Chapter 4
Integrated Concept for Service Life Design of Concrete Structures

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Abstract With regard to a sustainable development people have to use the existing range of resources in an optimum way. For the building industry this means that the durability of our civil structures is sufficient to withstand the different environmental conditions during a defined service life. Therefore, it is necessary to manage the whole lifetime of a civil structure, from the planning phase to the deconstruction, to avoid cost-intensive maintenance measures and corresponding downtimes. Against this background, the role of lifetime management is briefly shown in this paper. Furthermore, the requirements and tools for the realization of an effective lifetime management are presented. Within these considerations it is taken into account that different deterioration mechanisms occur in combination with each other. These interactions have to be implemented into deterioration time models. Special examples, which clarify some selective elements of the procedure of lifetime management, close this paper.

4.1 Introduction

During the last decade the concept of sustainable development came noticeably into the focus of public and political awareness. The most important principle of the concept is based on the maxim that current ambitions for resource use must not affect the needs of future generations. It is valid for all kind of human activities, and as a consequence the principle of sustainable development should be applied as well to construction and building activities and the building industry, respectively.

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M.N. Fardis (ed.), *Innovative Materials and Techniques in Concrete Construction: ACES Workshop*, DOI 10.1007/978-94-007-1997-2_4,

This holds particularly true as the construction industry covers a significant percentage of the material and energy use by mankind.

The realisation of the principle of sustainable development requires a successful lifetime management of civil structures. In this context it is very important to predict reliably the changing material behaviour with time and thus the durability of civil concrete structures, which underlie different and complex exposure conditions. Moreover, the deterioration mechanisms usually occur combined and interact among each other. Hence, for a realistic service life prediction these interactions have to be modelled by means of probabilistic tools. In this way, cost minimisation can also be promoted by optimised construction processes and maintenance. The latter involves an effective inspection and assessment management in order to upgrade the repair strategies as well as the preventive and repair work.

4.2 Prediction of the Durability of Civil Concrete Structures

The existing procedure for durability design of civil concrete structures is based on empirical experience in civil engineering. The national and international standards imply special deem-to-satisfy limits in connection with rough environmental classifications to ensure the durability of structures for an approximate defined minimum lifetime, e.g. 50 years according to DIN EN 206 (2001) and DIN 1045-2 (2001), respectively. For instance, the compliance of the regulations on a maximum water/cement ratio of a concrete and a minimum concrete cover is supposed to prevent the concrete and the reinforcement from damaging effects resulting from frost attack or chloride ingress, respectively. Therefore, this concept is a prescriptive approach which considers the different environmental actions on civil structures in a descriptive way.

It is quite evident that the above-mentioned concept is connected with several unfavourable consequences allowing only a rough estimate of the durability. Neither the environmental actions nor the material resistance, i.e. the different deterioration mechanisms in concrete, are considered in a realistic way. Instead of this, the different environmental actions are roughly subdivided in so-called exposure classes which are associated with limiting values for the concrete composition and the concrete compressive strength. The intensity of the different exposure conditions is described in terms like "moderate humidity" or "cyclical wet and dry". This means that the difference between action S and resistance R, which is a measure of the failure safety, is only estimated by experience (see Fig. 4.1, left). The effective safety margin is unknown to the designer. This descriptive concept is supposed to "guarantee" a sufficient concrete performance for a fixed service life of e.g. 50 years. Hence, it is not possible to quantify, among other things, the necessary concrete properties for a specified lifetime of e.g. 20 or 100 years. In addition, it is also not possible to consider different limit states in view of damage risks, e.g. the time span until the depassivation of the reinforcement occurs.

In contrast, the performance concept based on a probabilistic approach is appropriate to allow for quantitative estimations of the durability of concrete structures. Hereby, the increasing damage process with time, i.e. the interaction of action and

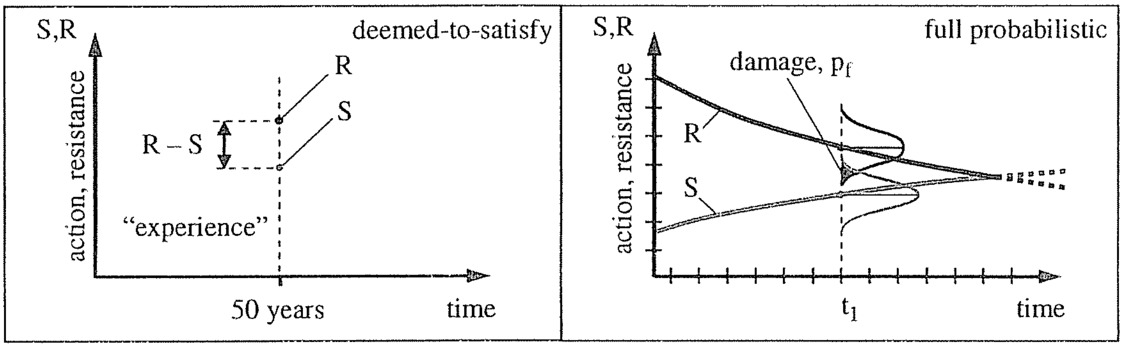


Fig. 4.1 Action and resistance in view of the durability of concrete members. Schema of the descriptive concept (*left*) and the performance concept including a probabilistic approach (*right*)

resistance, affecting the concrete structure is modelled by means of appropriate deterioration time laws, and the material resistance is additionally quantified. Since there are several uncertainties in the action- and resistance-related parameters, it is necessary that the variability and the observable scatter, e.g. for the material parameters, are described by means of related statistical parameters. As a consequence, the safety margin between the well defined functions for the action S and the resistance R can be expressed in terms of the failure probability; see the overlap area between the two curves in Fig. 4.1 on the right side.

By means of the probabilistic performance concept the time-dependent increase of damage and the failure probability according to a defined unintended condition of the structure can be calculated. It is obvious that the application of statistical methods in durability design is – in analogy to the structural design approach – an essential tool in order to quantify the performance of structural concrete. The decisive advantage of the performance concept is based on the fact that the time-dependent durability of concrete structures can be expressed in terms of the failure probability or reliability indices, respectively (see Sect. 4.3.5).

It is already evident that the next generation of standards will include probabilistic methods for durability design.

The required tools have been developed within recent years (The European Union 1997, 1998, 1999, 2000; Sarja and Vesikari 1996). For instance, well established models which describe the degradation process in uncracked concrete for the initiation phase are listed in the fib Model Code for Service Life Design (fib 2006). By means of the developed statistical tools and advanced degradation time laws the prediction of the lifetime of a structure is feasible for civil engineers in practice. This will improve the lifetime management significantly.

4.3 Prediction of Lifetime – Background and Basic Principles

It is evident from the preceding section that the prediction of the lifetime needs time functions for the actions S and the resistance R (see Fig. 4.1, right), including information on the related variability. Further statistical methods to quantify the interactions of the S and R functions have to be applied. These methods are already well-developed and usually implemented in commercial statistical software tools. In the subsequent paragraphs the essential elements and design steps for the prediction of the lifetime of civil structures are briefly summarised.

4.3.1 Description of the Deterioration Process

The increasing deterioration with time, i.e. the gradual loss of durability due to environmental actions, has to be described by means of deterioration time laws, also called material laws or material models. Such laws should preferably take into consideration real physical or chemical mechanisms. This holds true e.g. for the degradation process caused by carbonation. Of course, carbonation itself does not damage the concrete or the concrete cover, respectively, but if the carbonation front reaches the reinforcement depassivation takes place, which initiates corrosion of the reinforcement in the presence of moisture and oxygen. Considering this process in terms of action S and resistance R, the action is described by means of the material law for the progress of the carbonation front in concrete taking into account environmental and material parameters. The resistance is given by the thickness of the concrete cover.

4.3.2 Statistical Quantification of Parameters

The parameters included in the models for the action S and the resistance R are not exact values but they scatter around average values, see again Fig. 4.1, right. This can be easily observed for the carbonation depth (action) as well as for the concrete cover (resistance) in a concrete member in practice. Hence, the varying parameters are considered as random variables, also called basic variables. If such a basic variable is measured, the corresponding mean value and coefficient of variation as well as the type of the distribution function have to be determined.

4.3.3 Deterioration Process and Limit States

A limit state is understood as a condition at which a civil structure or a structural component ceases to comply with its intended serviceability. In the case of carbonation induced corrosion of the reinforcement, a limit state may be defined by the condition that the carbonation front reaches the reinforcement. Correspondingly, for chloride induced corrosion a limit state is reached if the actual chloride content is equal to the critical chloride content in the depth of the reinforcement. It is self-evident that further limit states may be defined, e.g. the initiation of cracks or any higher level of chloride content.

Table 4.1 Values for the failure probability p_f and the related reliability index β (DIN EN 1990 2002)

p_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1.28	2.32	3.09	3.72	4.27	4.75	5.20

4.3.4 *Intended Service Life of Civil Structures*

The loss of durability, i.e. the increase of the deterioration with time, reduces the reliability or the safety of a civil structure. In order to be able to evaluate this reliability or this safety at any age of the structure, a reference period for the service life has to be specified. Reference values of the service life of buildings and structures are listed in relevant standards and guidelines. As an example, the intended service life of residential buildings and other simple engineering structures is 50 years, for hydraulic structures and complex engineering structures it is 100 years (DIN EN 1990 2002).

4.3.5 *Failure Probability and Limit State Function*

The failure probability p_f is defined as the probability for exceeding a limit state within a defined reference time period. When this occurs an unintentional condition of a considered building component is reached.

The magnitude of the failure probability is closely connected with the interaction of the resistance and the action functions and varies with time (see Fig. 4.1, right). This interaction may be described by means of the so-called limit state function Z which is defined according to Eq. 4.1.

$$Z = R - S \tag{4.1}$$

The function Z represents the elementary form of a limit state function in which R and S are random variables. If the value of Z turns to zero, the limit state will be reached. The stochastical properties of the function Z can be expressed in the form of a distribution function, if this function is considered to be normal distributed and the resistance R as well as the action S are expressed using related mean values μ and standard deviations σ.

By means of the introduction of the so-called reliability index β, a direct correlation between the reliability index β and the failure probability p_f is obtained. In case of a normal distributed limit state function Z, the failure probability p_f can be determined directly by Eq. 4.2.

$$p_f = p\{Z < 0\} = \Phi(-\beta) \tag{4.2}$$

Here, the variable $\Phi(-)$ is the distribution function of the standardised normal distribution. The correlation between various values for the failure probability p_f and the reliability index β is shown in Table 4.1. Note e.g. that the often used 5%

Table 4.2 Target values of the reliability index β according to (DIN EN 1990 2002) and (Rackwitz 1999)

Relative cost of safety measures	Reliability index β (DIN EN 1990 2002)	Reliability index β (Rackwitz 1999)
High	1.3 ($p_f \approx 10\%$)	1.0 ($p_f \approx 16\%$)
Moderate	1.7 ($p_f \approx 5\%$)	1.5 ($p_f \approx 7\%$)
Low	2.3 ($p_f \approx 1\%$)	2.0 ($p_f \approx 2\%$)

quantile in civil engineering is equal to a failure probability of $5 \cdot 10^{-2}$ which corresponds to a reliability index $\beta = 1.645$.

The above given definitions and derivations are generally valid, i.e. for mechanical as well as for physical and chemical actions and resistances which are related to durability. As the durability of concrete is pronouncedly dependent on time t, the functions for S, R and Z are also time-dependent (see Fig. 4.1, right). As a consequence, the reliability index β is also obtained as a function of time, where the value of $\beta = \beta(t)$ is decreasing with time as durability decreases and failure probability increases, respectively.

Table 4.2 indicates target values of the reliability index for building components in the serviceability limit state (SLS) (DIN EN 1990 2002; Rackwitz 1999). Considering the case of depassivation of the reinforcement due to carbonation or chloride ingress the target reliability index is recommended to be $\beta = 1.3$, see (fib 2006).

The calculation of the failure probability p_f for a building component considering a particular mechanism related to durability (e.g. carbonation-induced corrosion of the reinforcement) may be performed by the use of the subsequent Eq. 4.3.

$$p_f = p\{Z < 0\} \leq p_{target} \tag{4.3}$$

As the failure probability increases with time, $p_f = p_f(t)$ approaches $p_{target} = \text{const}$. Finally, $p_f(t = t_{crit}) = p_{target}$ is obtained, where t_{crit} is the time when the failure probability of the member becomes equal to the target failure probability. In practical applications this analysis is done by means of the reliability index β as p_f and p_{target} may be easily expressed as the reliability indices β and β_{target}, see e.g. Fig. 4.4.

4.4 Lifetime Prediction – Application in Practice

The method of lifetime prediction can be applied to a single structural element or component as well as to complex engineering structures like bridges or tunnels. In the latter case additional procedures have to be taken into account. Further, it may be applied for new structures at the stage of planning and also for existing structures, e.g. in order to clarify the remaining lifetime. In the following the application of the procedures of lifetime prediction is shown for some practical cases.

The procedure of lifetime prediction involves the design steps which are summarised in Table 4.3. This overview also indicates the distinction between planned and existing civil structures. In the first case the concrete structure is designed for an intended service life, which is a so-called design for durability. In the latter case the

Table 4.3 Design steps for lifetime prediction

Planned structure	Existing structure
• Identification of action S and resistance R	• Investigation of the structure (ascertainment of the loss of durability and existing damages)
• Ascertainment of material performance (laboratory investigations)	• Identification of action S and resistance R (in situ)
• Definition of appropriate deterioration time laws	• Definition of appropriate deterioration time laws
• Statistical quantification of the model parameters	• Statistical quantification of the model parameters
Requirements for quality and lifetime	
• Definition of limit states with regard to safety and economic boundary conditions	
• Definition of the target failure probability p_f and the related target reliability index β	
Statistical and analytical investigations	
• Quantification of the failure probability p_f and reliability index β, respectively, according to the given exposure	
• Assessment of the lifetime of the structure and planning of required maintenance measures	

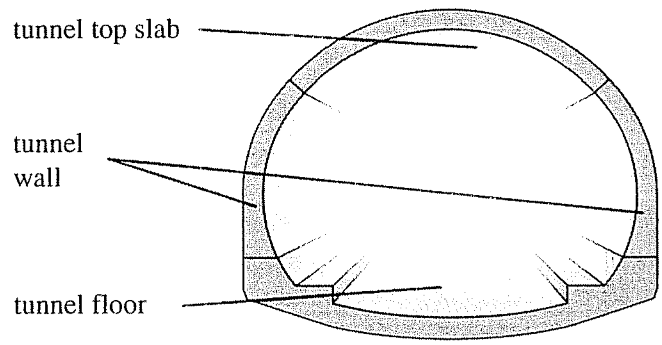


Fig. 4.2 Tunnel structure subdivided in its basic elements

residual lifetime of the structure is determined. Hereby a detailed investigation of the structure is necessary, where non-destructive test methods may help to reduce the costs and to increase the information on the structural status, which in turn improves the accuracy of the prediction of the residual lifetime.

4.4.1 *Service Life Prediction of Structural Components*

4.4.1.1 Planned Structures – Inner Shell of a Tunnel

In this first example, the developed tools for a probabilistic-based performance design concept for the durability behaviour of a concrete structure are applied considering the inner shell of a tunnel (see Fig. 4.2). The focus is on the question how the intended and designed lifetime changes if an insufficient quality management system was realised during the construction process.

Table 4.4 Lifetime depending on concrete cover

	Concrete cover c		Lifetime analysis		
Case	Mean value [mm]	Standard deviation [mm]	Reliability index β [–]	Failure probability p_f [–]	Achievement of the limit state [years]
A	55	8	1.7	5	100
B	55	16	1.3	11	60
C	45	8	1.0	15	62

For a profound lifetime management it is necessary to implement a quality management system already at the stage of design of the structure. Quality management comprises also the inspection of the planned and built structure with regard to the workmanship. This means that after the construction phase the corresponding material or structural parameters – e.g. the concrete strength or the concrete cover – have to be measured. By means of the determined data and based on the lifetime prediction of the structure – the design for durability – a verification of the planned reliability at the end of the structure's service life can be conducted, or, vice versa, a more precise (updated) lifetime prediction is obtained.

In particular, the concrete cover is subjected to several material dependent and production dependent influences. The most important among them are the form and quality of the bar spacer, the form and quality of the formwork, and the placing and compaction of the concrete.

For the inner shell of a tunnel construction – here the tunnel wall, see Fig. 4.2 – the concrete cover in particular is the focus of attention. The concrete cover is an essential parameter for the durability relevant deterioration process of carbonation-induced corrosion (see Sect. 4.3.1). Deviations from the planned cover thickness exert a pronounced effect on the long term durability. This effect is subsequently studied in more detail.

The intended lifetime of the tunnel is assumed to be 100 years. The target value of the reliability index is set to be $\beta = 1.7$. The limit state is defined as the depassivation of the reinforcement of the tunnel wall. Thus, when the carbonation front reaches the reinforcement, the intended lifetime of the wall ends. During the design process the concrete cover – i.e. the design cover thickness – has been specified in view of its mean value and its related standard deviation. By means of non-destructive testing the realised cover and its variation may be determined.

Table 4.4 shows the corresponding parameter study and the results of the reliability analysis, which was performed applying the software STRUREL (RCP GmbH 2003). Case A represents the design situation. At the end of the intended lifetime of the structure, the calculated maximum failure probability – the probability of depassivation – is about 5%.

For case B – which might represent the results of an investigation after the completion of the construction – it is assumed that during the construction process the mean value of the concrete cover was correctly performed, but the intended standard deviation got doubled (from 8 mm to 16 mm) due to poor workmanship. The effect

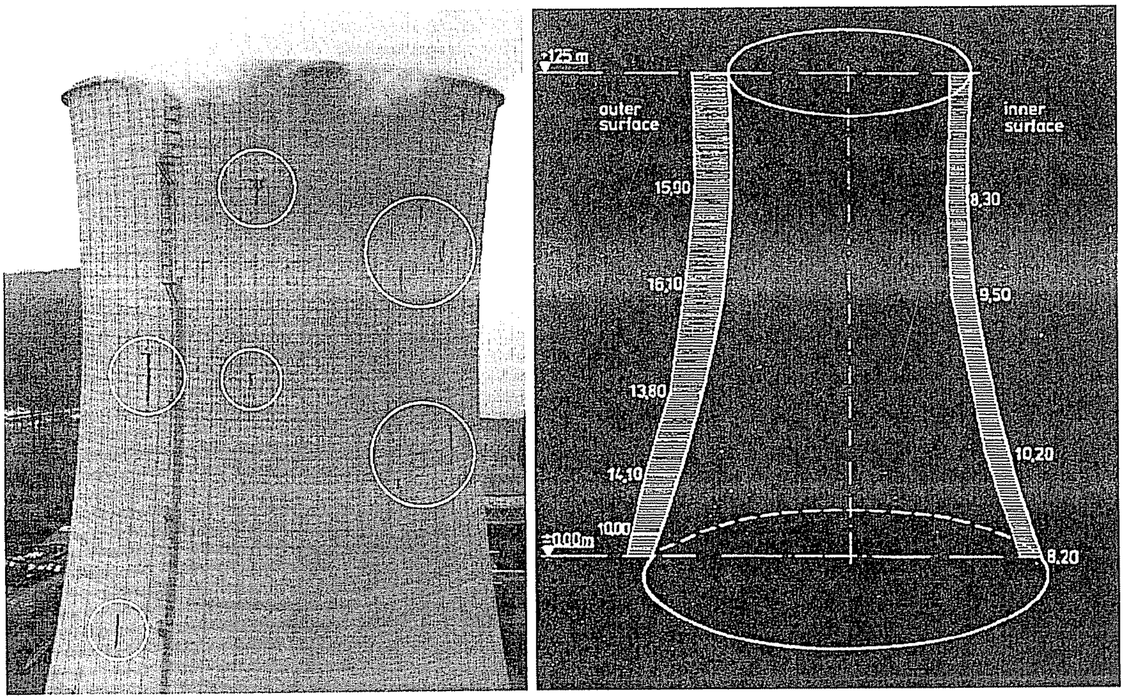


Fig. 4.3 Cooling tower in operation. Shell of the tower with visible rust discolourations (Harte et al. 2006) (*left*) and characteristics of the carbonation depth at the shell in mm (Busch 1991) (*right*)

of this deviation on the lifetime is significant. The probability of depassivation, i.e. the failure probability, is more than doubled (from 5% to 11%), and already after a calculated service life of 60 years appropriate maintenance measures are necessary to avoid further damages. In Case C the workmanship was in accordance with the assumption at the design stage, but a wrong bar spacer was used (mean cover 45 mm instead of 55 mm). For this case the failure probability is tripled compared to the design assumptions. Again, after approx. 60 years of service life, repair measures have to be conducted.

This simple study reveals two main aspects. First, by the application of a probabilistic-based performance concept, deterioration effects are quantified. The designer is not only able to design a structural member for durability, but he is also able to quantify changes in the durability behaviour due to deviations from the planned conditions. Second, it is evident from this study that poor or inaccurate workmanship, which is not identified during initial quality management measures, leads to extensive and expensive repair works.

4.4.1.2 Existing Structure – Cooling Tower

In the following example a cooling tower is considered which is in operation for many years. On the left side of Fig. 4.3, a typical shell of a cooling tower damaged by carbonation-induced corrosion is shown. The white circles mark the areas in

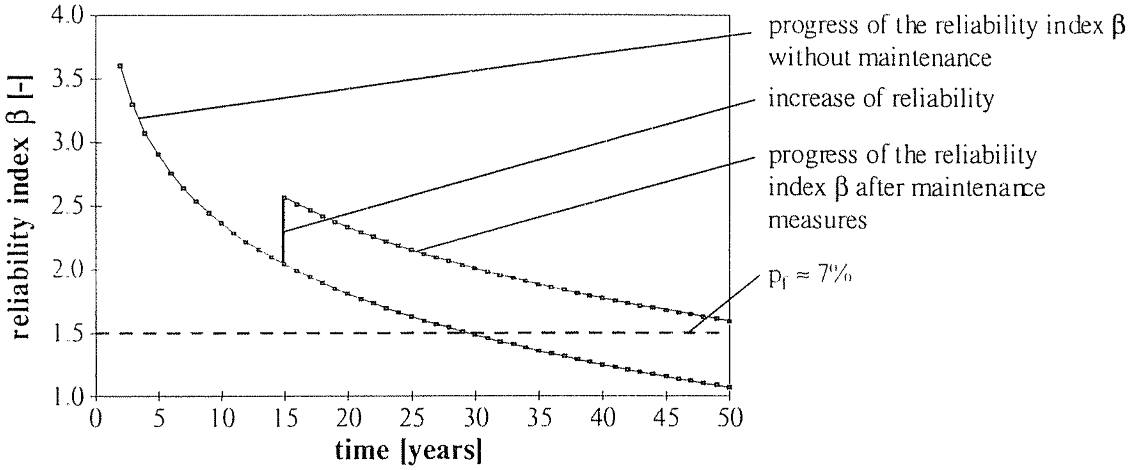


Fig. 4.4 Reliability index β vs. time for calculation "without maintenance" or "with maintenance"

which rust discolourations are visible. The right side in Fig. 4.3 shows schematically the varying carbonation depths in the concrete surface over the height of the cooling tower. It should be noted that especially at cooling towers carbonation depths up to 40 mm were measured after a service life of about 20 years (Harte et al. 2006; Busch 1991). For this structure the residual lifetime should be calculated by means of the probabilistic-based performance concept.

The intended lifetime of the cooling tower is considered to be 50 years. The target value of the reliability index β is set to be 1.5 which corresponds to a failure probability of about 7%. Based on investigations performed prior to this analysis it was found that the concrete cover has a mean thickness of $c = 35$ mm.

The results of the reliability calculations demonstrated in Fig. 4.4 show that the intended reliability index β after a calculated lifetime of 50 years is significantly below 1.5. Already at the age of about 30 years the limit state is reached, which corresponds to the end of the planned lifetime (see Fig. 4.4, lower curve).

For a reconditioning of the damaged shell of the considered cooling tower an appropriate maintenance measure is necessary. This includes the application of repair concrete with a sufficient concrete cover, e.g. $c = 50$ mm. Based on the new boundary conditions with regard to the concrete quality and the concrete cover, a reliability update by means of the Bayesian statistics can be performed (Melchers 2002). The result of this analysis shows that the maximum failure probability does no longer exceed the corresponding probability of the defined limit state after a lifetime period of 50 years. The repair led to a significant improvement of the safety of the structure (see Fig. 4.4, upper curve).

In view of lifetime management of civil structures, it becomes obvious from this example that the necessity of either protective or repair measures can be derived and quantified on the basis of structural investigations so that the intended lifetime may be reached at minimum cost. The decisive advantage of the applied method of probabilistic-based performance design is that a quantitative estimation of protective and maintenance measures is facilitated.

Table 4.5 Design steps for lifetime prediction of structural systems

I	*System analysis*, which includes: – Description of the system – Failure analysis – Fault tree analysis
II	*Failure probability analysis*, considering the individual structural components and the structural system as a whole
III	*Risk assessment*, which includes in particular economical considerations and calculation

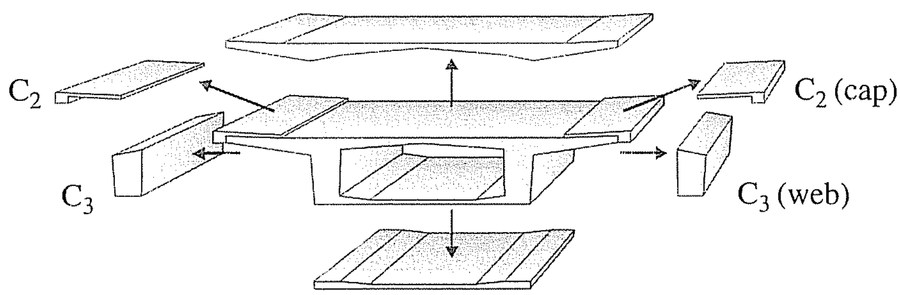


Fig. 4.5 Principle of a component breakdown of a bridge superstructure

4.4.2 Service Life Prediction of Structural Systems

In the previous section the lifetime prediction was performed only for structural components considering single limit states. However, one has to keep in mind that typical civil structures are complex systems. In general, they are composed of numerous structural components which have to satisfy more than one limit state criterion according to the different environmental exposures that stress the structure simultaneously.

In this section a brief introduction into the method of lifetime prediction for complex structural systems is given. Table 4.5 summarises the design steps which have to be considered.

There are three major design steps, namely system analysis, failure probability analysis and risk assessment. In the following paragraphs, system reliability analysis procedures are discussed considering the example of a superstructure of a concrete bridge, which is exposed to several environmental actions (see Fig. 4.5 and Table 4.6).

4.4.2.1 System Analysis

The aim of the system analysis is to understand the function of the structure and to simplify the structure for the reliability analysis. Therefore, it is necessary to describe the system, to analyse the individual failure modes according to the

Table 4.6 Structural components of a bridge superstructure and their exposure

Component	Denotation	Major exposure condition
C_1	Carriageway slab	– Chloride-induced corrosion
C_2	Caps	– Chloride-induced corrosion
		– Frost attack
C_3	Webs	– Carbonation-induced corrosion
		– Frost attack
C_4	Tension/compression chord	– Carbonation-induced corrosion

potential system failure and to perform a fault tree analysis by means of mathematical definitions (Klingmüller and Bourgund 1992). The framework for these design steps is shown in the following.

Description of the System

Within the description of the system it is necessary to identify its main components. Therefore, it is firstly subdivided into its structural components. Figure 4.5 shows the principle of a component breakdown using the example of a bridge superstructure. Here, the main components are the carriageway slab (C_1), the caps (C_2), the webs (C_3) and the tension/compression chord (C_4).

For the component breakdown the appropriate level of detail depends on the given structure itself. It is important to classify the different components according to their function as well as to the different environmental actions, e.g. frost attack. At a later stage, every component of the superstructure has to be assigned to the different exposure conditions which were identified at the structure. Table 4.6 indicates some examples for a reasonable assignment of the exposure conditions (carbonation- or chloride-induced corrosion and frost attack) to the corresponding structural components.

Failure Analysis

The aim of a failure analysis is the identification of the different failure modes of the structural components and their influences on the system. Hence, it is assumed that each component is either in a function state or in a failed state. On this basis, the state of the structure can be expressed in terms of the component functionality.

The building structure usually consists of a large number of components, which are connected in relation to their functions. The interaction of the different components of the structure influences the failure of the systems. The failure mode of one particular component may lead to system failure. For instance, if the carriageway slab fails due to the corrosion of the tendons, the total super-structure of the bridge fails too (see Fig. 4.6).

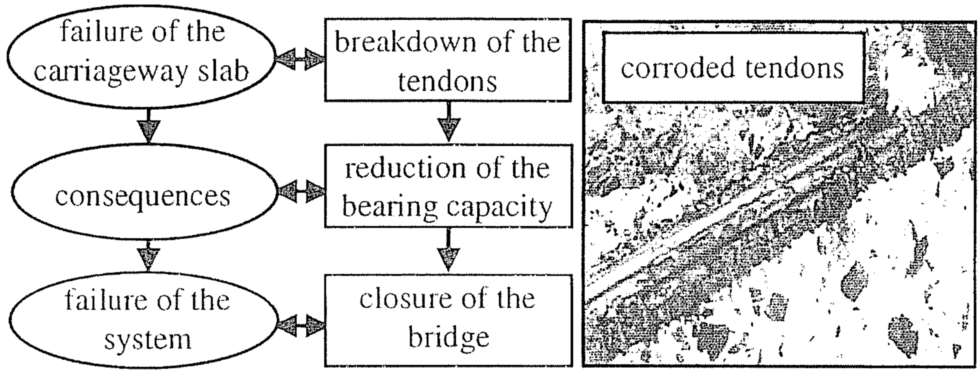


Fig. 4.6 Example of a failure analysis related to a bridge superstructure

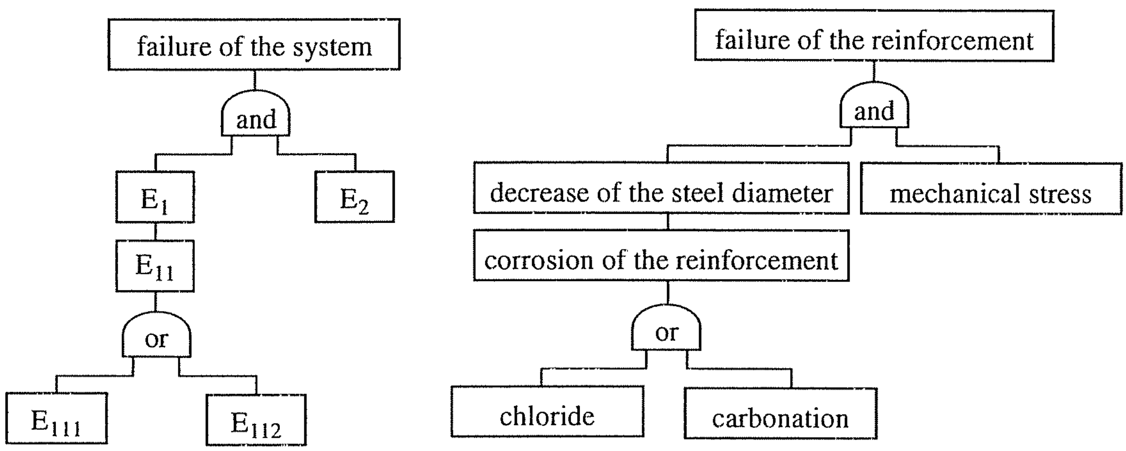


Fig. 4.7 Schema of a fault tree analysis (*left*, E_i = event i) and example of a fault tree according to carbonation- and chloride-induced corrosion (*right*)

Fault Tree Analysis

The fault tree analysis is an analytical method to identify all kinds of events which lead to a "top" event. The top event corresponds to an undesired condition of the structure; hence, it is an adverse event (see Fig. 4.7).

In view of the fault tree analysis there are two basic elementary systems: the series system, termed also weakest link system, and the parallel system, termed also redundant system. A series system fails if any of the system elements fails and a parallel system fails definitively if all elements fail. However, the parallel system does not fail, if just one element does not fail. By means of mathematical rules one can define the lower and upper bounds of the failure probability of the system (Melchers 2002). The simple bounds for the failure probability of a series system can be calculated by means of Eq. 4.4.

$$\max\left[p_{fi}\right] \leq p_{f,series\,syst} \leq 1-\prod_{i=1}^{n}\left(1-p_{fi}\right) < \sum_{i=1}^{n} p_{fi} \tag{4.4}$$

Table 4.7 Results of the reliability analysis carried out for the components of the bridge superstructure

Exposure	Component	Limit state	Time until the limit state is reached (β= 1.7)
E_1: chloride	C_1: carriageway slab	Critical chloride content at the reinforcement is reached	27 years
E_2: frost	C_3: webs	2/3 of the concrete cover is destroyed	35 years
E_3: carbonation	C_4: tension/ compression chord	Depassivation of the reinforcement	29 years

The simple bounds for the failure probability of a parallel system can be calculated using Eq. 4.5.

$$\prod_{i=1}^{n} p_{fi} \leq p_{f,parallel\ syst} \leq \min\left[p_{fi}\right] \tag{4.5}$$

The bounds for the failure probability of the above-mentioned bridge superstructure depend on the statistical dependences of the considered failure events.

4.4.2.2 Failure Probability Analysis

This is the second major design step according to Table 4.5, to be subdivided in the analysis of the individual components and of the system as a whole.

Failure of the Components

The failure probability of the components carriageway slab, webs and tension/compression chord (see Fig. 4.5) of the investigated superstructure is determined considering the relevant exposure conditions chloride- and carbonation-induced corrosion and frost attack (see Table 4.6). For this calculation example, the corresponding deterioration time laws have been taken from the literature (fib 2006; Sentler 1983). The magnitude of the necessary parameters and their statistical characteristics have been taken or derived from The European Union (1998, 2000).

The target reliability index β is set to be 1.7 and the considered lifetime is 80 years. Table 4.7 shows the obtained results of the reliability analysis of the components of the superstructure. If it is assumed that the most severe exposure – here the chloride-induced corrosion – controls the failure behaviour, a maintenance measure of the bridge superstructure is necessary after a service life period of 27 years. On the basis of this lifetime prediction, a significant reduction of the lifetime compared to the intended lifetime of the structure is determined.

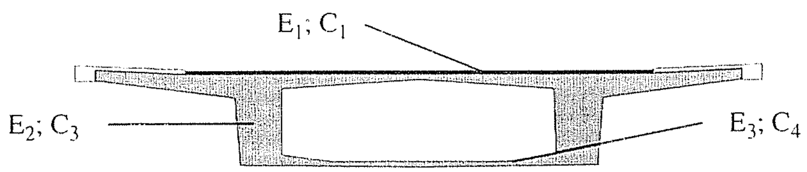


Fig. 4.8 Bridge superstructure with its components and corresponding relevant exposure conditions

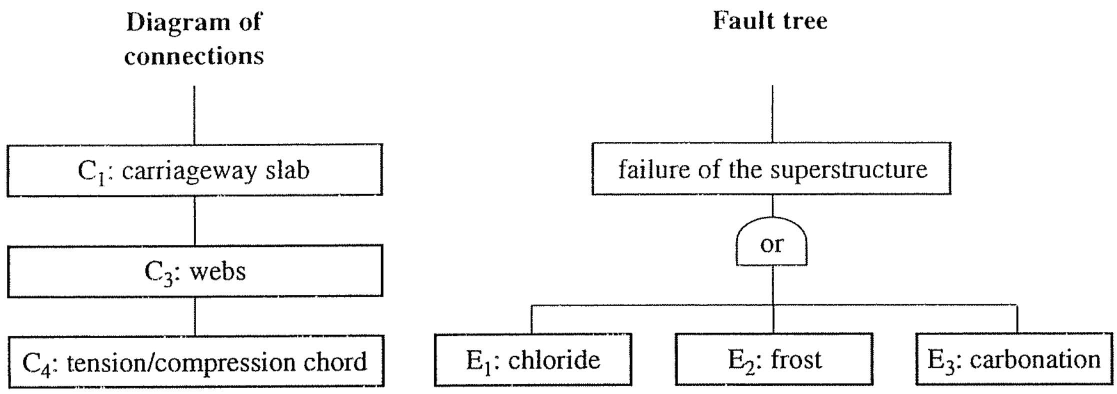


Fig. 4.9 Schema of the series system bridge superstructure

Failure of the System

Figure 4.8 visualises the relevant exposure conditions for the individual components of the bridge superstructure. The chloride-induced corrosion (E_1) is related to the component C_1 (carriageway slab), the frost attack (E_2) is related to the component C_3 (webs) and the carbonation-induced corrosion (E_3) is related to the component C_4 (tension/compression chord).

The superstructure of the bridge represents a series system. As explained above, this system fails – in terms of an undesired condition – when component C_1 or component C_3 or component C_4 fails. Figure 4.9 shows the principle of this series system indicating components and relevant exposure conditions.

The calculated result of the time-dependent system failure probability of the considered series system bridge superstructure with respect to the above-mentioned boundary conditions (see Eqs. 4.4 and 4.5) is shown in Fig. 4.10.

The lower bound curve is the result of the assumption that all failure events are statistically dependent. The upper bound curve is obtained when all failure events are statistically independent. Considering a series system, it should be kept in mind that the system failure probability increases if the correlation between the failure events decreases, since for a decreasing correlation the uncertainties between the failure events are increasing.

In comparison with the results of the lifetime prediction for the individual components (see Table 4.7), the reliability analysis for the system of the superstructure results in a further reduction of the intended lifetime. Hence, the structure has to be repaired

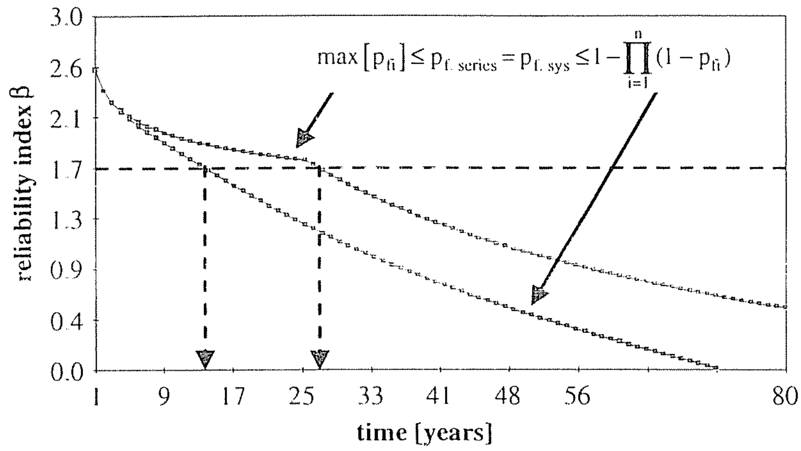


Fig. 4.10 Reliability index β vs. time, determined on the basis of the system reliability investigation of a bridge superstructure

before the calculated lifetime of 27 years; already after 14 years of service life maintenance measures are necessary. This surprising result is based on the fact that the reliability index β is decreasing if the failure events are statistically independent.

4.4.2.3 Risk Assessment

The risk assessment is the third major design step according to Table 4.5. For a risk assessment, the failure probability of a single event $p_{f,i}$ has to be considered in connection with potential consequences c_i, see Eq. 4.6. The potential consequences are usually given in the form of a monetary valuation, e.g. costs due to necessary repair works and corresponding production downtimes. Hence, the risk assessment relates to economical risks.

$$R_{total} = \sum (p_{f,i} \cdot c_i) \tag{4.6}$$

By means of the risk assessment it is possible to identify weak points of the civil concrete structure. On the basis of the risk assessment method, an economical optimisation of the on-site inspections can be achieved. Further, by identifying vulnerable components cost effective protection or repair measures can be undertaken before the occurrence of damages causes high repair costs. It is evident that by means of the risk assessment the lifetime management of civil structures may become very efficient. Material resources, energy and additional expenses may be considerably saved.

4.5 Analysis of Interactions and Singular Risks

The reliability index calculated in the examples beforehand designates the most decisive mechanism. This means that the reliability of a structure can be predicted for carbonation-induced or chloride-induced corrosion or for freeze/thaw attack,

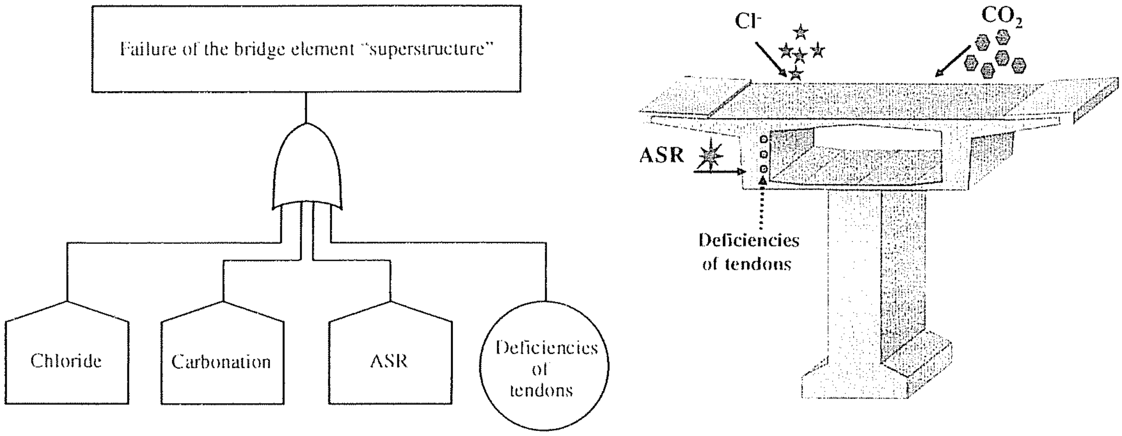


Fig. 4.11 Schema of the series system modelled for a bridge superstructure

considered as physically non-correlated effects. Their interactions with each other are not considered adequately. In practice, however, the exposures occur only in a combined manner (Holt et al. 2009). In contrast to the knowledge about single mechanisms, the knowledge about the impact of combined durability actions on concrete structures is limited to some few investigations. A comprehensive overview of the relevant combined actions does not exist.

Besides, it is even more difficult to quantify durability with these durability-relevant deterioration mechanisms, such as carbonation, chloride ingress and frost/thaw attack, if so-called singular risks occur at the same time. These singular risks can be for example a leaking sealing or cracking due to stress corrosion. The effect of a leaking sealing in durability is considerable but not measurable so far.

Intensive research has shown that the resulting deterioration progress is affected in its intensity by the interactions of various durability actions. This is not only influenced by the intensity of the single deterioration mechanisms, but also by the chronological occurrence of combined exposures, which has an influence on the type of interaction and the resultant deterioration progress, respectively.

The current methods for system and risk analysis do not allow modelling interactions against the background of material science considerations. The characteristic of fault tree analysis is that one only has the choice between a serial ("and") and a parallel ("or") connection of events (see Sect. 4.4.2).

Combined actions can be displayed easily by an "and"-connection, since all actions with their individual failure probability can be incorporated (Müller and Vogel 2008). Interaction may not be introduced by an "and" nor by an "or"-connection of events or mechanisms, respectively.

A suitable way of modelling interactions is the introduction of a dimensionless factor η. By introducing the factor η the material technological characteristics which have an influence on the interactions can be taken into account.

To exemplarily demonstrate the procedure of the new concept for modelling the interactions of combined actions, a reliability analysis was carried out. The following deterioration mechanisms were chosen: carbonation and chloride-induced corrosion, alkali-silica reaction (ASR) and an insufficient grouting of the tendon ducts. Simplifying, it was assumed that ASR and the tendon ducts do not have a significant influence on carbonation and chloride ingress. Figure 4.11 shows the fault tree for

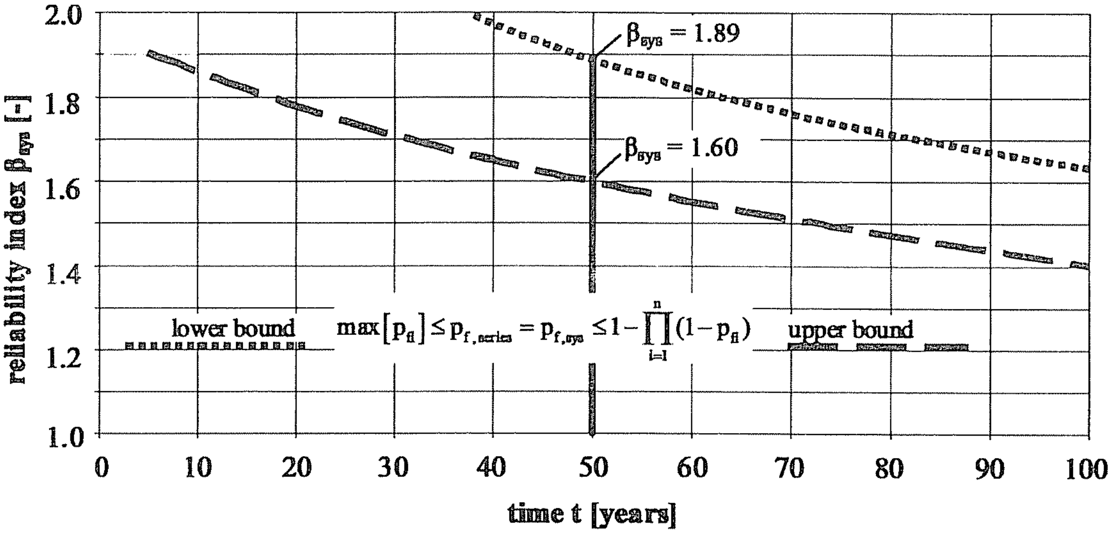


Fig. 4.12 Reliability index β_{sys} vs. time t according to the superstructure of the bridge, see Fig. 4.11

the deterioration of a bridge element superstructure. The superstructure of the bridge represents a series system.

For the actions of carbonation and chloride ingress in this exemplary calculation, the models of the *fib* Model Code 2010 were applied. On this basis, the limit state orientated probabilities were calculated. For the model parameters needed, appropriate values were chosen from the literature (The European Union 1998, 2000; Durable and Reliable Tunnel Structures (DARTS) 2004).

For the deterioration caused by alkali-silica reaction and the insufficient grouting of the tendon ducts, failure probabilities which refer to an example in literature were chosen (Zhu 2008). Therefore, the probability for an alkali-silica reaction was taken as $5 \cdot 10^{-3}$ ($p_f = 0{,}5\%$) and for the insufficient grouting of the tendon ducts the failure probability was assumed to be $2 \cdot 10^{-2}$ ($p_f = 2{,}0\%$).

The prediction of the system failure probability of the bridge structure was performed for a reference period of 100 years. The determination of the time dependent reliability β_{sys} was calculated with the FORM method using the software STRUREL (RCP GmbH 2003). The results are displayed in Fig. 4.12.

For every point in time a range of reliability can be calculated. After 50 years, for example, the reliability index β is expected to be within a range of 1.60 and 1.89. The lower bound indicates the reliability index in case of full correlated failure modes and the upper bound in case of uncorrelated failure modes. The correlation term used in this sense has only a mathematical reason. Material technological correlations or interactions are not considered.

In order to implement the material technological interactions, the factor η_{carbo} is introduced. The fault tree in Fig. 4.13 illustrates descriptively the chosen actions and the meaning of the factor η_{carbo}.

By using the factor η_{carbo} it is possible to take into account an interaction of carbonation and chloride ingress. There are different possible effects an interaction might cause. Due to the carbonation the concrete might have an increased density and

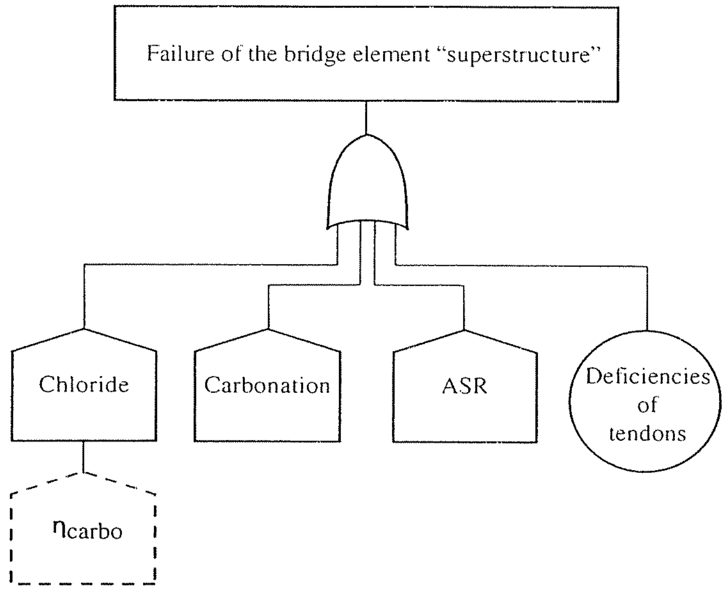


Fig. 4.13 Schema of the series system modelled for a bridge superstructure; the interaction of carbonation and chloride ingress by implementing the factor η_{carbo} is taken into account

lower porosity, which in turn might impede the further ingress of carbonates. On the other hand, the binding capacity of the concrete is lowered due to the carbonation process. The total chloride concentration might be higher than in non-carbonated concrete since the so far bound chlorides are set free. To indicate the influence of carbonation on the chloride ingress, the factor η_{carbo} is annexed to the chloride ingress. The effect simulated can be described simplified with the assumption that the chloride migration coefficient is either increased or decreased. The chronology in which the different actions occur plays an important role within this context, which is not further discussed here.

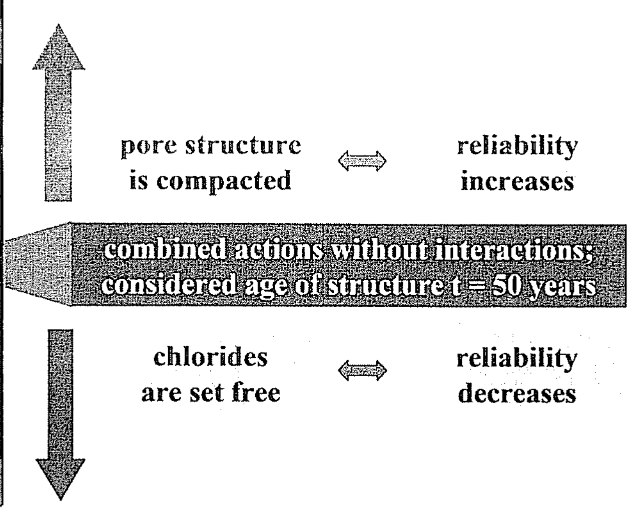
Although in this example the interaction is limited to the influence of carbonation on the chloride ingress, the principle of the procedure may be demonstrated. The deficiencies of the tendons do not have an impact in the chloride ingress, nor does the carbonation. The ASR may be considered for simplicity to be independent from the other accompanying factors. The only two actions which have an influence on each other are the ingress of chlorides and the carbonation of concrete.

In Table 4.8 the varying ranges of the reliability index β_{sys} and the failure probability $p_{f,sys}$ according to the varying factor η_{carbo} can be seen. The parameter study of the factor η_{carbo} was performed considering a lifetime of 50 years.

The influence of an increasing (>1.0) or decreasing (<1.0) factor η_{Carbo} on the development of the reliability is shown. In the example, a change of the chloride diffusion coefficient by an influence of carbonation was simulated with the factor η_{carbo}. If the average chloride diffusion coefficient is increased with a factor η_{carbo} higher than 1.0, a lower reliability is obtained. Correspondingly the reliability increases with a decreasing chloride diffusion coefficient as a result of a factor η_{carbo} lower than 1.0. The deviation of the reliability index β_{η} adds up to ±7%, which might have a noticeable impact on the service, even if interaction factor η_{carbo} changes only moderately.

Table 4.8 Parameter study on probability p_f and reliability index β_η depending on factor η_{carbo}

η_{carbo} [-]	lower bound		upper bound	
	p_f [%]	β_η [-]	p_f [%]	β_η [-]
0.80	2.1	2.03	4.7	1.68
0.84	2.3	2.00	4.8	1.66
0.88	2.5	1.97	5.0	1.65
0.92	2.6	1.94	5.2	1.63
0.96	2.8	1.91	5.3	1.61
1.00	3.0	1.89	5.5	1.60
1.04	3.1	1.86	5.7	1.58
1.08	3.3	1.84	5.8	1.57
1.12	3.5	1.82	6.0	1.55
1.16	3.7	1.79	6.2	1.54
1.20	3.8	1.77	6.4	1.53



4.6 Conclusions and Outlook

The political emphasis which is paid to the sustainable development in all areas of human activities necessitates the introduction of lifetime management for civil structures. Lifetime management will reduce the consumption of material and energy, and thus it will also reduce the total costs for civil structures. These total costs cover not only the costs for construction itself, but also the costs for maintenance and demolition.

While very often in the past only the investment for a building, i.e. the costs for the construction of a building, were considered, new financial concepts, such as PPP (private financing of public buildings, so-called public private partnership) or BOT (a concept where the contractor builds, operates and finally transfers the civil structure) are entering more and more the market. In these concepts, the total costs are taken into account. Hence, not only the political framework but also economical reasons will facilitate the introduction of the lifetime management of civil structures. This is possible as nowadays the aging process of buildings, i.e. the loss of durability with time, is reasonably well understood and can be described by models which are the core elements of lifetime management.

The procedure for durability design as regulated in the national standards (DIN 1045-2 2001) and e.g. in the *fib* Model Code for Service Life Design (fib 2006) is based on the most decisive deterioration mechanism a concrete structure is exposed to. The knowledge and the design methods established so far have to be extended from single to interacting actions and, therefore, to deterioration models which incorporate related effects. First approaches in regard to modelling of interactions have been developed and shown here.

The results of these investigations are not only relevant for the service life design but also for the prediction of the remaining service life of concrete structures.

During the lifetime of a building, investigations of the material and structural behaviour are necessary from time to time in order to improve the prediction accuracy of the models – for single mechanisms as well as for interactions – for the performance behaviour. These investigations may preferably be carried out by means of non-destructive test methods.

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