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Quality Control Method for the Service Life and Reliability of Concrete Structures

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Abstract: In the past few years, there has been an increasing societal and industrial demand for the reliable assessment and design of structural systems with service-life criteria of at least several decades. The life cycle characterisation of engineering structures in terms of an anticipated service life remains a significant aspect of sustainability in the construction industry. This requires special attention to the definition of structural performance under various actions, and to the implemented engineering materials and methods as well as to the inverse identification and monitoring of structural conditions. Subsequently, the focus remains on the development of a holistic performance-based design approach for new and existing structures and infrastructures. This paper presents the fundamental reliability concepts of performance-based design, with a focus on lifetime assessment. Case studies from actual structural components' design are used to verify the proposed methodology and indicate the significance of quality assurance in the lifetime assessment of engineering structures. We also confirmed that reliability and quality assurance criteria are strongly connected. Therefore, a methodology for quality-based service life assessment is presented and elaborated in the case studies.

Keywords: lifetime assessment; carbonation; chloride ingress; flexural capacity; fastenings; quality-based structural reliability; quality service time indicator; construction-defect-induced degradation

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1. Introduction

The inclination of the engineering society towards sustainable industries is evident in many facets of modern life and business. Current research focuses on sustainability as a fundamental requirement, based upon a holistic treatment of societal needs and impacts, life-cycle cost and environmental impact. The treatment of the existing building stock competes with the building of new projects in terms of resource volumes, with an increasing trend. This dictates the necessity to introduce codes and standards that coherently address new and existing structures. The safety of users, third parties and the environment; serviceability and durability; and other specific performance criteria, such as robustness and resilience, form the focus of upcoming design standards. Simultaneously, inherent uncertainty of materials and the environment, along with rapid technological advancements and innovations, create an increasing need for decisions based on reliability and holistic design concepts. State-of-the-art performance-based design is required to (a) address the structural safety and reliability of structures by use of analyses with various levels of complexity and accuracy, (b) assess the structure's condition during service life with inspection, testing and monitoring and (c) define necessary intervention practices. In Figure 1, the Ishikawa causal diagram with the analysis of causes and factors of material degradation is illustrated.

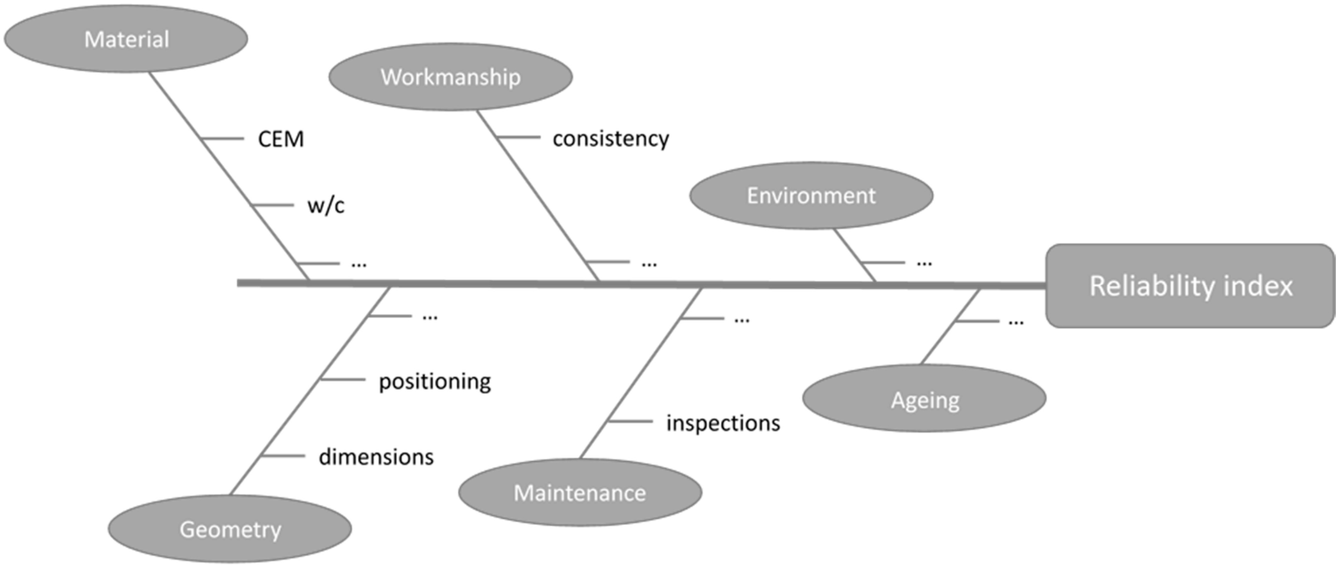


Figure 1. Ishikawa causal diagram with the analysis of causes and factors of material degradation.

During the implementation of life-cycle-management strategies, these intervention actions must be selected in order to maintain assets at a desired performance level. Therefore, specific performance indicators must be established at a technical component level. These indicators can be qualitative or quantitative, and they can be obtained during principal inspections through visual examination, a non-destructive test or a temporary or permanent monitoring system. The indicators derived compared with target performance indicators allow for the evaluation of adherence to the owner- or user-defined quality requirements. In this framework, quality is understood as fitness for purpose, or the degree to which a set of inherent characteristics of a product or service fulfils its requirements. The quality requirements are observed through a project-specific quality control (QC) plan. The QC plan specifies all activities and tools needed to ensure the quality requirements related to the investigated performance aspects (safety, serviceability, etc.). It can define the type and quantity of data necessary to assess the performance indicators of the structure or its individual components at the time of data collection and in the future. A QC plan should also include decision models for maintenance actions based on the forecast of performance indicators. In this sense, the QC plan overlaps with the strategic asset management plan (SAMP) as defined by the International Organization for Standardization [1].

The current work will help to fill the gap in the research that can be seen in less elaborated aspects of Eurocodes, namely the connection of reliability and execution classes [2–4]. In addition, this work reinforces the outcomes of the COST TU 1406 project in terms of standardisation of quality specifications. In other words, the present study aims to confirm an interconnection of life-cycle reliability measures with elements of QC and quality assurance by elaborating on the principles of reliability as applied to structural engineering problems. In addition, typical quality criteria with an emphasis on concrete structures are presented. These are translated to reliability principles through a methodology based on the reliability index β. This theoretical approach is then applied to two realistic case studies, being the durability assessment against concrete carbonation and chloride ingress, based on [5]. In these cases, quality is expressed through the concrete specifications in the selection and QC, mainly for the cement type and the water–cement (w/c) ratio. Two supplementary examples are used as case studies for the effects of quality defects on the load bearing reliability of structural components. The first one discusses the reinforcement placement tolerances in a one-way slab. The second example discusses the installation quality of bonded anchors in concrete. These examples are considered by the authors as suitable for demonstration and relevant to practice, without excluding the fact that the basic principles and tools apply to other types of materials, limit state functions and damage processes. The results of these demonstrative analyses are discussed in the final section of the paper, where the main conclusions of the study are highlighted, and practical recommendations are provided in relation to the sensitivity of a structure's reliability in terms of quality issues and the significance of the construction phase on the whole-life performance of a structure.

2. Reliability Concepts

Educated decisions for engineers fundamentally depend on the designed or constructed system's failure probability. Reliability is then understood as the probability that an item or facility will perform its intended function and, as such, as the complement of the probability of failure p_f. In its most basic formulation, reliability r is described by Equation (1):

$$r = 1 - p_f. \tag{1}$$

In general, the design requirement is expressed in the form of a limit state function (LSF), where, e.g., resistance is greater than the load effect, as expressed by Equation (2):

$$G = g(X) = R - S, \tag{2}$$

where x is the set of random variables (time dependent or independent), $G = g(X)$ is the limit state function (with $g(X) < 0$ being the failure state), R is the resistance and S is the load action.

The general solution for the estimation of the failure probability (p_f) can thus be obtained through the integral between $-\infty$ and $+\infty$, as shown in Equation (3).

$$p_f = \int_{-\infty}^{+\infty} f_S(X) \cdot F_R(X)\,\mathrm{dx}, \tag{3}$$

where f_S is the probability density function (PDF) of the limit state function S and F_R is the cumulative distribution function (CDF) of the limit state function R.

To quantify the reliability of the system based on the LSF, the Cornell reliability index β can be estimated based on the individual probability functions of the LSF input, $G = R - S$ [6]. Although the reliability assessment concept is universal, the specific mathematical formulation presented herein requires that R and S follow normal distributions and are statistically independent. The reliability index is defined as

$$\beta = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}}, \tag{4}$$

where μ_R and μ_S are the mean values of resistance and load action, respectively, and σ_R and σ_S are the standard deviations for resistance and load side, respectively.

Then, the failure probability p_f can be estimated as:

$$p_f = \phi(-\beta), \tag{5}$$

where ϕ is the standard normal probability function.

The main characteristics and limitations of the Cornell reliability index concept are that it is limited in the number of variables, it can be applied only to linear limit state functions, it considers only normally distribute variable and there should be no correlation between variables (see [7,8]).

Reliability is typically understood as the property of a system's reliability (multiple-failure modes) but also that of a component's reliability (single-failure mode) at a specific point in time. However, rational concepts need to accommodate life-cycle effects, such as the natural deterioration of materials, extreme events and alterations in human activities and environmental conditions, in long-term considerations [9]. These aspects are particularly governed by uncertainties, where probability-based concepts also find a more efficient implementation. In such cases, the reliability verification of a structure should be related to the structure's service life and failure is associated with two types of events, (a) first peak value beyond an action threshold and (b) the accumulation of propagating damage beyond a resistance threshold. These can be described with time-variant input variables. Consequently, the accepted (target) reliability threshold is also dependent on the lifetime interval under consideration. The service life requirement can be associated with the target reliability based on Equation (6), and assuming that the failure events per year are statistically independent, the values of β for different reference periods can be calculated.

$$\Phi(\beta_n) = [\Phi(\beta_1)]^n, \quad (6)$$

where β_n is the reliability index with respect to n time intervals (years), β_1 is the reliability index with respect to 1 year and n is the reference period (service life) in years.

Target-failure probabilities and associated reliability indices for different types of structures are normally defined in national or international standards, such as Eurocode 0 of the European Committee for Standardization [10,11]. For ultimate limit states, values in the range of $p_f = 10^{-5}$ to 10^{-7} and $\beta = 4.0$ to 5.0 per year are used, while in serviceability limit states, the failure probabilities (and, accordingly, the reliability indices) are decreased by 1 or 2 orders of magnitude. These values also depend on the structure's consequence class (CC) attributed to the design. These three consequence classes can then be associated with three reliability classes (RC) (see Table 1 for the paradigm of Eurocode 0). Furthermore, they are adopted accordingly for reference periods corresponding to the structure's design service life. These target values are originally specified according to experience based on engineering judgement and on calibrations against previous practice. In principle, however, target reliabilities can be designated based on customised criteria, such as cost optimisation, risk-acceptance criteria for the safety of individuals and groups affected by the failure of the structure or the life quality index (LQI) approach. Exemplarily, adjusted reliability levels are proposed to assess existing structures or temporary works.

Table 1. Definition of consequence classes as per [10] and associated target reliabilities and failure probabilities for different reference periods.

CC/RC	Description of Consequence Class (CC)	Reliability (β) and Associated Probability of Failure p_f	
		1 Year	**50 Years**
1 (High)	High consequence for loss of human life. Enormous economic, social or environmental consequences, e.g., grandstands and public buildings.	$\beta = 5.2$, p_f = approx. 10^{-7}	$\beta = 4.3$, p_f = approx. 10^{-5}
2 (Medium)	Medium consequence for loss of human life. Considerable economic, social or environmental consequences, e.g., residential and office buildings.	$\beta = 4.7$, p_f = approx. 10^{-6}	$\beta = 3.8$, p_f = approx. 10^{-4}
3 (Low)	Low consequence for loss of human life. Small or negligible economic, social or environmental consequences, e.g., agricultural, storage buildings or greenhouses.	$\beta = 4.2$, p_f = approx. 10^{-5}	$\beta = 3.3$, p_f = approx. 10^{-3}

3. Characterisation of the β Index vs. QC

A time-dependent reliability index can be efficiently used to define the quality criteria of a structure as per design and depending on the execution quality. Assuming an asset with a design service life of 100 years (typically required for transport infrastructure) and a "medium" reliability class (RC2) as per Table 1, the reliability level at the end of the reference period of t_L = 100 years is expected to decline from β_0 = 4.7 ($p_f = 10^{-6}$) per annum to β_{100} = 3.63 ($p_f = 1.4 \cdot 10^{-4}$). This value is also assumed as the target value for the reference period, below which the structure does not serve the set requirements. This can then also be used as the reference value for determining the economic gain or loss of an investment related to the quality of execution.

Depending on the standards adhered to, a classification is recommended for the design supervision, construction inspection, materials and tolerances, as in Annex B of [10], or for concrete and steel structures, as in [11,12], respectively. In addition, there are individual authorities or sector standards, such as the one mentioned in [13], which are valid in Austria. When a component is assigned a reliability class (RC2 in the demonstrated case), in principle, the project-execution class is also adjusted at the same (medium) level. However, this can be varied as per the owner's intent or the structure's particularities. This is also reflected in Figure 2, assuming a variation in the construction execution quality while maintaining a uniform design standard. Figure 2 presents the reliability index curves over the technical lifetime of a structure based on a classification according to [13]. This overall execution classification (EXC) includes four rating classes, from 0 to 3, that correspond to a decreasing execution quality in the original construction (where 0 = best and 3 = worst).

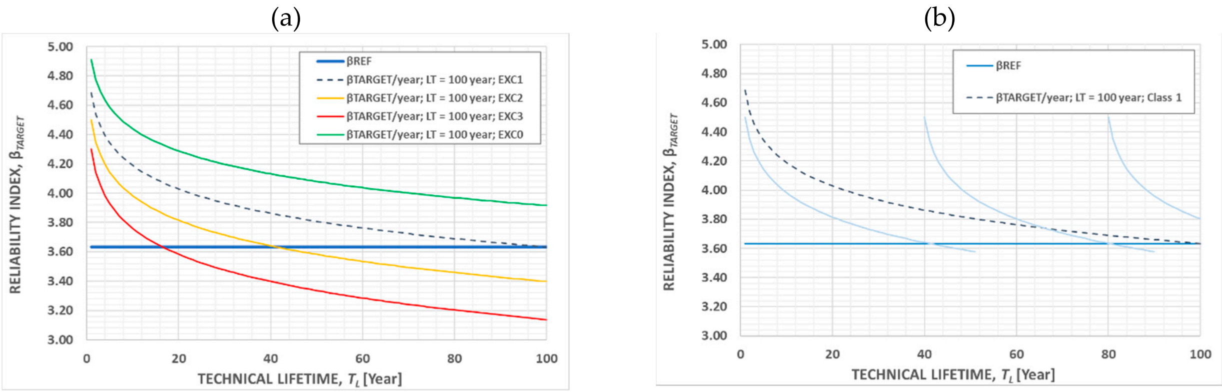


Figure 2. Quality assurance and technical service life. (**a**) Loss in lifetime and (**b**) monitoring or testing updating due to a reduced initial reliability index.

Based on the presented assessment, reduced uncertainty in action and resistance models and the execution of good standard can lead to a higher reliability, with a reliability index increase of 0.3 at $\beta_{0,EXC0}$ = 5.0/year and, consequently, a theoretical service life $t_{L,EXC0}$ > 100 years. This can be translated to a significant economic profit for the asset stakeholders due to a higher-quality and thus higher-reliance asset throughout its service life, associated with a prolonged operational timeframe.

On the contrary, a substandard or bad execution quality leads to lower initial reliability indices and correspondingly reduced service lives. As noted above, the theoretical formulations (Equation (6)) applied and presented in Figure 2a show a decrease in the target reliability from β_0 = 4.7 /year to β_{100} = 3.63 in 100 years for RC2 (dashed line). Assuming the mathematical formulation of Equation (6), this target reliability can be used as a reference for the investigated case. For a substandard execution quality (assumed here as Class 2 according to RVS standards (RVS stands for Richtlinien und Vorschriften für das Straßenwesen)) [14], a reduced initial reliability index of $\beta_{0,EXC2}$ = 4.5 can be assumed. In this case, (the amber line in Figure 2a), the reference target reliability of β = 3.63 is already reached at just over 40 years of operation. This indicates a loss of 60 years in the technical lifetime of the asset, already identified immediately after completion of the construction. For a bad-quality construction, a greater reduction in the annual target reliability can be observed, for instance, $\beta_{0,EXC3}$ = 4.3.

Note that the assessment herein does not account for the degradation process in the structural materials or for improvement interventions, such as repair and strengthening measures. Furthermore, it disregards the assessment of the asset's conditions through inspections, monitoring or non-destructive and destructive testing. If an assessment procedure is implemented, there is a possibility to extend the expected service life of the asset due to a reduction in the uncertainty regimes. This is graphically represented in Figure 2b. Here, the same situation detailed above is discussed, assuming a bad execution quality, i.e., Class 3 according to the RVS standards. The difference in this case is that a testing or interval-monitoring procedure is implemented in order to guarantee the condition and, thus, the reliability of the structure. As illustrated, this assessment process and accordingly the definition of the reliability level β can be processed each time when the reference line of $\beta_{100} = 3.63$ is violated, for example, at 40 and 80 years. This periodical assessment of the reliability level β associated with the respective technical assessment procedures can be understood as a periodic updating strategy according to the upcoming Model Code 2020.

4. Case Studies on Durability

As a state of practice, durability is specified through empirically limiting values (max. w/c, min. binder content, minimum reinforcement cover) defining the resistance of concrete elements against actions of the service environment, while environmental action is characterised by exposure classes. Models for describing the environmental actions are used to carry out a lifespan assessment, and the element's resistance against these actions needs to be accounted for as well. Both the actions and the mechanisms of damage are extremely complex, and their exact physical or chemical modelling cannot be described without a realistic elaboration of several significant parameters and associated uncertainties. A vital part of such modelling addresses the carbonation and chloride penetration in the concrete, which are presented below in detail. In the presented cases, particular focus is drawn to the used w/c ratio during concrete production and its consequences for the durability-related material parameters. Moreover, the effect of the selected cement types (CEM I, II or III) on the specification and construction of the structure is a driving parameter with a significant impact on the structure's durability-related reliability assessment.

4.1. Carbonation

In performance-based durability design with regard to carbonation, the carbonation process is described through the carbonation-prediction model presented in Table 2. Specifically, Figure 3a shows the basic function of the carbonation formulation, along with the complementary functions of the *fib* Model Code for Service Life [9]. This basic function is transformed to a limit state function for the definition of the β index (Figure 3b) according to the classical Basler–Cornell approach and, therefore, can be further used to define the failure probability p_f. The necessary input variables, indices and parameters for the function of Figure 3a are listed in Figure 3c, along with the symbols used in the *fib* Model Code 2010 presented in [15,16], the units and some suggested values proposed in the literature. The models presented in Figure 3a,b and their input parameters can be obtained through various inspection levels, as shown in Figure 3d.

(a) Model function

$$x_c(t) = k_{NAC} \cdot \sqrt{k_e \cdot k_c \cdot k_a} \cdot \sqrt{t} \cdot W(t)$$

$$k_e = \left[1 - \left(\frac{RH_a}{100}\right)^{g_e} / 1 - \left(\frac{RH_l}{100}\right)^{f_e}\right]$$

$$W(t) = \left(\frac{t_0}{t}\right)^{w}$$

$$k_c = a_c \cdot t_c^{\,b_c}$$

$$w = \left(\frac{p_{SR} \cdot TOW}{2}\right)^{b_w}$$

$$k_a = C_a / C_l$$

$$x_c(t_{exp}) = \sqrt{2 \cdot R_{NAC}^{\;-1} \cdot C_l} \cdot \sqrt{t_{exp}} = k_{NAC} \cdot \sqrt{t_{exp}}$$

(b) Limit state function

$$h(X) = d_k(t_{insp}) - k_{NAC} \cdot \sqrt{k_e \cdot k_c \cdot k_a} \cdot \sqrt{t} \cdot W(t) = 0$$

$$\beta(t) = \frac{m\left(x_{c,gr}(t)\right) - m(x_c(t))}{\sqrt{s\left(x_{c,gr}(t)\right)^2 + s(x_c(t))^2}}$$

$$p_f = \Phi(-\beta)$$

(c) Model Parameters

$x_c(t)$	*Carbonation depth in [mm] at time t*
R_{NAC}^{-1}	*inverse effective carbonation resistance of concrete (natural carbonation), determined in a standard test on test specimens that are exposed to a standard climate (relative humidity 65 ± 5% RH, temperature 20 ± 2 ° C, CO_2 concentration 0.04 ± 0.005 vol.% at atmospheric pressure) [(mm²/year)/(kg/m³)]*
k_{NAC}	*Carbonation rate for standard test conditions ($W(t) = 1$, $k_e = 1$, $k_c = 1$, $k_a = 1$, relative humidity 65 ± 5 % RH, temperature 20 ± 2 °C, CO_2 concentration 0.04 ± 0.005 vol.% CO_2 at atmospheric pressure) [(mm²/year)]*
k_e	*Function that describes the environmental influence of the relative air humidity (RHa) on the effective inverse carbonation resistance [–], (fib, 2006)*
k_c	*Function that describes the effect of curing / execution (time of curing tc) on the effective inverse carbonation resistance [–], (fib, 2006)*
k_a	*Function that describes the effect of CO_2 concentration in the ambient air [–]*
C_a	*CO_2 Concentration of the ambient air [kg/m³]*
C_l	*CO_2 Concentration during the concrete test [kg/m³]*
$W(t)$	*Function that describes the effect of wetting events that partially block the further penetration of CO_2. This parameter becomes more relevant with increasing depth of carbonation and thus with increasing time [–], (fib, 2006)*
t	*Exposure time [year]*
t_0	*Reference time point [a], here set to 0.0767, (Gehlen, 2000)*
w	*Weather exponent [–]*
TOW	*Frequency of rain (Time of Wetness) [–]*
p_{SR}	*Probability of rain [–]*
b_w	*Regression exponent [-], here set as normal distribution (mean value m = 0.446; standard deviation s = 0.163), (Gehlen, 2000)*
RHa	*Relative humidity in the carbonated surface layer, in a first approximation to the nearest weather station [%]*
RH_l	*Relative humidity, which is set as the reference humidity during the basic test, is set to RH_{ref} = 65%, taking into account the reference climate in ACC [%]*
f_e	*Exponent [–], here set to 5, (Gehlen, 2000)*
g_e	*Exponent [–], here set to 2.5, (Gehlen, 2000)*
a_c	*Regression parameter [–], $a_c = 7(-b_c)$*
t_c	*Duration of the aftertreatment [d]*
b_c	*Regression exponent [-], here set as normal distribution (mean value m = –0.567; standard deviation s = 0.024), (Gehlen, 2000)*
t_{exp}	*Time of experimental investigation [year]*
t_{insp}	*Time of inspection [year]*
d_k	*Limit carbonation depth in [mm] at the time t_{insp}*
$h(X)$	*Limit state function for the determination of the reliability index [M.-%/z]*
m	*Mean value [–]*
s	*Standard deviation [–]*
$\beta(t)$	*Reliability index [-]*
Φ	*Normal distribution [-]*
$p_f(t)$	*Failure probability [-]*

(d) Levels of obtaining model parameters

Level 1	*Documentary study, plans, short concrete names, documents about bridge testing, data collection from third parties, …*
Level 2	*On-site investigations, concrete composition of the bridge inspection including a spot check with the help of an external partner (acc. Test facility).*
Level 3	*Sampling and testing by external partners (e.g. battery test facility) with systematic determination of (i) the state of the building (including use of literature sources), and (ii) the depth of carbonation in the laboratory.*

Due to a non-linear limit state function and non-normally distributed variables, the β(t) factor has been computed by using sophisticated Monte Carlo simulation methods such as the LHS method used in FReET.

Figure 3. Carbonation prediction model. (**a**) Mathematical formulations to assess concrete carbonation, (**b**) limit state function and reliability assessment formulations, (**c**) their basic variables and (**d**) the levels of inspection to obtain the respective parameter.

Table 2. Model parameters of the carbonation model.

Variable ①		Symbol ②	Unit ③	Default Value ④	Level for Obtaining the Model Parameters ⑤	Test Procedure, Test Details ⑥	Literature and Background Information ⑦	Distribution * ⑧	Maximal Value ⑨	Minimal Value ⑩	Mean Value ⑪	Standard Deviation ⑫
(1a) R_{NAC}^{-1}			(mm^2/year)/(kg/m^3)	-	*3(ii)*	[17–19]	[5,9,20–26]	N			μ	Σ
(1b) k_{NAC} (alternative)			mm/year$^{0.5}$	-	*3(i)* and *3(ii)*	[5,20]	[5,9,20–26]	N	11	1	μ	σ
(2) k_c		b_c	–	-0.6	*3(i)*	-	[5,9,20–26]	N	−0.5	−0.8	−0.6	0.25
		t_c	days	3	*1* and *3(i)*	-	[5,9,17,20–26]	K	7	2	μ	–
(3) k_e		RH_a	%RH	75	*1, 2* and *3(i)*	-	[5,9,21–26]	W	85	75	μ	σ
		RH_l	%RH	65	*1, 2* and *3(i)*	-	[5,9,21–26]	K	65	65	65	–
		g_e	–	2.5	*3(i)*	-	[5,9,17,20–26]	K	2.5	2.5	2,5	–
		f_e	–	5	*3(i)*	-	[9,21–29]	K	5	5	5	–
(4) W(t)		ToW	–	0.2	*1* and *3(i)*	-	[5,9,20,23–26]	K	0.5	0	μ	–
		p_{dr}	–	0.3	*1* and *3(i)*	-	[5,9,20,23–26]	K	0.6	0	μ	–
		b_w	–	0.3	*1* and *3(i)*	-	[5,9,20,23–26]	K	0.71	0.24	0.5	0.16
		t_0	year	0.767	*1*	-	[5,9,20,23–26]	K	0.0767	0.0767	0.0767	–
(5a) Ca			kg/m^3		*3*	-	[5,9,20,23–26]	N			μ	σ
(5b) k_a (alternative)		C_a	vol.%	0.042	*1* and *2*	-	[9,25–27]	N	0.044	0.036	μ	σ
		C_l	vol.%	0.044	*3b*	-	[25–27]	K	0.044	0.044	0.04	–
(6) c			mm	35	*1, 2* and *3*	-	[25–27]	N	85	15	μ	σ
(7) t_{SL}			year	100	*1* and *3*	-	-	K	120	1	50	–

* Normal distribution, *N*; lognormal distribution, LN; beta distribution, *B*; constant, *K*.

There is a difference in the forms of survey between classical inspection (Level 1), the more detailed on-site inspection (Level 2) and laboratory testing (Level 3). A more detailed classification of the inspection levels to obtain the model parameters is presented in Table 2.

Table 2 provides global information that is useful for stakeholders involved in this assessment procedure. In particular, it provides a general overview of the common default values to be used in the model function (column ④), the level of inspection for each parameter with reference to Figure 3d (column ⑤), the testing methods to obtain each model parameter (column ⑥) and the respective literature source in which the parameters are described (column ⑦). Table 2 also presents statistical descriptions of certain values (grey-shaded areas, columns ⑧ to ⑫), such as the distribution type, the minimum and maximum values, the mean value and the standard deviation for each parameter.

As mentioned above, the equations in Figure 3b are used to calculate the reliability index β of the model function. Table 3 presents the relationship of the reliability index β with the failure probability for clarity and further reference in the paper. Specifically, the value of k_{NAC} of the model function that corresponds to certain carbonation-resistance classes depends on the type of cement and the w/c ratio, as indicated in Table 4, for a range of typical boundary conditions relevant to practice. Table 5 demonstrates the classification of the environmental or exposure classes (XC1 to XC4) and the respective values for each exposure class that correspond to unfavourable environmental conditions. Similarly, Table 6 presents the mean value and the standard deviation of CO_2 concentration Ca in ambient air for each of the aforementioned exposure classes. Thus, Tables 4 to 6 provide a detailed set of values based on the environmental and cement conditions for the selection of the most appropriate input for the carbonation function of Figure 3a,b.

Keeping in mind the global values presented in Table 2 and using the already presented model functions, a first set of analyses was performed, using the commercial software FReET [30] to calculate the reliability indices β and the carbonation depth. The method used in software FReET to obtain the reliability indices is the LHS-based Monte Carlo simulation; for details, see [31]. The analysis input parameters for each cement type from the software are represented in Figure 4. The analyses were performed for the different cement classes CEM I, CEM II/B and CEM III/B, with a corresponding w/c ratio of 0.55, set into the exposure class XC4. The increase in the carbonation depth in relation to the exposure time for 50 years is as shown in Figure 5a. The CEM I compound shows the best performance (the lowest carbonation depth), with the carbonation front reaching a maximum depth of 8.6 mm after 50 years (solid blue line). The CEM II/B composition shows the same carbonation depth after just 17 years of exposure (solid green line) and for the CEM III/B composition, a carbonation depth of 8.6 mm is reached after less than 10 years (solid orange line) and almost double the one of CEM I at the 50-year service life.

Table 3. Reliability index β vs. failure probability p_f.

Reliability index β	0	1	2	3	4	4.5	5
Failure probability p_f	50%	15%	*2.3%*	0.1%	0.03%	0.003%	0.0005%

Table 4. W/b ratios depending on the cement type and the carbonation resistance class according to [25,26].

Resistance Class		**2**	**3**	**4**	**5**	**6**	**7**
Ranges of k_{NAC} (mm/year$^{0.5}$)		$1 < k_{NAC} \leqq 2$	$2 < k_{NAC} \leqq 3$	$3 < k_{NAC} \leqq 4$	$4 < k_{NAC} \leqq 5$	$5 < k_{NAC} \leqq 6$	$6 < k_{NAC} \leqq 7$
Cement type	CEM I	0.45	0.50	0.55	0.60	0.65	–
	CEM II/A	0.45	0.50	0.55	0.60	0.65	–
	CEM II/B	0.40	0.45	0.50	0.55	0.60	0.65
	CEM III/A	0.40	0.45	0.50	0.55	0.60	0.65
	CEM III/B	–	0.40	0.45	0.50	0.55	0.60

Table 5. Proposals for unfavorable climatic conditions according to [25,26].

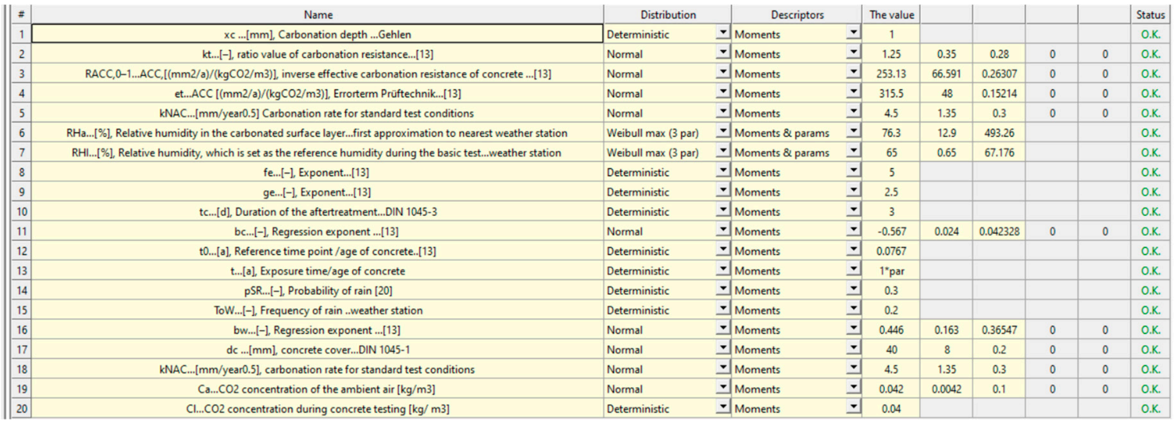
Exposure Class		**RH_a (%)**	**ToW (–)**	**p_{dr} (–)**
XC1	Dry	≥65	= 0.0	= 0.0

XC2	Wet	≥90	= 0.0	= 0.0
XC3	Outdoor, sheltered	≥75	= 0.0	= 0.0
XC4	Outdoor, unsheltered	≥75	≥0.2	≥0.1

Table 6. CO_2 concentration *Ca* in ambient air according to [25,26].

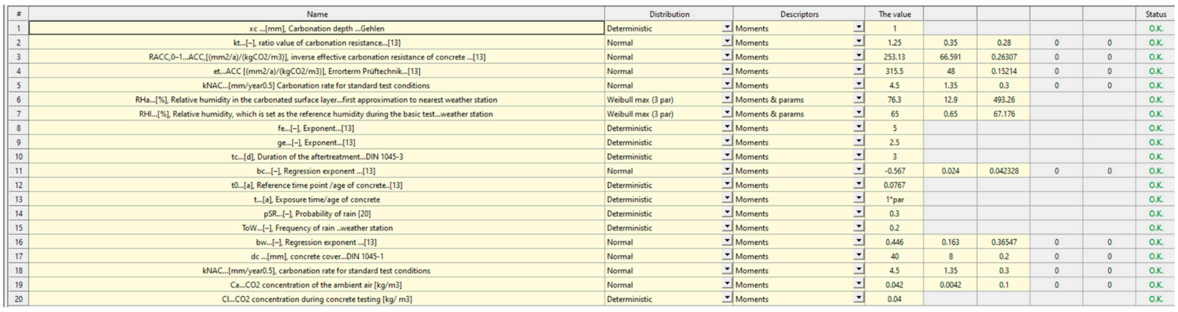
Exposure Class			**CO_2 Concentration *Ca* (vol.%)**	
			Mean Value μ	**Standard Deviation σ**
XC1	Normal	Indoor	$0.036 \leqq \mu \leqq 0.043$	$0.0010 \leqq \sigma \leqq 0.0080$
XC2	Normal	E.g., foundation	$0.036 \leqq \mu \leqq 0.043$	$0.0010 \leqq \sigma \leqq 0.0080$
XC3/XC4	Normal	Rural	$0.036 \leqq \mu \leqq 0.042$	$0.0010 \leqq \sigma \leqq 0.0055$
		Urban	$0.038 \leqq \mu \leqq 0.043$	$0.0015 \leqq \sigma \leqq 0.0080$

(**a**) CEM I



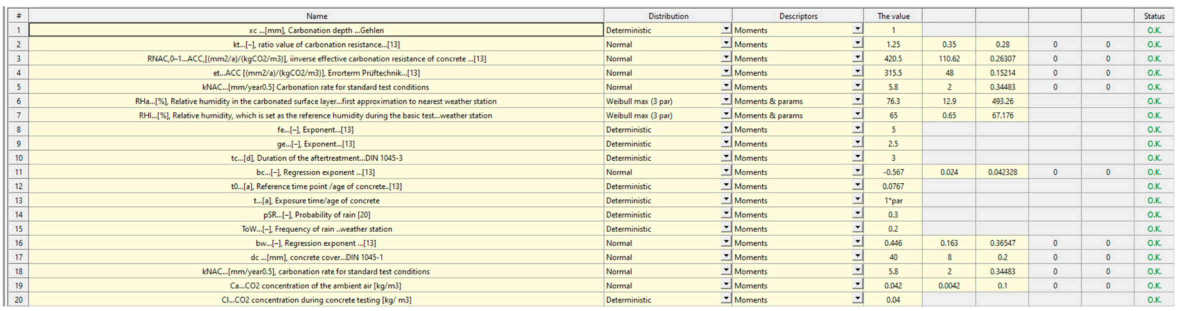
#	Name	Distribution	Descriptors	The value					Status
1	xc ...[mm], Carbonation depth ...Gehlen	Deterministic	Moments	1					O.K.
2	kt...[−], ratio value of carbonation resistance...[13]	Normal	Moments	1.25	0.35	0.28	0	0	O.K.
3	RACC,0–1...ACC,[(mm2/a)/(kgCO2/m3)], inverse effective carbonation resistance of concrete ...[13]	Normal	Moments	253.13	66.591	0.26307	0	0	O.K.
4	et...ACC [(mm2/a)/(kgCO2/m3)], Errorterm Prüftechnik...[13]	Normal	Moments	315.5	48	0.15214	0	0	O.K.
5	kNAC...[mm/year0.5] Carbonation rate for standard test conditions	Normal	Moments	4.5	1.35	0.3	0	0	O.K.
6	RHa...[%], Relative humidity in the carbonated surface layer...first approximation to nearest weather station	Weibull max (3 par)	Moments & params	76.3	12.9	493.26			O.K.
7	RHl...[%], Relative humidity, which is set as the reference humidity during the basic test...weather station	Weibull max (3 par)	Moments & params	65	0.65	67.176			O.K.
8	fe...[−], Exponent...[13]	Deterministic	Moments	5					O.K.
9	ge...[−], Exponent...[13]	Deterministic	Moments	2.5					O.K.
10	tc...[d], Duration of the aftertreatment...DIN 1045-3	Deterministic	Moments	3					O.K.
11	bc...[−], Regression exponent ...[13]	Normal	Moments	-0.567	0.024	0.042328	0	0	O.K.
12	t0...[a], Reference time point /age of concrete..[13]	Deterministic	Moments	0.0767					O.K.
13	t...[a], Exposure time/age of concrete	Deterministic	Moments	1*par					O.K.
14	pSR...[−], Probability of rain [20]	Deterministic	Moments	0.3					O.K.
15	ToW...[−], Frequency of rain ..weather station	Deterministic	Moments	0.2					O.K.
16	bw...[−], Regression exponent ...[13]	Normal	Moments	0.446	0.163	0.36547	0	0	O.K.
17	dc ...[mm], concrete cover...DIN 1045-1	Normal	Moments	40	8	0.2	0	0	O.K.
18	kNAC...[mm/year0.5], carbonation rate for standard test conditions	Normal	Moments	4.5	1.35	0.3	0	0	O.K.
19	Ca...CO2 concentration of the ambient air [kg/m3]	Normal	Moments	0.042	0.0042	0.1	0	0	O.K.
20	Cl...CO2 concentration during concrete testing [kg/ m3]	Deterministic	Moments	0.04					O.K.

(**b**) CEM II



#	Name	Distribution	Descriptors	The value					Status
1	xc ...[mm], Carbonation depth ...Gehlen	Deterministic	Moments	1					O.K.
2	kt...[−], ratio value of carbonation resistance...[13]	Normal	Moments	1.25	0.35	0.28	0	0	O.K.
3	RACC,0–1...ACC,[(mm2/a)/(kgCO2/m3)], inverse effective carbonation resistance of concrete ...[13]	Normal	Moments	253.13	66.591	0.26307	0	0	O.K.
4	et...ACC [(mm2/a)/(kgCO2/m3)], Errorterm Prüftechnik...[13]	Normal	Moments	315.5	48	0.15214	0	0	O.K.
5	kNAC...[mm/year0.5] Carbonation rate for standard test conditions	Normal	Moments	4.5	1.35	0.3	0	0	O.K.
6	RHa...[%], Relative humidity in the carbonated surface layer...first approximation to nearest weather station	Weibull max (3 par)	Moments & params	76.3	12.9	493.26			O.K.
7	RHl...[%], Relative humidity, which is set as the reference humidity during the basic test...weather station	Weibull max (3 par)	Moments & params	65	0.65	67.176			O.K.
8	fe...[−], Exponent...[13]	Deterministic	Moments	5					O.K.
9	ge...[−], Exponent...[13]	Deterministic	Moments	2.5					O.K.
10	tc...[d], Duration of the aftertreatment...DIN 1045-3	Deterministic	Moments	3					O.K.
11	bc...[−], Regression exponent ...[13]	Normal	Moments	-0.567	0.024	0.042328	0	0	O.K.
12	t0...[a], Reference time point /age of concrete..[13]	Deterministic	Moments	0.0767					O.K.
13	t...[a], Exposure time/age of concrete	Deterministic	Moments	1*par					O.K.
14	pSR...[−], Probability of rain [20]	Deterministic	Moments	0.3					O.K.
15	ToW...[−], Frequency of rain ..weather station	Deterministic	Moments	0.2					O.K.
16	bw...[−], Regression exponent ...[13]	Normal	Moments	0.446	0.163	0.36547	0	0	O.K.
17	dc ...[mm], concrete cover...DIN 1045-1	Normal	Moments	40	8	0.2	0	0	O.K.
18	kNAC...[mm/year0.5], carbonation rate for standard test conditions	Normal	Moments	4.5	1.35	0.3	0	0	O.K.
19	Ca...CO2 concentration of the ambient air [kg/m3]	Normal	Moments	0.042	0.0042	0.1	0	0	O.K.
20	Cl...CO2 concentration during concrete testing [kg/ m3]	Deterministic	Moments	0.04					O.K.

(**c**) CEM III



#	Name	Distribution	Descriptors	The value					Status
1	xc ...[mm], Carbonation depth ...Gehlen	Deterministic	Moments	1					O.K.
2	kt...[−], ratio value of carbonation resistance...[13]	Normal	Moments	1.25	0.35	0.28	0	0	O.K.
3	RNAC,0–1...ACC,[(mm2/a)/(kgCO2/m3)], iinverse effective carbonation resistance of concrete ...[13]	Normal	Moments	420.5	110.62	0.26307	0	0	O.K.
4	et...ACC [(mm2/a)/(kgCO2/m3)], Errorterm Prüftechnik...[13]	Normal	Moments	315.5	48	0.15214	0	0	O.K.
5	kNAC...[mm/year0.5] Carbonation rate for standard test conditions	Normal	Moments	5.8	2	0.34483	0	0	O.K.
6	RHa...[%], Relative humidity in the carbonated surface layer...first approximation to nearest weather station	Weibull max (3 par)	Moments & params	76.3	12.9	493.26			O.K.
7	RHl...[%], Relative humidity, which is set as the reference humidity during the basic test...weather station	Weibull max (3 par)	Moments & params	65	0.65	67.176			O.K.
8	fe...[−], Exponent...[13]	Deterministic	Moments	5					O.K.
9	ge...[−], Exponent...[13]	Deterministic	Moments	2.5					O.K.
10	tc...[d], Duration of the aftertreatment...DIN 1045-3	Deterministic	Moments	3					O.K.
11	bc...[−], Regression exponent ...[13]	Normal	Moments	-0.567	0.024	0.042328	0	0	O.K.
12	t0...[a], Reference time point /age of concrete..[13]	Deterministic	Moments	0.0767					O.K.
13	t...[a], Exposure time/age of concrete	Deterministic	Moments	1*par					O.K.
14	pSR...[−], Probability of rain [20]	Deterministic	Moments	0.3					O.K.
15	ToW...[−], Frequency of rain ..weather station	Deterministic	Moments	0.2					O.K.
16	bw...[−], Regression exponent ...[13]	Normal	Moments	0.446	0.163	0.36547	0	0	O.K.
17	dc ...[mm], concrete cover...DIN 1045-1	Normal	Moments	40	8	0.2	0	0	O.K.
18	kNAC...[mm/year0.5], carbonation rate for standard test conditions	Normal	Moments	5.8	2	0.34483	0	0	O.K.
19	Ca...CO2 concentration of the ambient air [kg/m3]	Normal	Moments	0.042	0.0042	0.1	0	0	O.K.
20	Cl...CO2 concentration during concrete testing [kg/ m3]	Deterministic	Moments	0.04					O.K.

Figure 4. Case studies of model parameters in software FReET used for (**a**) CEM I, (**b**) CEM II and (**c**) CEM III.

The respective results in terms of the reliability index β are obtained through the limit state function, and they are shown in Figure 5b. For CEM I, the reliability index curve shows a value of $\beta = 2.9$ for 50 years. For CEM II/B, it shows a value of $\beta = 1.9$ for 50 years, and for CEM III/B, it shows a value of $\beta = 1.59$ for 50 years. The initial reliability indices (at year 1) are 4.66, 4.41 and 4.27, respectively. As seen in Table 3, the w/c ratio highly influences the k_{NAC} values, which in turn strongly control the reliability prediction. Assuming that the cement type CEM I is chosen, with a w/c ratio value of 0.6, the k_{NAC} value also increases to 4.0. The results of using this input are presented by the dashed blue line in Figure 5, with the mean value of carbonation depth at 50 years' exposure time increasing to 11.5 mm and the reliability index decreasing to $\beta = 2.44$.

In Figure 6a, the reliability index of the exposure time of 50 years for CEM I and the two alternative k_{NAC} values of 3.0 and 4.0 are presented. It can be demonstrated that, for $k_{NAC} = 3.0$ (w/c = 0.55), the β_{REF} is reached after a technical lifetime of 50 years. In the case of $k_{NAC} = 4.0$ (w/c = 0.60), the performance of the structure declines, with the β_{REF} reached only after 27 years of exposure. Similar to the concept presented in Figure 2b, Figure 6b accounts for an assessment of the asset's conditions through inspections, monitoring or non-destructive and destructive testing. Through this process, the expected service life of the asset can be extended. As illustrated in Figure 6b, this periodic adjustment, in line with the upcoming Model Code 2020, is applied every time the reliability level β exceeds the reference line of $\beta_{50} = 2.91$, for example, at 27 years.

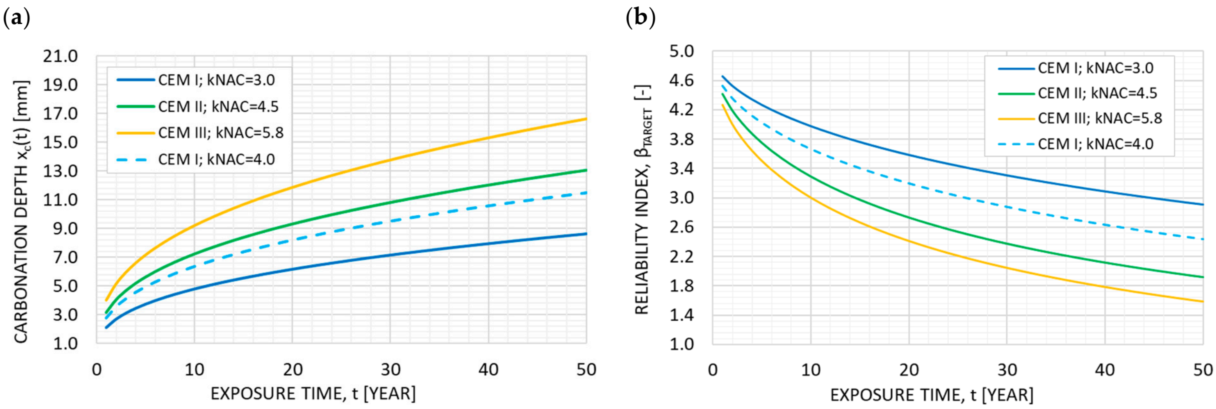


Figure 5. (**a**) Carbonation depth vs. exposure time for CEM I, CEM II and CEM III and (**b**) reliability index vs. exposure time.

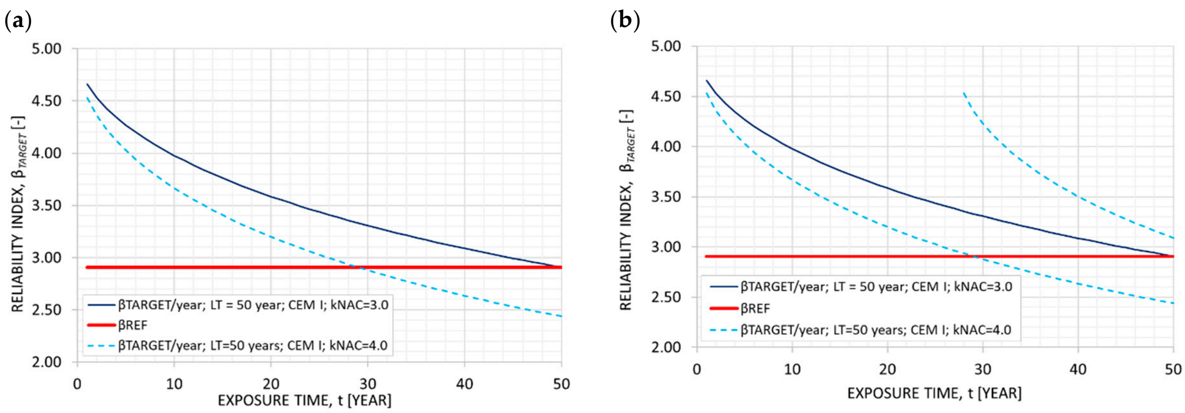


Figure 6. Quality assurance and technical service life. (**a**) Loss in lifetime and (**b**) monitoring or testing updating due to a reduced initial reliability index.

4.2. Chloride Ingress

There are several numerical models to describe chloride penetration in concrete, most of which are based on Fick's second law of diffusion. In the current study, the modified DuraCRETE model [32] was applied for the time-dependent penetration of chloride in concrete. Specifically, Figure 7a shows the basic function of the chloride ingress formulation, which is also used in the *fib* Model Code for Service Life Design [9]. This basic function is transformed into a limit-state function for the definition of the β index (Figure 7b), according to the classical Basler–Cornell approach; therefore, it can be further used to define the failure probability p_f, as per Table 3. The necessary input variables, indices and parameters for the function of Figure 7a are listed in Figure 7c, together with their descriptions and suggested values as proposed in the literature. The input parameters of the models presented in Figure 7a,b can be derived through the same inspection levels as with carbonation, repeated in Figure 7d for completeness. A more detailed representation of the inspection levels to obtain the model parameters is presented in Table 7.

In a similar way to Table 2, Table 7 also provides a general overview of the common default values used in the chloride ingress model function and the following analysis (column ④). Furthermore, the level of inspection for each parameter (column ⑤), the testing methods to obtain each model parameter (column ⑥) and the respective literature source in which the parameters are described (column ⑦) are presented. Furthermore, Table 7 provides the statistical descriptions of certain values (grey-highlighted areas, columns ⑧ to ⑫), such as the distribution type, the minimum and maximum values and each parameter's first two statistical moments as used in the chloride ingress analysis herein.

The input parameters of the study below were chosen by assuming an exposure class XD3/XS3, with a minimum concrete cover of c_{min} = 40 mm and a value w/c = 0.45, according to [27]. The parameters used with the suggestive values for CEM III are shown in Figure 8 as an example of the software input. A durability calculation of the chloride penetration was carried out over a period of 50 years, and the critical chloride concentration was selected as equal to C_{gr} = 0.60 M.-%/z. Figure 9a shows the chloride concentration at the depth of the concrete cover in relation to the exposure time. The values of CEM III show an improved behaviour compared to CEM I and CEM II. Specifically, after an exposure time of 50 years, the chloride concentration values for CEM III are approximately 9 times lower than those for CEM I. A major factor that influences the durability calculations of concrete for chloride penetration is the chloride migration coefficient of water-saturated concrete $D_{RCM,0}$, which is further influenced by the w/c ratio. The results of the reliability index in relation to the exposure time are presented in Figure 9a. It can be seen that CEM III again shows the best performance and has a reliability index β = 1.56 after 50 years, with CEM I and CEM II reaching this reference value after 4 and 6 years of exposure time, respectively.

Table 7. Model parameters of the chloride ingress model.

Variable ①	Symbol ②	Unit ③	Default Value ④	Level for Obtaining the Model ⑤	Test Procedure, Test Details ⑥	Literature and Background Information ⑦	Distribution * ⑧	Maximum Value ⑨	Minimum Value ⑩	Mean Value ⑪	Standard Deviation ⑫
(1) C(x,t)	M.-%/z	-	*3(ii)*			[9,27–39]	–	–	–	–	–

(2) $C_{S,\Delta x}$		M.-%/z	-	*3(ii)*	[9,27–38]	[9,27–39]	LN	3	0.75	2.6	1.2
(3) x		mm	35	*1* and *2*		[9,27–39]	N	80	15	55	5
(4) Δx		mm	5	*3(ii)*		[9,27–39]	B	50	0	8.9	5.6
(5) $D_{Eff,C(t)}$ (mm²/a)	k_e	–	-	*3(i)* and *3(ii)*	[33,39,40]	[9,27–39]	–	–	–	–	–
	k_t	–	1	*3(i)* and *3(ii)*		[9,27–39]	K	1	2.5	2.5	–
	$D_{RCM,0}$ *CEM I*	mm²/a	60	*3(i)* and *3(ii)*		[9,27–40]	N	280	789	320	64
	$D_{RCM,0}$ *CEM III/B*	mm²/a	320	*3(i)* and *3(ii)*		[9,27–39]	N	107	44	60	12
	A	–	-	*3(i)* and *3(ii)*		[9,27–39]	–	–	–	–	–
(6) k_e (-)	b_e	K	4800	*3a*	-	[9,27–39]	N	5950	3650	4800	700
	T_{ref}	K	293	*3(ii)*	-	[9,27–39]	K	293		–	–
	T_{IST}	K	280	*1* and *2*	-	[5,9,27–39]	N	293	271	282	7
(7) A(t) (-)	$a_{CEM\ I}$	–	0.3	*3(ii)*	-	[9,27–39]	B	1	0.1	0.3	0.12
	$a_{CEM\ III/B}$	–	0.45	*3(ii)*	-	[9,27–39]	B	1	0.1	0.45	0.20
	t_0	a	0.076	*3(i)*	-	[9,27–39]	LN	0.088	0.065	0.0767	0.007

* Normal distribution, *N;* lognormal distribution, LN; beta distribution, *B;* constant, *K.*

(a) Model Function

$$C(x,t) = C_{S,\Delta x} \cdot \left[1 - erf\,\frac{x - \Delta x}{2 \cdot \sqrt{D_{Eff,C}(t) \cdot t}}\right]$$

$$D_{Eff,C}(t) = k_e \cdot k_t \cdot D_{RCM,0} \cdot A(t)$$

$$A(t) = \left(\frac{t_0}{t}\right)^a$$

$$k_e = \exp\left(b_e\left(\frac{1}{T_{ref}} - \frac{1}{T_{IST}}\right)\right)$$

(b) Limit state function

$$h(X) = C_{gr}(x) - C_{S,\Delta x} \cdot \left[1 - erf\,\frac{x - \Delta x}{2 \cdot \sqrt{D_{Eff,C}(t) \cdot t}}\right] = 0$$

$$\beta(t) = \frac{m\left(C_{gr}(x)\right) - m(C(x,t))}{\sqrt{s\left(C_{gr}(x)\right)^2 + s(C(x,t))^2}}$$

$$p_f = \Phi(-\beta)$$

(c) Model Parameter

C(x,t) *Chloride content of the concrete at depth x (component surface)*

$C_{S,\Delta x}$ *Chloride concentration at depth Δx depending on the pending chloride exposure [M.-%/z]*

x *Depth [mm], with a corresponding chloride content C(x,t)*

k_e *Function that describes the environmental influence of the relative air humidity (RHa) on the effective inverse carbonation resistance [–] (fib, 2015)*

Δx *Depth range [mm], which shows chloride concentrations deviating from Fick's behavior due to intermittent chloride effects*

$D_{Eff,C}(t)$ *Effective chloride diffusion coefficient of concrete at the time of observation t [mm²/a]*

t *Age of Concrete [a]*

k_e *Parameters for taking the temperature dependence of $D_{Eff,C}(t)$ into account [–]*

k_t *Transmission parameters to consider deviations between chloride migration coefficients that are determined under accelerated conditions (Rapid Chloride Migration - $D_{RCM,0}$) (* and diffusion coefficients that are determined under natural conditions e.g. in the laboratory [-], here set to 1 (Gehlen, 2000)*

$D_{RCM,0}$ *Chloride migration coefficient of water-saturated concrete, determined at the reference point in time t_0 at defined and produced upstream test specimens [mm²/a]*

A *Aging Term [–]*

b_e *Regression parameters [K], here set as normal distribution (mean value m = 4800; standard deviation s = 700) (Gehlen, 2000)*

T_{ref} *Reference temperature [K], here set to 293 (Gehlen, 2000)*

T_{IST} *Temperature of the ambient air, or structural element temperature [K]*

a *Aging exponent [–]*

t_0 *Reference point in time [a], here set to 0.0767 (Gehlen, 2000)*

$h(X)$ *Limit state function for the determination of the reliability index [M.-%/z]*

$C_{gr}(x)$ *Limit value of the chloride content of the concrete at depth x (component surface)*

m *Mean Value [–]*

s *Standard Deviation [–]*

$\beta(t)$ *Reliability index [-]*

Φ *Normal Distribution [-]*

$p_f(t)$ *Failure probability [-]*

(d) Levels for obtaining the model parameters

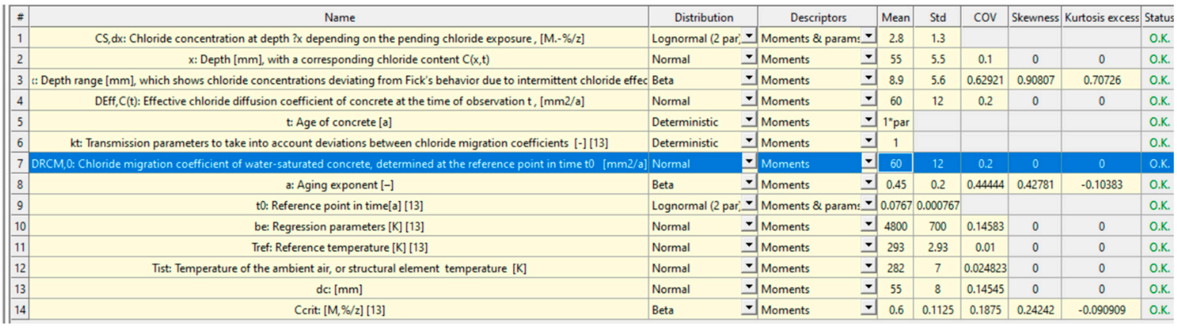
Level 1 *Documentation study, plans, short concrete names, documents about bridge testing, data collection from third parties, ...*

Level 2 *On-site investigations, concrete composition of the bridge inspection including spot checks with the help of external partners (acc. Testing facility).*

Level 3 *Testing and sampling by external partners (e.g. battery test facility) with systematic determination of (a) the state of the building (including use of literature sources), and (b) the chloride concentration depth in the laboratory.*

Due to a non-linear limit state function and non-normally distributed variables, the β(t) factor has been computed by using sophisticated Monte Carlo simulation methods such as the LHS method used in FReET.

Figure 7. Chloride prediction model. (**a**) Mathematical formulations to assess concrete carbonation, (**b**) limit state function and reliability assessment formulations, (**c**) their basic variables and (**d**) the levels of inspection to obtain the respective parameters.



#	Name	Distribution	Descriptors	Mean	Std	COV	Skewness	Kurtosis excess	Status
1	CS,dx: Chloride concentration at depth ?x depending on the pending chloride exposure , [M.-%/z]	Lognormal (2 par)	Moments & params	2.8	1.3				O.K.
2	x: Depth [mm], with a corresponding chloride content C(x,t)	Normal	Moments	55	5.5	0.1	0	0	O.K.
3	:: Depth range [mm], which shows chloride concentrations deviating from Fick's behavior due to intermittent chloride effec	Beta	Moments	8.9	5.6	0.62921	0.90807	0.70726	O.K.
4	DEff,C(t): Effective chloride diffusion coefficient of concrete at the time of observation t , [mm2/a]	Normal	Moments	60	12	0.2	0	0	O.K.
5	t: Age of concrete [a]	Deterministic	Moments	1*par					O.K.
6	kt: Transmission parameters to take into account deviations between chloride migration coefficients [-] [13]	Deterministic	Moments	1					O.K.
7	DRCM,0: Chloride migration coefficient of water-saturated concrete, determined at the reference point in time t0 [mm2/a]	Normal	Moments	60	12	0.2	0	0	O.K.
8	a: Aging exponent [–]	Beta	Moments	0.45	0.2	0.44444	0.42781	-0.10383	O.K.
9	t0: Reference point in time[a] [13]	Lognormal (2 par)	Moments & params	0.0767	0.000767				O.K.
10	be: Regression parameters [K] [13]	Normal	Moments	4800	700	0.14583	0	0	O.K.
11	Tref: Reference temperature [K] [13]	Normal	Moments	293	2.93	0.01	0	0	O.K.
12	Tist: Temperature of the ambient air, or structural element temperature [K]	Normal	Moments	282	7	0.024823	0	0	O.K.
13	dc: [mm]	Normal	Moments	55	8	0.14545	0	0	O.K.
14	Ccrit: [M,%/z] [13]	Beta	Moments	0.6	0.1125	0.1875	0.24242	-0.090909	O.K.

Figure 8. Input parameters for CEM I, CEM II and CEM III assigned in software FReET.

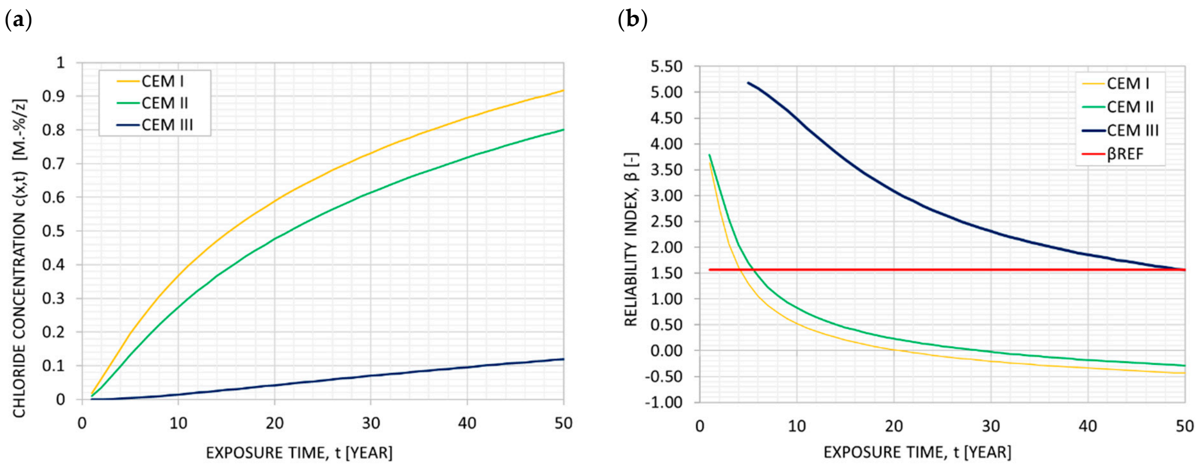


Figure 9. (**a**) Chloride concentration vs. exposure time and (**b**) reliability index vs. exposure time of CEM I, CEM II and CEM II for chloride concentration at the reinforcement level of 55 mm, according to [5].

In a further analysis of durability, the influence of the w/c factor of CEM III on the $D_{RCM,0}$ was investigated. The variations in w/c values as an agent of $D_{RCM,0}$ were chosen according to Gehlen (2000), and they are presented in Table 8; the analysis results are shown in Figure 10a. It can be observed that increasing the chloride mitigation factor from 60 to 95 mm²/year reduces the reliability index after 50 years from $\beta_{REF} = 1.56$ to $\beta = 0.94$, with β_{REF} reached by 28 years of exposure time. As per the concept presented in Figure 2b, Figure 10b accounts for a periodic adjustment of the asset's conditions through inspections, monitoring or non-destructive and destructive testing, which facilitates the extension of the structure's expected service life. As illustrated in Figure 10b, this assessment process is applied every time the reliability level β exceeds the reference line of $\beta_{50} = 1.56$, for example, at 28 years.

Table 8. Chloride mitigation coefficient $D_{RCM,0}$ for CEM III/B used in the analyses displayed in Figure 10.

w/c (-)	$D_{RCM,0}$ (10^{-12} m²/s)	$D_{RCM,0}$ (mm²/year)
0.40	1.4	44
0.45	1.9	60
0.50	2.8	88
0.55	3.0	95
0.60	3.4	107

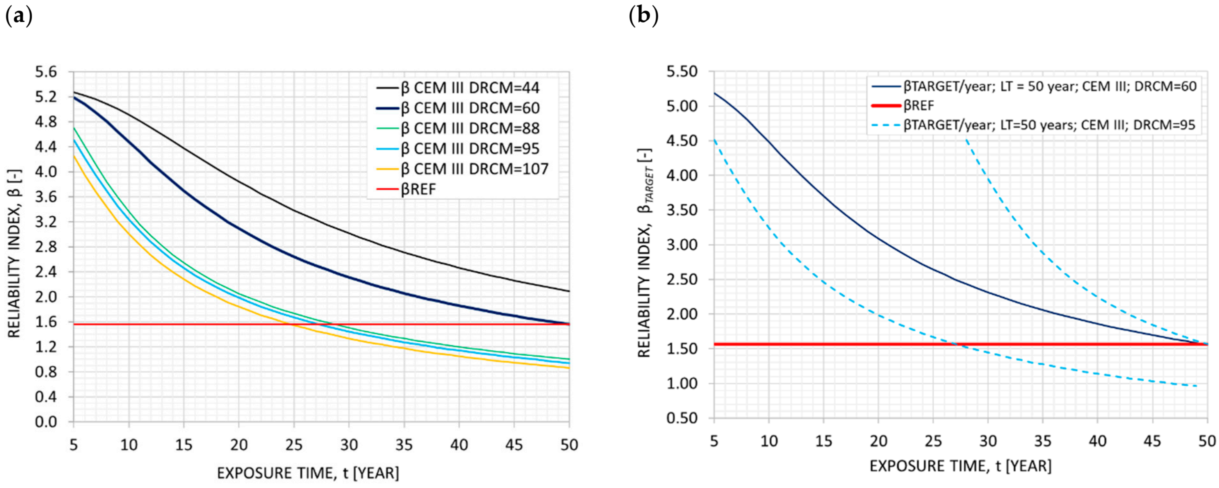


Figure 10. Quality assurance and technical service life for chloride ingress. (**a**) Loss in lifetime and (**b**) monitoring or testing updating due to a reduced initial reliability index.

5. Case Study on Bearing Capacity

In this case study, a one-way-reinforced concrete slab was used, with a width of b = 1 m and a height h = 0.20 m. The calculation of the moment bearing capacity assumes a longitudinal reinforcement area of A_s = 27.14 cm²/m. The limit-state function (Equation (2)) is formulated with respect to the bending-tensile strength of the reinforcement on the ULS (ultimate limit state) level based on [41,42]. The geometrical dimension of the slab, the reinforcement area, the material properties and the applied load are elaborated on in the probabilistic analysis. The model uncertainty of the resistance bending moment capacity was characterised as 1.20, according to the JCSS Probabilistic Model Code [43]. The influence parameters addressed in this case are the static height of the reinforcement, depending on the reinforcement placement accuracy, and the loss of reinforcement action due to, e.g., corrosion and gradual bond loss as a result of an inadequate execution of the design. An additional influence parameter is the concrete class used, which also strongly affects the reliability assessment results.

Specifically, Figure 11a shows the basic function of the bearing capacity formulation, along with the complementary functions presented in [44]. The basic bearing-capacity function is transformed to a limit state function (Figure 11b) to be used as a basis for the definition of the β index, according to the classical Basler–Cornell approach, and it can therefore be used to define the failure probability (p_f). The necessary input variables, indices and parameters for the function of Figure 11a are presented in detail in Figure 11c. The models presented in Figure 11a,b and their input parameters can be obtained through various inspection levels, as shown in Figure 3d. As already mentioned in Section 4.1 and included in Figure 11 for consistency, a distinction can be made in the form of survey between the Level 1, Level 2 and Level 3 inspections.

Table 9 gives a general overview of the chosen input parameters that were used in the model function, with the respective literature sources in which the parameters are described. The table also presents the statistical model description of certain parameters (grey-highlighted area). In the analyses, the reinforcement area was taken as a constant variable, and it was multiplied by the parameters in the range of 0.4 to 1 to simulate the loss of reinforcement steel. The analysis was performed with two different types of concrete (C20/25 and C30/37), and for each type of concrete, the distance from the bottom of the slab to the reinforcement was varied between 32 and 42 mm. To calculate the limit-state excess probabilities and subsequently the reliability indices β, the software FReET was used and the first set of input parameters is shown in Figure 12.

Table 9. Model parameters of the bearing capacity model.

Variable Symbol	Unit	Literature and Background Information	Distribution *	Mean Value	Standard Deviation	COV
(1) $\boldsymbol{M_{RD}}$	(MNm)	[43–45]	-	-	-	-
(2) $\boldsymbol{M_{ED}}$	(MNm)	[43–45]	LN	0.05	0.01	0.2
(3) $\boldsymbol{A_S}$	(cm²/m)	[43–45]	K	27.14	0.5428	0.02
(4) $\boldsymbol{f_y}$	(N/mm²)	[43–45]	LN	560	30	0.05357
(5.1) $\boldsymbol{f_c}$ C30/37	(N/mm²)	[43–45]	LN	38	4.75	0.125
(5.1) $\boldsymbol{f_c}$ C30/37	(N/mm²)	[43–45]	LN	27	3.5	0.125
(6) b	(m)	[43–45]	N	0.2	0.2	0.02
(7) h	(m)	[43–45]	N	1	0.004	0.02
(8.1) d_1	(m)	[43–45]	G	0.032	0.0054	0.17
(8.2) $d1_2$	(m)	[43–45]	G	0.042	0.00714	0.17
(9) $\boldsymbol{k_a}$	(-)	[43–45]	N	0.416	0.0405	0.05
(10) $\boldsymbol{\alpha_c}$	(-)	[43–45]	K	0.85	-	-
(11) $\boldsymbol{\alpha_R}$	(-)	[43–45]	N	0.816	0.0208	0.05
(12) $\Theta_{R(M)}$	(-)	[43–45]	LN	1.2	0.18	0.15
(13) Θ_E	(-)	[43–45]	LN	1	0.07	0.07

* Normal distribution, *N*; lognormal distribution, LN; constant, *K*; gamma distribution, *G*.

(a) Model Functions

$$M_{RD} = A_s \cdot f_y \cdot \left((h - d_1) - \frac{f_y \cdot A_s * k_a}{b \cdot \alpha_c \cdot \alpha_R \cdot f_c} \right)$$

resistance model function

$$M_{RD} = F_{yd} \cdot d - F_{cd} \cdot a \quad \text{and} \quad F_{yd} = F_{cd}$$

$$M_{RD} = A_s \cdot f_y \cdot (h - d_1) \cdot (1 - \frac{k_a}{\alpha_R} * \frac{A_s \cdot f_y}{b \cdot (h - d_1) \cdot \alpha_c \cdot f_c})$$

$$M_{RD} = A_s \cdot f_y \cdot \left((h - d_1) - \frac{f_y \cdot A_s * k_a}{b \cdot \alpha_c \cdot \alpha_R \cdot f_c} \right)$$

$$d_1 = c_{nom} + d_{B\ddot{u}} + 0.5 \cdot d_{St}$$

acting force model function

$$M_{ED} = M_{EG} + M_{EQ}$$

(b) Limit state functions

probabilistic model function

$$G(M_R) = \Theta_{R(M)} \cdot A_s \cdot f_y \cdot \left((h - d_1) - \frac{f_y \cdot A_s * k_a}{b \cdot \alpha_c \cdot \alpha_R \cdot f_c} \right) - (\Theta_{E(MG)} \cdot M_{EG} + \Theta_{E(MQ)} \cdot M_{EQ})$$

simplified model function

$$G(M_R) = \Theta_{R(M)} \cdot A_s \cdot f_y \cdot \left((h - d_1) - \frac{f_y \cdot A_s * k_a}{b \cdot \alpha_c \cdot \alpha_R \cdot f_c} \right) - (\Theta_E \cdot M_E)$$

$$\beta(t) = \frac{m(M_{RD}) - m(M_{ED})}{\sqrt{s(M_{RD})^2 + s(M_{ED})^2}}$$

$$p_f = \Phi(-\beta)$$

(c) Model parameter

M_{RD} *Resistance force in the reinforcement layer [MNm]*
M_{ED} *Acting force in the reinforcement layer [MNm]*
A_S *Reinforcement area [m²/m]*
f_y *Reinforcement yield strength of steel BSt 500 [N/mm²]*
f_c *Concrete compression strength of Concrete C30/37 [N/mm²]*
b *width of the slab [m]*
h *height of the slab [m]*
c_{nom} *concrete cover [m]*
$d_{B\ddot{u}}$ *diameter stirrup reinforcement [m]*
d_{St} *diameter longitudinal reinforcement [m]*
d_1 *Distance of the reinforcement layer from the bottom surface of the reinforced concrete slab [m]*
k_a *height coefficient = 0,416 at the limit strain of the concrete εc = -3,5%‰ [-] (Zilch, & Zehetmaier, 2006).*
α_c *Coefficient to take into account long-term effects on compressive strength and for conversion between cylinder compressive strength and uniaxail compressive strength of concrecte = 0,85 [–] (Zilch, & Zehetmaier, 2006).*
α_R *fulness coefficent for the compressive strength distribution = 0,810 [-] (Zilch, & Zehetmaier, 2006).*
$\Theta_{R(M)}$ *Modell uncertainty of the resistance bending moment capacity[-]*
$\Theta_{E(MG)}$ *Modell uncertainty of the acting force, caused by dead load [-]*
$\Theta_{E(MQ)}$ *Modell uncertainty of the action force, caused by live load [-]*
Θ_E *Modell uncertainty of the acting force [-]*
$G(M_R)$ *Limit state function for the determination of the reliability index [-]*
m *mean value [–]*
s *Standard Deviation [–]*
ß(t) *Reliability index [-]*
Φ *normal distribution [-]*
$p_f(t)$ *V Failure probability [-]*

(d) Levels for obtaining the model parameters

Level 1 *Documentation study, plans, short concrete names, documents about bridge testing, data collection from third parties, ...*

Level 2 *On-site investigations, concrete composition of the bridge inspection including spot checks with the help of external partners (acc. Testing facility).*

Level 3 *Testing and sampling by external partners (e.g. battery test facility) with systematic determination of (a) the state of the building (including use of literature sources), and (b) the chloride concentration depth in the laboratory.*

Due to a non-linear limit state function and non-normally distributed variables, the β(t) factor has been computed by using sophisticated Monte Carlo simulation methods, such as the LHS method used in FReET.

Figure 11. Bearing capacity prediction model. (**a**,**b**) Mathematical formulations, (**c**) their basic variables and (**d**) the levels of inspection to obtain them.

Figure 13a demonstrates the development of the reliability index for a variation of reinforcement loss. It can be seen that the distance between the reinforcement and the concrete surface (d_1) is more important in terms of reliability than compression strength. As the reinforcement area A_s decreases, the influence of the concrete compression strength becomes less significant. For a concrete class C30/37 and d_1 = 32 mm (solid dark-blue line), the reliability index is $\beta = 4.75$, which corresponds to a failure probability of $p_f = 1.01 \cdot 10^{-6}$. For a lifetime of 100 years, the β_{TARGET} is reduced to 3.72, as seen in Figure 13b. For this assessment situation, and a loss of 45% reinforcement area, the β_{TARGET} would be critical for the flexural load-bearing assessment. For the same concrete type C30/37 but an increased distance between the reinforcement and the concrete surface with d_1 = 42 mm (solid orange line), a loss of 40% would already lead to the service life limit of β_{TARGET} = 3.72 after 30 years. On the contrary, a lower concrete class (C20/25) and the same distance between the reinforcement and the concrete surface d_1 = 32 mm (solid green line) would allow for a loss of 43% of the reinforcement area and the reliably index β_{TARGET} would be attained at 48 years.

In the baseline assessment and as shown in Figure 12, the COV for the positioning of the reinforcement is set to COV(d_1) = 0.17. For a COV(d_1) of 0.30, the reliability index without any loss of reinforcement drops to 4.52 ($p_f = 3.07 \cdot 10^{-6}$), with a notable consequence for the remaining expected service life, as shown in Figure 13a,b (dashed blue line).

#	Name	Distribution	Descriptors	Mean	Std				Status
1	As...[cm^2/m] reinforcement area	Deterministic	Moments	0.002714*par					O.K.
2	fc...[N/mm^2] concrete compression strength C30/37	Lognormal (2 par)	Moments	38	4.75	0.125	0.37695	0.25369	O.K.
3	fy...[N/mm^2] yield strength	Lognormal (2 par)	Moments	560	30	0.053571	0.16087	0.046042	O.K.
4	b...[m] width of the slab	Normal	Moments	1	0.02	0.02	0	0	O.K.
5	h...[m] height of the slab	Normal	Moments	0.2	0.004	0.02	0	0	O.K.
6	d1...[m] reinforcement distance from the concrete surface	Gamma (2 par)	Moments	0.032	0.00544	0.17	0.34	0.1734	O.K.
7	mR...[-] model uncertainty resistance	Lognormal (2 par)	Moments	1.2	0.18	0.15	0.45337	0.36766	O.K.
8	FE...[MNm] acting force in the reinforcement	Lognormal (2 par)	Moments	0.05	0.01	0.2	0.608	0.66439	O.K.
9	aR...[-] coefficient for the compressive strength distribution	Normal	Moments	0.81	0.0405	0.05	0	0	O.K.
10	ka...[-] height coefficient at edge strain of concrene e = 0.035%	Normal	Moments	0.416	0.0208	0.05	0	0	O.K.
11	aC...[-] coefficient for long term effects on compressive stength	Deterministic	Moments	0.85					O.K.
12	mE...[-] model uncertainty acting force	Lognormal (2 par)	Moments	1	0.07	0.07	0.21034	0.078761	O.K.
13	fc...[N/mm^2] concrete compression strength C20/25	Lognormal (2 par)	Moments	28	3.5	0.125	0.37695	0.25369	O.K.
14	d1 +1cm ...[m] reinforcement distance from the concrete surface	Gamma (2 par)	Moments	0.042	0.00714	0.17	0.34	0.1734	O.K.
15	d1 cov=0.30 ...[m] reinforcement distance from the concrete surface	Gamma (2 par)	Moments	0.032	0.0096	0.3	0.6	0.54	O.K.

Figure 12. Input parameters for the bearing capacity assigned in software FReET.

The development of the system's reliability with this change in the standard deviation in the reinforcement layer location (distance from the concrete surface), under the assumption of a lognormal distribution and the same concrete class (C30/37), is illustrated in Figure 14a. This represents a typical quality issue in construction, and the influence of quality on the structure's reliability is reflected. Figure 14b shows that the system with reduced reliability, as the standard deviation in the reinforcement location needs to be updated more than two times throughout the asset's lifetime in order to maintain the target reliability index for at least 100 years.

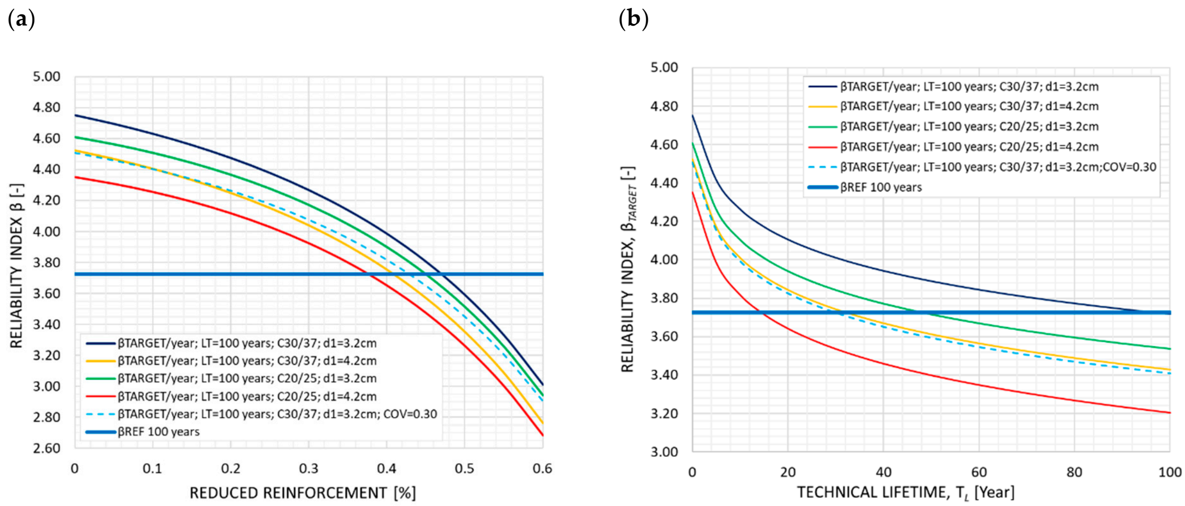


Figure 13. (**a**) Development of the reliability index vs. the loss of reinforcement and (**b**) development of the reliability index over the lifetime of 100 years.

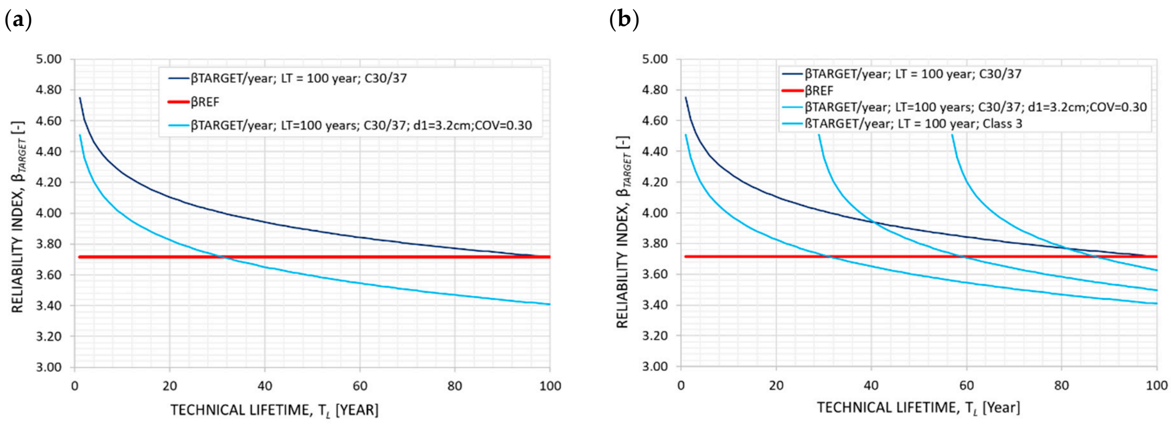


Figure 14. Quality assurance and technical service life. (**a**) Loss in lifetime and (**b**) monitoring or testing updating due to a reduced initial reliability index.

6. Case Study on Fastenings

Quality in fastening installation is known to have a substantial influence on the assembly's load-bearing resistance and reliability, particularly for post-installed fastenings, which is also noted in the newly introduced Eurocode 2 part 4 EN 1992-4 "Design of fastenings for use in concrete" [46], see Table 10. The design code is based on the typical partial factor design, while it considers that the partial safety factor for the resistance side is the product of the material safety factor γ_{Mc} familiar for concrete and the safety factor to account for the sensitivity to installation of post-installed fasteners γ_{inst}. The factor γ_{inst} is generally derived from standardised installation safety tests. The predecessor of EN 1992-4, namely the ETAG 001—Annex C [47], proposed the following values for the partial safety factor taking account of the installation safety of an anchor system as follows:

Tension loading

γ_2 = 1.0 for systems with high installation safety;

= 1.2 for systems with normal installation safety;

= 1.4 for systems with low but still acceptable installation safety.

In many project specifications, the installers require training and certification. A typical procedure is presented in [48], while standardisation institutes provide detailed guidance for the installation of fastenings [49,50]. Notable failures with significant consequences have been attributed to design and installation errors, and it has also been reported that inspections could have helped avoid the incidents [51,52]. Site surveys have also indicated that the installation quality and the level of awareness for the exact installation procedures may strongly vary [53,54]. Inherent defects are also present during the production of fastening products, in the range of $2{\cdot}10^{-5}$ [55], although factory QC processes may withhold most defective products from the market.

Table 10. Excerpt from "Table 4.1—Recommended values of partial factors" from EN 1992-4:2018 [46].

Concrete Failure	
Cone break-out failure, edge break-out failure, blow-out failure and pry-out failure	$\gamma_{Mc} = \gamma_c * \gamma_{inst}$. $\gamma_c = 1.5^a$; see EN 1998 for seismic repair and strengthening. γ_{inst} = 1.0 for headed fasteners and anchor channels satisfying the requirement of 4.5 (in tension and shear),

	≥ 1.0 for post-installed fasteners in tension; see relevant European Technical Product Specification, = 1.0 for post-installed fasteners in shear.
Splitting failure	$\gamma_{Msp} = \gamma_{Mc}$
Pull-out Failure	
Pull-out and combined pull-out and concrete failure	$\gamma_{Mp} = \gamma_{Mc}$
a The values are in accordance with EN 1992-1-1.	

Based on test results for bonded anchors [56], the random variables are provided in Table 11 for (a) an anchor with good installation quality (Class 1) and (b) an anchor with substandard quality (Class 2). The load is assumed from typical applications at 40 kN with a variation coefficient of COV(E) = 0.125. The worst-case scenario of hole cleaning (no hole cleaning) in tests led to a decrease in load bearing by 30%, which is roughly proportional to the added safety reserves represented by the installation factor of γ_{inst} = 1.4. However, this also leads to an increase in the standard deviation by a factor of 7.0, which automatically renders the reliability of the system below any acceptable level (β = 1.1; p_f = 0.135), and hence this case is not further considered. The random variables associated with substandard borehole preparation in the assessment below are taken with a multiplication factor of 0.95. For Class 1, and the results indicate a reliability index of 4.68, while for Class 2, although the differences are marginal, the results show a much lower reliability index of 3.99. Assuming an 80-year service life, the β_{REF} is assumed equal to the end-of-life reliability of the Class 1 system, equal to β_{120} = 3.7, as shown in Figure 15. Consequently, for an 80-year lifetime of a Class 2 system, multiple inspections are needed in order to maintain the target reliability of 3.7, ideally every 3 years. In an idealised case, applying the mean values of Class 1 used as the deterministic input with an absolute absence of variation (Class 0) is also given in the graph for comparison. This set of parameters yields an origin reliability value of β_0 = 4.91.

Table 11. Model parameters of the anchor pull-out probabilistic model; the range of values for the embedment depth equals the minimum and maximum boundary deviations of the variable, and it is equal to the standard deviation for the bond strength and the load.

Variable Symbol		Unit	Literature and Background Information	Distribution *	Mean Value	Range	COV	Distribution *	Mean value	Range	COV
(1)	Anchor diameter $\boldsymbol{d}$	(mm)	-	K	16	-	-	K	16	-	-
(2)	Embedment depth $\boldsymbol{h_{eff}}$	(mm)	-	R	80	0.5	0.004	R	80	2	0.018
(3)	Bond strength $\boldsymbol{\tau}$	(N/mm²)	[56]	LN	16.05	0.39	0.024	LN	15.25	0.41	0.027
(4)	Load $\boldsymbol{E}$	(N)	-	N	40,000	5000	0.125	N	40,000	5000	0.125

* Normal distribution, *N*; lognormal distribution, LN; rectangular distribution, *R*; constant, *K*.

(**a**) (**b**)

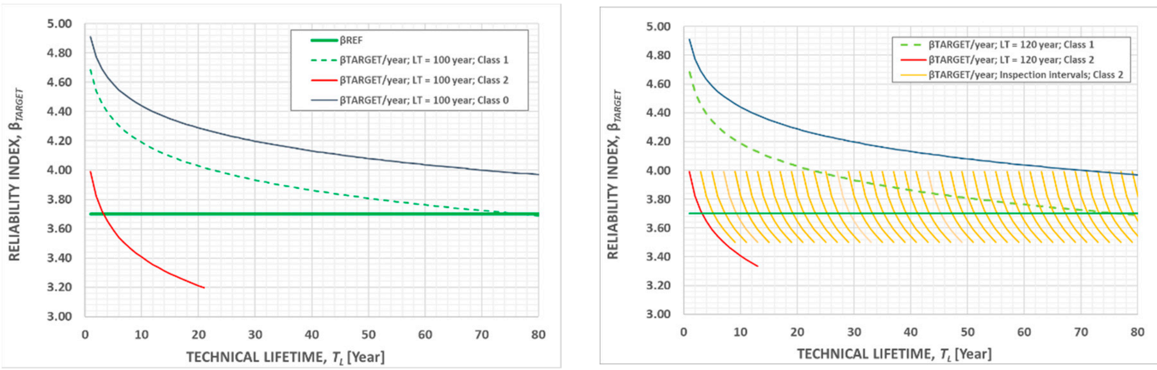


Figure 15. Quality assurance and technical service life. (**a**) Loss in lifetime and (**b**) monitoring or testing updating due to a reduced initial reliability index.

7. Summary of Findings

In the carbonation analysis, it is made evident that the wrong selection of cement (CEM) type and the associated w/c ratio can reduce the component's service life while an adequate selection of these parameters can increase the component's service life. For example, although all investigated cases have a reliability index of the same orders of magnitude (between 4.27 and 4.66), assuming a target reliability index of $\beta = 2.9$, a service life of 50 years can be guaranteed for CEM I while the service life with a CEM III-based mix can reach just over 10 years. Inversely, for the same target service life, the CEM III component reaches a reliability index of only 1.59 for 50 years. With a variation of 5% in the w/c ratio of the CEM I mix, the service life of 50 years ($\beta = 2.9$) drops to only 27 years. Along the same lines, a chloride ingress analysis qualifies CEM III with an allowable exposure time of 50 years for a reliability index of $\beta = 1.56$, whereas structures with CEM I and CEM II are deemed to maintain this performance level for only 4 and 6 years of exposure, respectively. With regard to the load-bearing capacity of a one-way slab under flexural response and tolerances in the placement of reinforcement and its effective area, for a placement error (e.g., due to the lack of rebar support in the formwork) of only 10 mm, the estimated service life of the structure drops from 100 years to under 50 years. Finally, a minimal variation in the bond strength of the adhesive used, together with a common installation tolerance in the embedment depth, can reduce a typical bonded anchor's service life from 80 years to only 3 years.

To counterbalance the significant loss of structural reliability, time intervals for periodic updating in the spirit of the *fib* Model Code 2020 are calculated. The inspection and updating intervals aim to maintain the reliability of the investigated systems above the reference value, which is estimated as the target reliability of a well-executed concrete specification or structural detail. These intervals were found to be 30 years in the case of the one-way slab and 3 years in the case of anchors. Depending on these results, a periodic adjustment procedure and the means used to perform this adjustment can be decided. This study distinguishes between three levels of updating the assessment parameters: documentation-based assessment, on-site investigations and inspections and systematic sampling (e.g., by using regular laboratory tests or monitoring).

8. Conclusions

In this paper, the structural reliability of typical engineering design situations is evaluated with regard to the structural component's execution quality. In particular, it is shown on the basis of case studies that even marginal quality variations in the initial condition of the structure can dramatically affect the structure's service-life duration. The strong sensitivity of the life-cycle reliability of structures and elements of QC and quality assurance at construction and along the service life of a structure is hence indicated. The quality criteria can be associated with structural reliability by means of the involved parameters' statistical models, based on the novel methodology, using the reliability index β. This methodology was trialled on two realistic case studies of durability assessment (durability design for concrete carbonation and chloride ingress assessment), a typical flexural design for a one-way concrete slab and a design for a single bonded anchor in concrete. The method is applicable to all new concrete structures with on-site or off-site construction elements, particularly with special requirements in terms of life-cycle performance. However, although this concept was tested for each presented design situation, this study does not cover all possible aspects of advanced materials and the respective whole-life assessment and environmental influences. In future investigations, this concept can be applied not only to a wider range of modern civil engineering problems, such as construction materials under various environmental impacts, high- and ultra-high-performance concrete and advanced concrete composites in terms of resilience, durability and sustainability, but also extreme events, such as seismic actions, as discussed in [57–70].

As noted, marginal quality variations can lead to significant variations in the performance levels of the structure, for example, the residual service life. Considering the investigated cases, the reduction in service life can fluctuate from 20% to 95% of the envisaged target service life, which can translate to an equal loss of whole-life value of the asset. The monetary loss in this case can be represented by a reduced duration of the operational performance or by the inspection and upgrading activities. In all cases, the presented investigations affirm the importance of an adequate and effective quality management plan at the initial phases of a construction project, which is of minor expense but can lead to gain in terms whole-life costing and life-cycle performance perspective.

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