# Target reliabilities for key members in building structures

## *Requisitos de fiabilidad para elementos clave en estructuras de edificación*

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**ABSTRACT**

Modern codes for structural reliability establish human safety criteria based on the Life Quality Index (LQI) as a constraint to economic optimization principles. However, the use of the LQI may yield higher life safety risks compared to current design approaches. Where this may be the case, as in the design or assessment of key members where failure entails potentially large life safety risks, it is advisable to specify reliability levels in line with accepted practices. Annual target reliabilities for such members in buildings are therefore inferred from implicitly accepted life safety risks associated with current Eurocode practices, with a focus on the correlation between failure events in subsequent years of the structural working life.

**RESUMEN**

Las normas modernas sobre fiabilidad estructural establecen criterios de seguridad de las personas, basados en el Índice de Calidad de Vida (LQI), como una restricción a los principios de optimización económica. Sin embargo, el uso del LQI puede generar mayores riesgos para las personas que las actuales reglas de dimensionado de las estructuras. Eso puede ocurrir, por ejemplo, en el dimensionado o la evaluación de elementos clave, cuyo fallo implica riesgos para las personas potencialmente grandes. Por ello se infieren requisitos de fiabilidad acordes con los riesgos implícitamente aceptados por los actuales Eurocódigos, considerando la correlación entre eventos de fallo en sucesivos años del período de servicio.

**KEYWORDS:** buildings, key members, acceptance criteria, life safety, reliability, design codes.

**PALABRAS CLAVE:** edificios, elementos clave, criterio de aceptación, seguridad personal, fiabilidad, normas.

## 1. Introduction

Most modern structural codes and standards include probabilistic reliability requirements for the design of new or the assessment of existing structures. Formulated in terms of target reliability indices, $\beta_t$, in turn related to admissible failure probabilities, such (nominal) requirements may be used for decision making based on explicit probabilistic analysis, e.g. in unusual design situations with regard to the involved uncertainties or potential failure consequences. Moreover, they serve as a basis for calibration of semi-probabilistic safety rules for addressing usual design situations in routine engineering practice [1].

For a specified reference period $T_{ref}$, many structural codes, e.g. the Eurocodes [2, 3], use exclusively the expected failure consequences to explicitly differentiate target reliability levels.

The JCSS Probabilistic Model Code [4] recognizes the relevant role of other parameters for fixing such levels, among them the relative cost of safety measures to increase reliability or the variability in actions, influences and structural resistance parameters. In compliance with the JCSS recommendations, the international standard ISO 2394 [5] and the fib Model Code 2020 [6] support the use of target reliabilities as given by Eq. (1) for a specified reference period, preferably $T_{ref}$ = 1 year, which expresses that any economically optimal solution, associated with a target value $\beta_{t,eco}$, should be subject to the constraint $\beta_{t,LR}$ ensuring that life safety risks are within the bounds of acceptability.

$$\beta_t = \max(\beta_{t,eco}; \beta_{t,LR}) \tag{1}$$

Different generic frameworks for probabilistic optimization have been proposed in the literature [7 - 9]. Tentative values for $\beta_{t,eco}$ based on the fundamental work by Rackwitz [10], are provided in the above mentioned codes [4 - 6] and reproduced in Table 1 (see footnote with respect to definition of consequence classes). As a default case for design of new structures, the MC 2020 code [6] identifies consequence class CC2 and normal relative cost of safety measure (class B), associated with medium variabilities of the yearly extreme values of the total loads and resistances and a service life duration of 30 to 70 years. For the assessment of existing structures, lower target reliabilities than for design are justified on the grounds of comparatively higher costs for achieving specific safety levels [11 - 15]. For the minimum target level, below which an existing structure should be upgraded, large relative cost of safety measures (class C) may be assumed according to [6].

Following the recommendations in [4 - 6], the target reliability of structures related to life safety risks, $\beta_{t,LR}$, should be adopted in compliance with the marginal lifesaving cost principle (MLSC). On the basis of this approach, implemented in practice by the Life Quality Index (LQI) [17], the conformity of structural performance can be assessed with respect to the societal preferences for investments into life safety [9, 18, 19]. Tentative values for LQI-based target reliabilities, provided in ISO 2394 [5], grounded on the studies [9, 18], suggest that the application of the MLSC principle may yield higher risk levels compared to current practice (e.g. design according to Eurocodes). Among other reasons, this might be attributable to the observation that assumptions and choices on which the current design codes are based tend to be conservative.

**Table 1. Tentative values for annual target reliabilities $\beta_{t,eco}$ for Ultimate Limit States based on economic optimization [4 - 6].**

| Relative cost of safety measure | Consequence Class (CC)* | | |
|---|---|---|---|
| | CC1 | CC2 | CC3 |
| Large (A) | 3.1 | 3.3 | 3.7 |
| Normal (B) | 3.7 | 4.2 | 4.4 |
| Small (C) | 4.2 | 4.4 | 4.7 |

* The definition of consequence classes (CC) in, respectively, the JCSS Model Code [4], ISO 2394 [5] and MC 2020 [6] obeys different criteria. The latter [6] distinguishes three CC's, based, among other criteria, on the number of persons at risk at the level of the whole structure aiming to facilitate their use in practice, following recommendations in [16].

In the opinion of social scientists, it is unlikely that the public would accept higher failure rates than those associated with current practice [20, 21], even if they were based on rational acceptance criteria such as the MLSC. Where structural failure is expected to result in significant human losses, it is therefore advisable that life safety risk levels are in line with those implicitly accepted in current practice. This is particularly the case in connection with the possible failure of so-called key members. The failure of such members may induce progressive collapse, which would normally result in damage and associated risk to persons, that exceeds a tolerable limit. Therefore, when designing or assessing key members for identified hazard scenarios, persistent or accidental, an appropriate target reliability level should be provided.

In a former study [21], an operational criterion suitable for practical applications was proposed, in which annual reliability requirements $\beta_{t,LR}$ for failure modes of individual structural members were inferred from acceptable life safety risks associated with building structures strictly compliant with current Eurocode design practices and represented depending on the expected failure consequences to the building users. Subsequently, these requirements were further developed [22], what included an improvement of the models for predicting loss of life due to collapse [23], while in parallel their utility for the layout of key members was substantiated [24, 25].

It was further acknowledged that since the previous studies [21, 22] assumed statistically independent failure events in subsequent years of the considered reference period of 50 years, the annual reliability index targets $\beta_{t,LR}$ may be lowered. In view of time-invariant influences (e.g., strength variables in the absence of deterioration, self-weight, etc.) on the failure probabilities, failure events are indeed partially correlated over the years of the planned service life of a structure [26 - 29]. The simplifying assumption of statistical independence therefore leads to an underestimation of the implicitly acceptable annual risks and, with the approach chosen in the previous studies, to overly conservative (i.e., comparatively high) estimates of the required annual reliability. Taking correlation into account is therefore essential for a consistent comparison of the proposed key member requirements, $\beta_{t,LR}$, to the outcome of monetary optimization exercises, $\beta_{t,eco}$ (Table 1), as per Eq. (1). The present paper focusses on these developments.

Section 2 provides a summary of the procedure and the main assumptions for the establishment of implicitly acceptable life safety risks associated with building structures. Afterwards, section 3 addresses the recomputation of these risks under consideration of correlated failure events in subsequent years of the structural service life. Based on the results obtained, the reliability requirements derived from the earlier studies cited [21, 22] are updated in section 4 and discussed on the background of a comparison to target values established in current codes and standards. The paper concludes with a summary of the most relevant findings in section 5.

## 2. Inferring implicitly acceptable risks

### *2.1 Approach*

In daily structural design practice, risks are not explicitly quantified. The question of their acceptability is judged on the basis of prescriptive, codified design rules. Developed on the grounds of "calibration to a long experience of building tradition" [2], these design rules result in a level of structural performance that reflects what is often referred to as Best Current Practice, BCP. Risk levels associated with structures designed and built in accordance with these rules are perceived as reasonable [20] since otherwise the public would demand far-reaching changes.

Based on this postulate, the present study defines acceptable risk as the risk associated with structures that are strictly compliant with the safety requirements set out in existing structural standards, in particular the Eurocodes [2]. Acceptable risks are therefore directly linked to the reliability level implicitly required by the Eurocodes, which might differ fundamentally from the nominal target ceilings established in these codes [28, 30, 31].

### *2.2 Procedure*

The procedure sketched in Fig. 1 was established to determine implicitly acceptable risks to persons associated with building structures. Initially, the scope of the study was to be determined: types of structures, types of failure, etc.

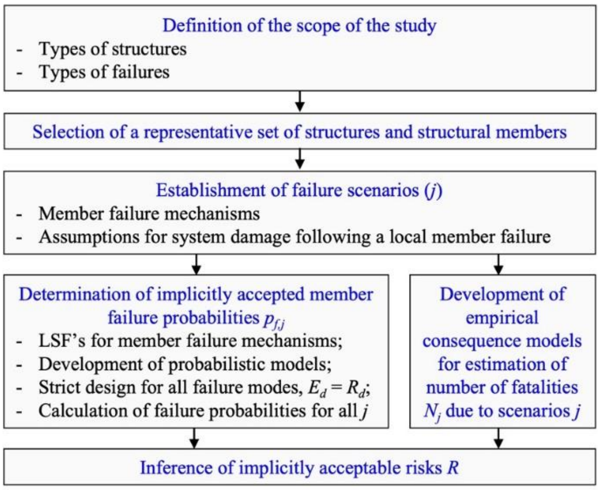


**Figure 1. Procedure for inference of implicitly acceptable risks.**

Then a series of hypothetical but realistic structures were selected, which consisted of standard structural elements. Varying the parameters with the greatest effect on design, within reasonable ranges that make it possible to cover the vast majority of the cases encountered in practice, yielded a representative set of such structures.

The failure scenarios, $j$, associated with the selected structures were then established. This involved in first place identifying the most representative failure mechanisms for the constitutive structural members. In addition, assumptions had to be established to describe damage to the structural system following a potential failure of each of these members.

Establishing the occurrence probabilities of the failure scenarios, $p_{f,j}$ , implied the formulation of the limit state functions (LSF) based on the member failure mechanisms identified. For the variables involved in these LSF, probabilistic models were developed that represent the state of uncertainty associated with the rules specified in the Eurocodes [31]. All structural members were designed strictly to Eurocode specifications, 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 LSF and the previously developed probabilistic models, applying reliability methods [32] the implicitly acceptable failure probabilities, $p_{f,j}$, for each representative failure mechanism, associated with a specific reference period, $T_{ref}$, were determined for all the members designed in such a way. The corresponding reliability indices, $\beta_j$, represent the degree of reliability implicitly required by the Eurocodes.

To estimate the consequences to persons given a specific failure scenario, $j$, an empirical model was established that predicts the expected number of fatalities, $N_j$ , in terms of the system damage extensions, described by the building area affected by the collapse, $A_{col}$, and the consequence class, CC, established as a function of the number of persons at risk, $Ocu_{col}$, per unit of $A_{col}$ [23]. The consequence model is structural design and execution-independent, i.e. it is irrelevant to the estimation of the consequences whether the failure is due, for example, to a heavy overload or some error in design or execution. In the same vein, it is also irrelevant to the estimation of the failure consequences according to which design standard (current or former) the failed structures have been designed.

Structure-related notional risks, $R$, for each of the structures constituting the representative set established were then obtained from Eq. (2) on the grounds of the probability of occurrence, $p_{f,j}$, of each of the, $n_{sc}$, failure scenarios, $j$, associated with the structure and its consequences in case of occurrence in terms of the expected number of fatalities, $N_j$.

$$R = \sum_{j=1}^{n_{sc}} (p_{f,j} \cdot N_j) \qquad (2)$$

The procedure adopted was applied to building structures, considering exceedance of ultimate limit states in connection with, respectively, persistent or transient [21, 22] and accidental [33] design situations. The present contribution only applies to persistent situations. Fairly conventional structural systems have been considered. For the assumptions made regarding the structural systems, members, failure modes, the calculation of failure probabilities and the risks associated with the selected structures, reference is made to the mentioned previous studies [21, 22].

## 3. Correlation of failure events

### *3.1 Generalities*

Equation (3), which relates the annual risk $R_1$, e.g. the risk to persons associated with building structures, to the risk for a reference period of 50 years, $R_{50}$, by dividing the latter by the corresponding reference period, $T_{ref.}$ = 50 years, assumes statistically independent failure events in subsequent years of $T_{ref.}$.

$$R_1 = R_{50}/50 \tag{3}$$

However, due to the unavoidable correlation of such events (section 1), this assumption leads to an underestimation of the implicitly acceptable annual risks $R_1$ and, with the approach chosen in this study, to overly conservative estimates (i.e., comparatively high values) of the required annual reliability levels. With the aim to eliminate this simplifying assumption, the annual risks $R_1$ were recomputed by feeding Eq. (2) directly with annual failure probabilities, $p_{f,1}$. Section 3.2 addresses the determination of these probabilities. The subsequently obtained risks $R_1$ are presented in section 3.3.

### *3.2 Annual reliability level implicitly required by the Eurocodes*

The annual failure probabilities $p_{f,1}$ were estimated by means of Eq. (4), suggested in the study [29],

$$\Phi(\beta_{nk}) = \Phi(\beta_1)^{n/k} \tag{4}$$

in which Φ(…) denotes the cumulative distribution function of the standardised normal distribution, $\beta_{nk}$ the reliability index associated with reference period $n$ (= $T_{ref}$) and the so-called independence interval $k \leq n$ ($n$ and $k$ in years). In the present study, $\beta_{nk}$ represents the 50-year reliability indices $\beta_{50}$, corresponding to the previously determined 50-year failure probabilities, $p_{f,50}$ [21, 22].

The independence interval, $k$, corresponds to the mean time period for which the failures may be assumed to be mutually independent in subsequent years and depends primarily on the type of structural member and the dominant actions to which they are subjected [29]. In the present study, $k$ is estimated dependent upon ratio $\nu$ of the variable loads to the total loads acting on the structural member in question, formulated in terms of characteristic values. A power-law function was assumed to represent this relationship,

$$k = a \cdot \nu^b \tag{5}$$

where constants $a$ and $b$ are inferred from the following boundary conditions:

- If $\nu = \nu_{max}$ (influence of permanent loads negligible), $k = k_{min}$;
- If $\nu = \nu_{min}$ (influence of variable loads negligible), $k = n$ ($n$ = 50 years).

The minimum and maximum load ratios, respectively, $\nu_{min}$ and $\nu_{max}$, vary depending on the structural member type (roof beams, floor beams, columns) and its constituent material [31]. However, for sake of simplicity, the extreme values of the entire set of structural members analyzed in the study, respectively $\nu_{min} \approx 0.03$ and $\nu_{max} \approx 0.93$, were adopted here for the computation of the independence interval $k$ of all members.

The minimum independence interval $k_{min}$ depends on the type of variable load dominating the failure events. For climatic actions, the failures in subsequent years are assumed to be independent and hence $k_{min} \approx 1$. In case of imposed loads on building floors, $k_{min} = 5$ is assumed in accordance with the mean occurrence rate, $\lambda$, of sustained load changes that may be considered, e.g. in office buildings [34, 35]. Dependant on the building use, the choice of higher $k_{min}$, e.g. $k_{min}$ = 10, might be more reasonable for imposed loads. In any case, the impact of this choice on the result is low.

In the present study, $k_{min}$ = 1 applies to the roof beams, which are subjected to snow loads, and the columns sustaining these beams, whereas $k_{min}$ = 5 applies to the floor beams. For the columns that are submitted to both snow and imposed loads, $k_{min}$ is linearly interpolated between 1 and 5 depending on the respective load intensity. Table 2 summarizes the constants $a$ and $b$ (Eq. (5)) obtained for these $k_{min}$.

**Table 2. Assumed minimum independence interval $k_{min}$ and resulting constants $a$ and $b$ for different member types.**

| Member type | $k_{min}$ | a | b |
|---|---|---|---|
| Roof beams | 1 | 0.922 | -1.157 |
| Floor beams | 5 | 4.765 | -0.681 |
| Columns | ≥ 1; < 5 | ≥ 0.922; < 4.765 | ≥ -1.157; < -0.681 |

Based on these assumptions, the independence intervals $k$ for different RC members calculated from Eq. (5) are shown in Fig. 2. Equivalent results were also obtained for the other materials analysed but are not presented here. It is shown that in comparison to the large variations across load ratio $\nu$, the differences between the $k$'s for the different member types are small.

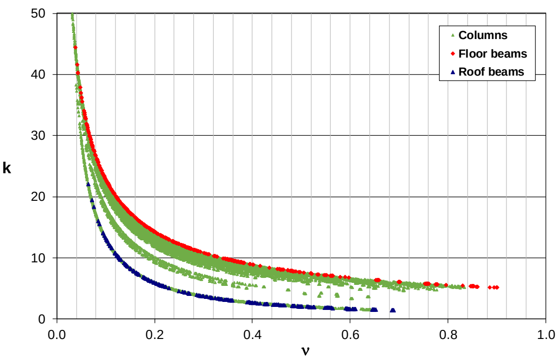


**Figure 2. Independence interval $k$ for different RC members as a function of load ratio $\nu$.**

When comparing the reliability indices inferred from Eqs. (4) and (5), $\beta_{1,Eqs.(4,5)}$, to the previously established 50 year indices [31], $\beta_{50}$, used as a basis for computation of $\beta_{1,Eqs.(4,5)}$, it is noticeable that, irrespective of the member type and failure mode under consideration, the differences between the annual reliability indices $\beta_{1,Eqs.(4,5)}$ and the 50-year values $\beta_{50}$ increase with rising influence of the time-dependent variable loads (i.e. increasing $\nu$), as was to be expected. For small $\nu$, the differences appear to be practically negligible in most cases due to the strong correlation of failure events over subsequent years in consequence of a significant contribution of permanent loads. Similar findings are obtained for members of all construction materials.

In order to validate the results obtained with Eqs. (4) and (5), $\beta_{1,Eqs.(4,5)}$, the annual failure probabilities were also determined based on a reliability (FORM) analysis with appropriate 1-y probabilistic models for the variable loads (imposed and snow loads), maintaining all other models as established for the 50-year analysis. In general, a good approximation between the obtained $\beta_1$ and the $\beta_{1,Eqs.(4,5)}$ is observed for most design situations considered. Table 3, which compares the mean values $\mu$ of specific sets of $\beta_1$ and $\beta_{1,Eqs.(4,5)}$, shows that, on average, the computations based on Eqs. (4) and (5) underestimate the FORM results in most cases, with deviations of not more than 4%, however. Moreover, Table 3 reveals for most situations slightly lower coefficients of variation, $\nu$, for the obtained $\beta_{1,Eqs.(4,5)}$, although when compared in absolute terms, the difference is insignificant.

**Table 3. Comparison of mean values, $\mu$), and coefficients of variation, $v$, of specific sets of annual reliability indices obtained by, respectively, FORM [32], $\beta_1$, and Eqs.(4) and (5), $\beta_{1,Eq.(4,\,5)}$.**

| Member type | Material | Design situations | Mean values | | | Coefficients of variation | | |
|---|---|---|---|---|---|---|---|---|
| | | | $\mu_{\beta 1}$ | $\mu_{\beta 1,Eq.(4,\,5)}$ | $\mu_{\beta 1}/\mu_{\beta 1,Eq.(4,\,5)}$ | $v_{\beta 1}$ | $v_{\beta 1,Eq.(4,\,5)}$ | $v_{\beta 1}/v_{\beta 1,Eq.(4,\,5)}$ |
| Roof beams | Concrete | 720 | 4.73 | 4.74 | 1.00 | 0.06 | 0.04 | 1.37 |
| | Steel | 288 | 4.60 | 4.55 | 1.01 | 0.04 | 0.04 | 1.19 |
| | Composite | 960 | 5.09 | 4.90 | 1.04 | 0.04 | 0.04 | 0.99 |
| | Timber | 160 | 4.90 | 4.95 | 0.99 | 0.03 | 0.02 | 1.57 |
| Floor beams | Concrete | 1350 | 4.51 | 4.47 | 1.01 | 0.07 | 0.06 | 1.24 |
| | Steel | 540 | 4.44 | 4.35 | 1.02 | 0.06 | 0.05 | 1.19 |
| | Composite | 1800 | 4.85 | 4.73 | 1.03 | 0.04 | 0.04 | 1.12 |
| | Timber | 300 | 4.78 | 4.73 | 1.01 | 0.04 | 0.03 | 1.26 |
| Columns | Concrete | 22320 | 5.59 | 5.50 | 1.02 | 0.05 | 0.04 | 1.17 |
| | Steel | 12168 | 5.43 | 5.26 | 1.03 | 0.09 | 0.08 | 1.07 |
| | Composite | 76464 | 6.35 | 6.14 | 1.03 | 0.11 | 0.11 | 1.02 |
| | Timber | 5280 | 5.60 | 5.38 | 1.04 | 0.05 | 0.04 | 1.26 |

### *3.3 Implicitly acceptable risks*

The annual failure probabilities $p_{f,1,Eqs.(4,5)}$ converted by means of Eqs. (4) and (5) from the 50-year probabilities $p_{f,50}$ representing the implicit reliability level associated with the Eurocode rules were employed to recompute (with reference to the previous studies [21, 22], as explained in section 1), based on Eq. (2), the implicitly acceptable annual risks $R_1$ associated with each of the structures constituting the defined representative set [21, 22].

The mean and upper fractile (98%) values of risks $R_1$ (based on $p_{f,1,Eqs.(4,5)}$) associated with the 10,872 building structures classified under CC2 [21, 22], normalised for the total building areas $A$ or the total number of failure scenarios $n_{sc}$ per building, respectively $R/A$ or $R/n_{sc}$, are compared in Table 4 to the results obtained in [22], where statistically independent failure events in subsequent years were assumed (Eq. (3)). As expressed by ratio $C/I$ (last column in Table 4), risk levels established under this assumption appear to be roughly one order of magnitude lower than compared to the results obtained here, accounting for correlation between failure events. The same conclusion is obtained for the analysed 8,136 CC3 structures [21, 22].

**Table 4. Mean and upper fractile (98%) values of normalised, implicitly acceptable risks $R_1$ (fatalities per year) associated with building structures, CC2, established under assumptions of independent ($I$) and correlated ($C$) failure events in subsequent years.**

| Risks $R_1$ normalised for | Value | Notation | Failure events in subsequent years | | |
|---|---|---|---|---|---|
| | | | Independent, $I$ [22] | Correlated, $C$ | Ratio, $C/I$ |
| building area $A$ | Mean | $(R_1/A)_\mu$ | $1\cdot10^{-7}$ | $1\cdot10^{-6}$ | ≈ 13 |
| | 98% fract. | $(R_1/A)_{k,98}$ | $6\cdot10^{-7}$ | $9\cdot10^{-6}$ | ≈ 14 |
| nº of failure scenarios $n_{sc}$ | Mean | $(R_1/n_{sc})_\mu$ | $2\cdot10^{-6}$ | $4\cdot10^{-5}$ | ≈ 19 |
| | 98% fract. | $(R_1/n_{sc})_{k,98}$ | $2\cdot10^{-5}$ | $3\cdot10^{-4}$ | ≈ 20 |

## 4. Reliability requirements

The results summarized in the previous section, particularly in Table 4, were used to derive admissible annual fatality rates, $r_{LR,adm}$, and acceptable frequencies of failure events with $n \geq 1$ fatalities, $F(1)$, following the procedure established in [21, 22]. On this basis, admissible annual failure probabilities, $p_{ft,LR}$, were obtained by merging the former two criteria into a single decision criterion according to [22]. After conversion, the result was expressed in terms of the target reliability index $\beta_{t,LR}$, according to Eq. (6), where $A_{col}$ represents the area affected by the collapse and $C$ a consequence class-dependant constant.

$$\beta_{t,LR} = C - 2 \cdot A_{col}{}^{-0.1} \qquad (6)$$

Numerical values for constant $C$ are given in Table 5 as a function of the consequence class CC, depending on the number of persons at risk per unit of area affected by the collapse ($Ocu_{col}/A_{col}$) [23]. Constants $C_{design}$ are based on the expected values of the normalised implicitly acceptable risks to persons (Table 4) and are recommended for the design of new structures [21, 22]. The obtained difference between CC2 and CC3 situations, $\Delta C = 0.5$, matches the $\Delta\beta$ between the annual target reliability index associated with subsequent consequence classes defined in the Eurocodes [2, 3] and complies with recommendations in [15]. Although the associated risk levels are not addressed in the present study, a criterion for consequence class CC1 is suggested based on this $\Delta\beta$ for sake of practical applications (Table 5).
The computations based on the 98% fractile values of the implicitly acceptable risks (Table 4) are suggested for use in the context of assessment of existing structures. The corresponding constants $C_{assessment}$ were found to be between $\Delta C$ = 0.42 and 0.52 below $C_{design}$, depending on the CC. For sake of simplicity, $C_{assessment}$ was rounded to ($C_{design}$ - 0.5) for all CC's (Table 5). Lower requirements for the assessment of existing structures are justified in order to account for the comparatively higher investments needed for reducing structure-related risks (section 1), in the present context related to life-safety.

**Table 5. Recommended values for constant *C* depending on the application context and the consequence class.**

| Consequence class | $Ocu_{col}/A_{col}$ [23] [P/m²] | $C_{design}$ | $C_{assessment}$ |
|---|---|---|---|
| CC1 | < 1/100 | 4.75 | 4.25 |
| CC2 | ≥ 1/100 and ≤ 1/10 | 5.25 | 4.75 |
| CC3 | > 1/10 | 5.75 | 5.25 |

## 5. Conclusions

The paper addresses the development of human safety requirements for structural design or assessment, building on previous studies in which annual target reliabilities $\beta_{t,LR}$ for failure modes were inferred from implicitly acceptable life safety risks associated with a representative set of building structures strictly compliant with current Eurocode design practices. One of the principal limitations of these $\beta_{t,LR}$ was that they were grounded on overly conservative annual risk levels, deduced from lifetime values (50 years) under the assumption of statistically independent failure events in subsequent years. With the aim to eliminate this strongly simplifying assumption, valid solely for situations governed by time-dependent influences, the annual risks associated with the selected building structures were recomputed accounting for correlation between failure events. The results revealed acceptable risk levels of about one order of magnitude higher than those obtained assuming independent failure events.

In consequence of this finding, the subsequently updated requirements $\beta_{t,LR}$ were found to be substantially less stringent compared to the acceptance criteria deduced in the previous studies. These requirements complement the LQI approach as a constraint on decisions based on economic optimization principles, $\beta_{t,eco}$, as recommended by international standard ISO 2394 [5] and the fib Model Code 2020 [6]. In particular, this criterion appears to be meaningful in situations where higher acceptable life safety risks than under current structural design practices would not be deemed acceptable by decision makers. A principal application field is the design or the assessment of key members, the failure of which entails potentially large life safety risks.

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