reliability Background of the Eurocodes [15] suggests an average equivalency between $\beta_{EC,50} = 3.8$ and $\beta_{EC,1} = 4.4$ on the basis of tentative calculations. A closer look at the results of the same report shows different equivalences depending on the load combination considered and the resistance variability. Therefore, higher equivalent β_1 values are generally to be expected in steel structures than in concrete structures. For the latter, the fib Model Code [16] recommends using $\beta_1 = 4.2$ as roughly equivalent to $\beta_{50} = 3.8$, equivalence also advocated for in [12]. Interestingly enough, the same requirement, β_t = 4.2, is also recommended in other standards [17, 18] as the target reliability for all kinds of structures, based on monetary optimization assuming intermediate levels for both, failure consequences and relative cost of safety measures. Despite these considerations, the second generation of Eurocodes [19] keeps the current set of reliability requirements and partial factors unchanged.

In the aforementioned context, the objective of the paper is to derive and analyze the reliability level implicitly required by the current (and upcoming) set of Eurocodes. To achieve this goal, a large, representative set of structural members (≈ 3,000 beams and ≈ 116,000 columns) is defined. The members are designed using the Eurocode partial factor format, under explicit distinction of different failure modes, and then subjected to a structural reliability analysis. The probabilistic models specifically deduced for this purpose not only represent the physical properties of the respective basic variables. Inasmuch as they establish the link to the standardized Eurocode design rules based on the partial factor format, these models approximately represent the state of uncertainty associated with these rules.

Under use of these probabilistic models, the study addresses the determination of the lifetime (T_{ref} = 50 years) reliability level of the selected set of structural members (Section 2). This is followed by derivation of the annual reliability and subsequent establishment of equivalencies to the lifetime results (Section 3). Conclusions of the study are exposed in Section 4.

2. Lifetime reliability

2.1 Procedure

The procedure for determining the level of reliability implicitly required by the consistent set of Eurocodes on the basis of design [2], actions on structures [20, 21] and structural resistance [22 - 25] initially involves determining the scope of the study: types of structures (e.g. building structures, bridges) and types of failure (e.g. structural collapse, failure to meet serviceability requirements, fatigue) to consider, etc. (Fig. 1). Thereafter a series of hypothetical but realistic standard structural members are selected (Section 2.2). The most representative failure mechanisms for these members are then identified to formulate the respective limit state functions, LSF (Section 2.3). For the structural design variables involved in these LSF, probabilistic models are developed that represent the state of uncertainty associated with the rules specified in the Eurocodes (Section 2.4).

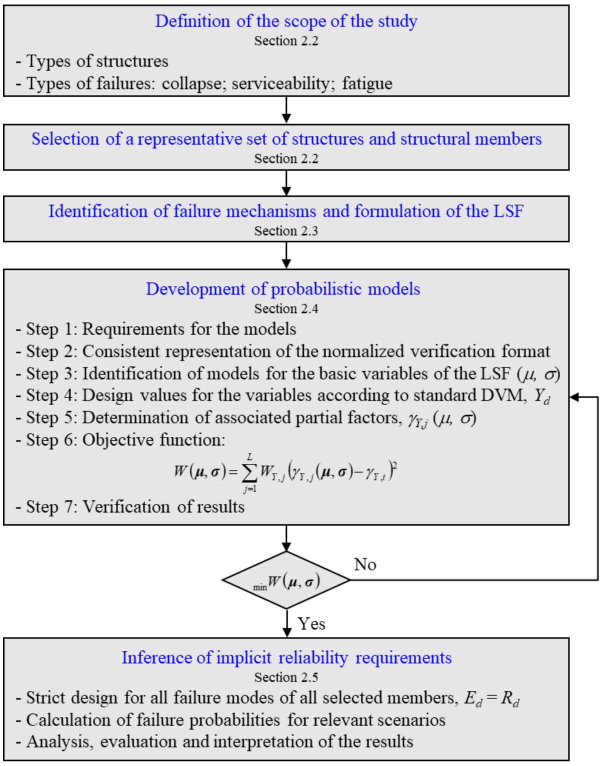

Figure 1. Procedure for determining the reliability level implicitly required by the Eurocodes

The structural members selected are subsequently designed strictly to code specifications. Designed strictly to code is defined to mean that structural member performance comply exactly with the structural safety requirements laid down in the codes in question, i.e., the design value of the action effects, E_d, equals the design value of the corresponding resistance, R_d, $E_d = R_d$. Lastly, using the previously developed probabilistic models, the implicitly acceptable failure probabilities for each representative failure mechanism are determined for all the members designed in such a way, and the results are evaluated (Section 2.5).

2.2 Selection of structural members

The procedure adopted is applied to building structures, considering exceedance of ultimate limit states in connection with persistent design situations. Realistic buildings with a maximum number of 30 storeys are chosen as the starting point since medium-rise residential and office buildings with heights of around or over 100 meters are common in many urban developments. It is also considered that buildings where people can congregate or public buildings where function is critical, such as hospitals, are typically built with fewer floors. In addition, and despite their boom in recent years, it is taken into account that tall buildings in which timber plays an important structural role are still rare.

Rather conventional, frame-like structural systems are considered, consisting of beams and columns as the main members. In such constructions, providing bracing systems to withstand horizontal forces entails a number of advantages over the sway frame design. Both horizontal displacements and column buckling lengths can be reduced and beam-to-column joints simplified. Floors and roofs would act as diaphragms, transmitting any horizontal forces to the bracing systems. The assumption is therefore that overall stability would be achieved through such systems, including areas of stairwells and elevators, that transfer the horizontal forces to the foundations. Since the load-bearing capacity of properly conceived and designed bracing systems for the types of structure under consideration generally exceeds the effects of wind actions and sway imperfections, these are not considered in the study, while the analysis of seismic actions would also go beyond its scope. In addition, member instability should be avoided by taking advantage of the bracing effect of the secondary elements, thereby freeing the full load-bearing capacity for vertical loads. Since the hypothesis adopted is conservative with regard to the reliability level, only statically determinate members are considered.

Table 1. Number of selected structural members

Constituent material	Number of members		
	Beams		Columns
	Roof	Floor	
Concrete	240	450	22,320
Steel	144	270	12,168
Composite	480	900	76,464
Timber	80	150	5,280
All	944	1,770	116,232

Based on the above considerations, representative sets of structural members made from different constituent materials such as reinforced concrete (in the following referred to as "concrete"), structural steel ("steel"), composite steel and concrete ("composite") and glued laminated timber ("timber") are established. The parameters with the greatest impact on the design are varied:

- Geometry: number of storeys, beam spans, spacing of columns;
- Constituent material and type;
- Actions: self-weight resulting from strict design, permanent loads, imposed loads, snow loads.

More details can be found in reference [26]. In all, the analysis covers 944 different roof girders, 1,770 floor girders and 116,232 types of columns (Table 1).

2.3 Limit State Functions

Failure modes due to bending moments (M) or shear forces (V) are considered for beams.

Compression axial force (N) induced failures are investigated for columns. The corresponding LSF's are represented below exemplary for floor beams made of steel. The LSF's for floor and roof beams as well as columns made of the constitutive materials considered in the study (steel, concrete, composite, timber) are equivalent. All LSF's are based on the relevant Eurocode formulations for the combination of actions and their effects for persistent design situations and the corresponding structural resistance. The symbols and abbreviations used are explained below for steel beams. It should be noted that no model uncertainties explicitly appear in the Eurocode design equations. Since such uncertainties must be taken into account when calculating structural reliability, variables are included in the LSF's for uncertainties in connection with the models for action effects and resistance, respectively θ_E and θ_R.
Equations (1) and (2) represent the LSF's formulated for, respectively, bending and shear failure of steel floor beams:

$$\theta_{R,M} \cdot W_{pl} \cdot f_y - \theta_{E,M} \cdot (M_{G,a} + M_{G,p,f} + M_Q) = 0 \quad (1)$$

$$\theta_{R,V} \cdot A_v \cdot \frac{f_{yw}}{\sqrt{3}} - \theta_{E,V} \cdot (V_{G,a} + V_{G,p,f} + V_Q) = 0 \quad (2)$$

$M_{G,a}$; $V_{G,a}$ bending moments (M) and shear forces (V) due to steel beam self-weight
$M_{G,p,f}$; $V_{G,p,f}$ bending moments (M) and shear forces (V) due to permanent floor load
M_Q; V_Q bending moments (M) and shear forces (V) due to variable action (imposed load)
$\theta_{E,M}$; $\theta_{E,V}$ uncertainties associated with action effect models for bending (M) and shear (V)
f_y; f_{yw} yield strength of steel for bending and shear, respectively
W_{pl}; A_v plastic section modulus and shear area, respectively
$\theta_{R,M}$; $\theta_{R,V}$ uncertainties associated with resistance models for bending (M) and shear (V).

2.4 Probabilistic models

The uncertainties associated with basic variables Y_i constituting the different LSF's are represented as random variables, in terms of a given type of probability density function and its parameters: the mean value, μ_{Yi}, and standard deviation, σ_{Yi}, translated into normalized, dimensionless quantities, i.e. the ratio S_{Yi} of the mean value μ_{Yi} to the characteristic value specified in the relevant Eurocode, Y_{ki}, and the coefficient of variation, V_{Yi} (σ_{Yi}/μ_{Yi}). It should be noted that the characteristic values for material properties are often associated to strength classes and therefore do not always correspond to the 5% fractile of the underlying probability distribution function according to the definition (of the characteristic value of material properties) in the Eurocode [2]. In such cases, e.g. for the yield strength of structural steel, these nominal values are used as characteristic values for material strength. Likewise, no percentile is defined for the characteristic value of imposed loads in [2], which is why the nominal values specified in [20] are used as characteristic values in the present study. Characteristic values of geometric properties are represented by their nominal values [2].
After identifying the most representative failure mechanisms of the previously selected members (Section 2.2) and formulating the corresponding limit state functions (Section 2.3), probabilistic models for the variables involved in these functions must be derived in a way that they are representative of the state of uncertainty associated with current design and construction practices. For this purpose, a procedure divided into different steps is followed, as shown in Fig. 1 (see box on "Development of probabilistic models"). The probabilistic models derived following that procedure are summarized in reference [26]. The results obtained are compatible with the models adopted in the probabilistic model code [17] or the background

document on the Eurocodes [15]. The derived probabilistic model for snow loads seems to be an exception to this general observation. This apparent inconsistency is due to the fact that the snow load models refer to different climatic regions: the model used to establish the reliability background in the Eurocodes [15] applies to regions with frequent snowfalls and is characterized by relatively high mean values and low coefficients of variation for a reference period of 50 years, while the opposite is true for the snow load model used in this study [26] (see also Section 3.1).

2.5 Implicit lifetime reliability requirements

Following the procedure illustrated in Fig. 1, the selected structural members (Section 2.2) are designed strictly according to the Eurocode specifications ($E_d = R_d$). Based on the established LSF (Section 2.3) and probabilistic models (Section 2.4), the reliability index, β_j, and the corresponding failure probability, $p_{f,j}$, for each representative failure mode, j, is then determined for all the members using probabilistic methods [27], implemented in the program [28]. The obtained β_j values, which can be interpreted as implicitly required reliability indices β_{50} ($T_{ref} = 50$ years), vary as a function of ratio ν between the variable loads and the total loads. These results are evaluated statistically in Fig. 2 in the form of box plots, indicating the minimum (β_{min}) and maximum (β_{max}) values of the respective samples, the upper and lower quartile values (limits of the box), the median (horizontal line inside the box) as well as the mean values μ_β (crosses).

Fig. 2 shows that the mean values μ_β are generally higher than the target value for the reliability index, recommended by the Eurocodes for consequence class CC2 and a reference period of T_{ref} = 50 years, $\beta_{EC,50}$ = 3.8. Nevertheless, taking account of the observed scatter, a certain percentage ($P_{3.8}$) of the obtained β values falls below $\beta_{EC,50}$ = 3.8, as anticipated in Section 1, based on [3]. Quantification of this percentage assuming normally distributed β's reveals significant scatter across the different member types and materials. While for composite and timber members, $P_{3.8}$ appears to be insignificant, it reaches values as high as 37% in case of steel roof beams (Table 2).

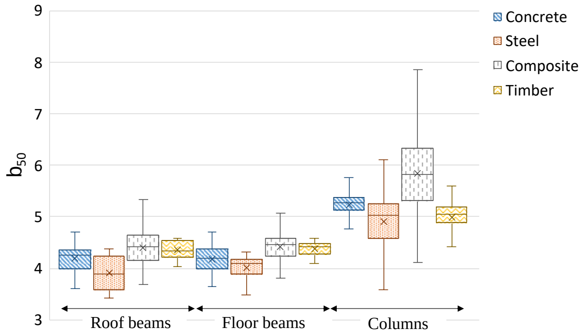

Figure 2. Box plots for β_{50} samples corresponding to specific structural member types of different constitutive materials; T_{ref} = 50 years

Table 2. Percentages $P_{3.8}$ of the β_{50} sample results below $\beta_{EC,50}$ = 3.8

Constitutive material	Percentages $P_{3.8} < \beta_{EC,50} = 3.8$		
	Floor beams	Roof beams	Columns
Concrete	6	7	≈ 0
Steel	15	37	3
Composite	1	2	≈ 0
Timber	≈ 0	≈ 0	≈ 0

3. Annual reliability

3.1 Procedure, assumptions and models

For the establishment of the annual reliability level (T_{ref} = 1 year), the (strictly designed) structural members described under Section 2.2 are adopted. Also, the member failure modes and the associated LSF's are the same as employed in the determination of the lifetime reliabilities (Section 2.3). For the variables that do not change with time, like strength if deterioration is avoided or self-weight, the probabilistic models mentioned in Section 2.4 are applied. The parameters describing the time variant variables, i.e. the imposed loads on building floors and the snow loads on roofs, need to be converted for consistency with the one-year reference period.

A basic reference period of 5 to 10 years is often considered for imposed loads, over which they may be assumed to be correlated, see e.g. [16, 29]. Under this assumption, in the present study the annual reliabilities are based, as a simplification, on the 5-year parameters for imposed loads. Taking basis in the model developed in the present study, these parameters, $S_{q,5}$ (= $S_{q,1}$) and $V_{q,5}$ (= $V_{q,1}$) are derived from their respective 50-year counterparts, in compliance with the known properties of the Gumbel [GUM] distribution [1]. The results are summarised in Table 3.

Table 3. Probabilistic models for variable loads (T_{ref} = 1 year)

Variable load model	Distribution function	Ratio S_q	Coefficient of variation V_q
Imposed	GUM	0.36*	0.49*
Snow		0.09	2.25

* Parameters corresponding to a 5-year basic reference period, over which imposed loads are assumed to be fully correlated.

For the snow loads on roofs, a coefficient of variation of $V_{q,1}$ = 2,25 is derived for a 1-year reference period [30] from the same data base used to deduce the probabilistic model associated with T_{ref} = 50 years [26], representing climatic conditions of the Iberian Peninsula, with the exception of Andalusia. The obtained parameters (Table 3) may at first seem unrealistic. However, they are not only consistent with those identified for a reference period of 50 years (Section 2.4) but are also plausible. The low mean value and the very high coefficient of variation reflect the situation – typical for large areas of the Iberian Peninsula, except at higher altitudes that are not covered by the snow model– that the winters are not snow-free, but the snow depths are almost negligible for most structures, with extremely rare weather events with heavy snowfall also occurring [31]. This feature should be considered when comparing the results of this study with those of studies using probabilistic snow load models for other climate regions [7, 11, 12, 15].

3.2 Results

The implicitly required annual reliability index, β_1 for each representative failure mode j depends on the load ratio ν, similar to what is observed for lifetime reliability. Fig. 3 provides a summary in terms of box plots of the obtained β_1 sample results, using the same symbols as in Fig. 2. Lowest average reliabilities (crosses) are found for steel members, whereas the highest mean values correspond to composite members. As for the 50-year reference period (Fig. 2), the mean values for the beams, $\mu_{\beta,FB}$ and $\mu_{\beta,RB}$, appear to be far lower than the respective mean values for the columns, $\mu_{\beta,C}$. Also the scatter is of similar order as observed for the 50-year period.

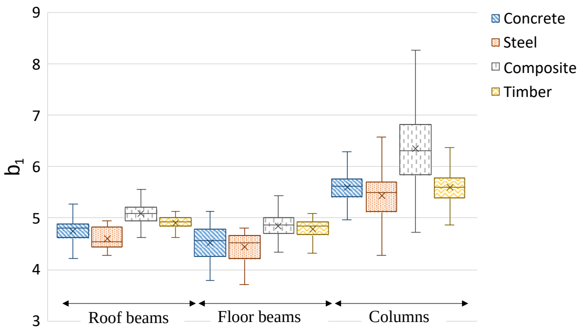

Figure 3. Box plots for β_1 samples corresponding to specific structural member types of different constitutive materials; T_{ref} = 1 year

Table 4 shows that important percentages of the β_1 sample results for the roof and floor beams fall below the target value of the reliability index recommended by the Eurocodes for consequence class CC2, $\beta_{EC,1}$ = 4.7, ranging from $P_{4.7}$ = 2% for composite roof beams to $P_{4.7}$ = 84% for steel floor beams (see the discussion in

Section 3.3). In case of the columns, this percentage appears to be significant for steel only ($P_{4.7}$ = 7%).

Table 4. Percentages $P_{4.7}$ of the β_1 sample results below $\beta_{EC,1}$ = 4.7

Constitutive material	Percentages $P_{4.7} < \beta_{EC,1} = 4.7$		
	Floor beams	Roof beams	Columns
Concrete	71	45	≈ 0
Steel	84	69	7
Composite	24	2	1
Timber	34	11	≈ 0

3.3 Annual versus lifetime reliability

The percentages $P_{4.7}$ of the implicitly required annual reliability indices β_1 falling below the Eurocode target $\beta_{EC,1}$ = 4.7 (Table 4) largely exceed the percentages $P_{3.8}$ of the β_{50} values below the corresponding target $\beta_{EC,50}$ = 3.8 (Table 2). This points to a certain incompatibility between $\beta_{EC,1}$ = 4.7 and $\beta_{EC,50}$ = 3.8. To further substantiate this observation, the values of the β_1 distribution corresponding to the percentages $P_{3.8}$ in Table 2 are being determined. The results, given in Table 5, suggest that compatibility with $\beta_{EC,50}$ = 3.8 would be achieved for $\beta_{EC,1}$-values ranging from 3.9 to 4.3 for the floor beams, from 4.3 to 4.7 for the roof beams and between 4.0 and 4.5 for the columns. This finding corroborates the conservatism of $\beta_{EC,1}$ = 4.7, which, as stressed in Section 1, is based on the assumption of mutually independent failure events in subsequent years.

Another way to assess the influence of correlated failure events on the relationship between lifetime and annual failure probabilities, respectively $p_{f,50}$ and $p_{f,1}$, is to analyze the ratio between both, i.e. $p_{f,50}/p_{f,1}$. For mutually independent failure events in subsequent years, $p_{f,50}/p_{f,1}$ would be characterized by a value of 50, whereas for fully correlated events, $p_{f,50}/p_{f,1}$ = 1.0. Keeping this in mind, as an example Fig. 4 shows ratio $p_{f,50}/p_{f,1}$ as a function of load ratio ν, for the steel floor and roof beams subjected to bending. Due to the progressively rising influence of the time-dependent variable loads with increasing ν, ratio $p_{f,50}/p_{f,1}$ for both beam types increases as does ν, before converging to a maximum value.

Table 5. Annual reliability indices β_1 compatible with $\beta_{EC,50}$ = 3.8 (based on percentages $P_{3.8}$ in Table 2)

Constitutive material	β_1 compatible with $\beta_{EC,50}$ = 3.8		
	Floor beams	Roof beams	Columns
Concrete	4.0	4.3	4.0
Steel	4.2	4.5	4.5
Composite	4.3	4.7	4.5
Timber	3.9	4.4	4.5

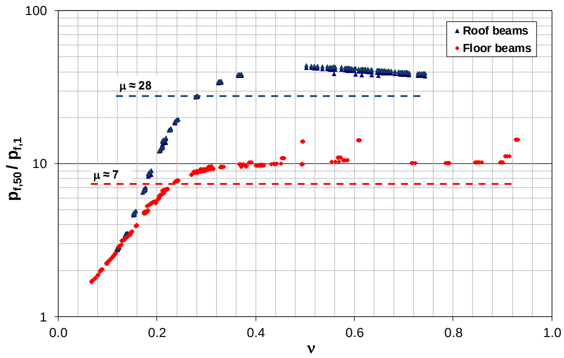

Figure 4. Ratio of 50 to 1-year failure probabilities for steel floor (red) and roof (blue) beams vs. load ratio between the characteristic values of variable and total loads, ν

Reasonably, this maximum is far higher for the roof beams, which are subjected to uncorrelated (over successive years) snow loads, whereas imposed loads on building floors are assumed to be fully correlated over a basic reference period of 5 years (Section 3.1). This is also reflected in the comparatively lower average ratios $p_{f,50}/p_{f,1}$ for the floor beams, of about 7, in comparison to the mean value of about 28 obtained for the roof beams (Fig. 4). Table 6 shows similar average values for the other materials analyzed. The average ratios $p_{f,50}/p_{f,1}$ of the order of 7 to 9 found for the floor beams correspond to an average equivalency between $\beta_{EC,50}$ = 3.8 and β_1

= 4.2 to 4.3. The mean ratios $p_{f,50}/p_{f,1}$ of 21 to 36 for the roof beams are equivalent to slightly higher β_1, of about 4.5 to 4.6. Unsurprisingly, these results are in broad agreement with those in Table 5.

Table 6. Mean values of the ratios of 50 to 1-year probabilities of bending moment induced failure for floor and roof beams

Constitutive material	Floor beams	Roof beams
Steel	**7**	**28**
Concrete	**7**	**30**
Composite	**9**	**36**
Timber	**7**	**21**

4. Conclusions

The paper addresses a comprehensive analysis of the reliability level implicitly required by the current version of the Eurocodes. The study differentiates from previous investigations pursuing a similar goal [7, 11, 12] in the consideration of an extraordinarily large set of realistic design situations. In addition, the parameters of the probabilistic models representing the uncertainties associated with the load and resistance variables not only reflect the physical properties of these variables but are also compatible with the corresponding design values as established by the Eurocode partial factor method (PFM) and the design value method (DVM), respectively.

Using these models, the lifetime reliability level (T_{ref} = 50 years), associated with a strict Eurocode design ($E_d = R_d$) is quantified in the paper. The reliability analysis reveals a large scatter across different member types, associated failure modes, constitutive materials and load ratios ν between variable and total loads, although the obtained average reliability indices generally exceed the Eurocode target value, $\beta_{EC,50}$ = 3.8.

Following the establishment of the annual reliabilities β_1 of the same members previously analyzed for T_{ref} = 50 years, the results are then used to infer equivalencies between the implicit reliability levels for a reference period of 1 and 50 years, respectively. The outcome corroborates the conservatism of the Eurocode requirement for T_{ref} = 1 year and medium (normal) failure consequences, $\beta_{EC,1}$ = 4.7. Reasonable values for $\beta_{EC,1}$ in line with $\beta_{EC,50}$ = 3.8 should oscillate around 4.1 for members under imposed loads and around 4.5 for failure governed by snow loads. These values reflect average values over all materials analyzed.

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