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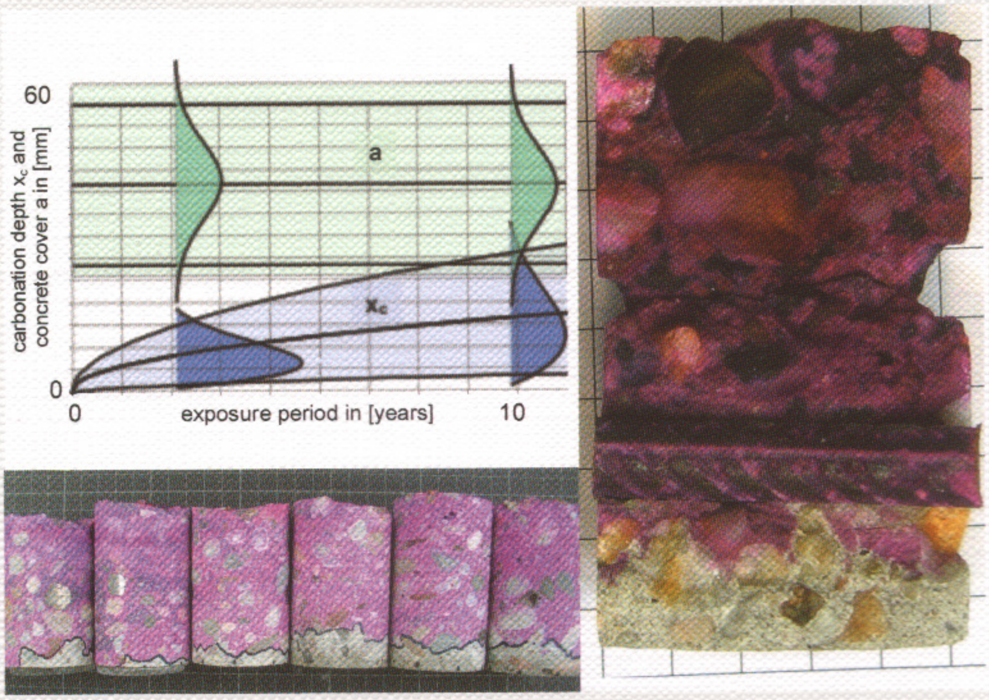

Model Code for Service Life Design

model code

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Model code prepared by

Task Group 5.6

February 2006

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category	minimum approval procedure required prior to publication
Technical Report	approved by a Task Group and the Chairpersons of the Commission
State-of-Art Report	approved by a Commission
Manual or Guide (to good practice)	approved by the Steering Committee of ***fib*** or its Publication Board
Recommendation	approved by the Council of ***fib***
Model Code	approved by the General Assembly of ***fib***

Any publication not having met the above requirements will be clearly identified as preliminary draft.

This Bulletin N° 34 will be submitted to the General Assembly for approval as an ***fib*** Model Code in June 2006.

This report was prepared within Task Group 5.6, *Model code for service life design of concrete structures*:

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Cover images: The photos show the carbonation depth of a vertical concrete surface of an existing building after 8 years of exposure without shelter from rain. A phenolphthalein indicator distinguishes areas with pH < 9.5 (not coloured) and areas with a higher pH (coloured). The graph shows the development of the carbonation depth over time, $x_c(t)$, compared to the cover depth, a. Scatter of both variables is also given.

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Contents

Preface

fib and its preceding organizations, CEB and FIP, have a long tradition in treating durability aspects and to design for them. In 1978 CEB created a first working group, the “Task Group Durability”. Milestones in the CEB and FIP work on durability are CEB Bulletins 148 “Durability of concrete structures”, 182 “Durable concrete structures” and 238 “New approach to durability design”. In the latter document the framework for a probabilistic design approach was set. In 2002 *fib* established Task Group 5.6 “Model code for service life design of concrete structures” with the objective to develop a model code document on probabilistic service life design. The approach developed in this document is intended to be the basis for the service life design approach of the new *fib* Model Code, currently under development. Furthermore it might serve as a basis for further work in ISO (TC 71) and CEN (TC 104 and TC 250/SC2).

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The format of this Model Code follows the CEB-FIP tradition: the main provisions are given on the right-hand side of the page, and on the left-hand side, the comments.

Peter SCHIESSL

Convener of *fib* Task Group 5.6

0 Introduction

The objective of this document is to identify agreed durability related models and to prepare the framework for standardization of performance based design approaches.

This Model Code treats design for environmental actions leading to degradation of concrete and embedded steel.

The basic idea of service life design as presented in this document is to establish a design approach to avoid deterioration caused by environmental action comparable to load design as we are used to have it in our design codes (e.g. EC2). That means quantifiable models on the load side (these are the environmental actions) and on the resistance side (this is the resistance of the concrete against the considered environmental actions). The design approach will be examplified for design against reinforcement corrosion caused by carbonation of concrete without load or restraint induced cracks.

The first step in the design approach is to quantify the deterioration mechanism with realistic models describing the process physically and/or chemically with sufficient accuracy (e.g. ingress of carbonation into the concrete depending on the environment and the relevant concrete quality parameters). Such a model for ingress of carbonation is given in the document. Sufficient accuracy means that the model should be validated by realistic laboratory experiments and by practice observations, so that mean values and scatter of the material resistance parameters are known and can be considered in the model. In the same way models for the environmental actions with statistically quantified environmental parameters (e.g. temperature, relative humidity, splash rain events etc.) need to exist.

The second step is the definition of limit states against the structure should be designed for. Appropriate limit states would be

- depassivation of reinforcement caused by carbonation
- cracking due to reinforcement corrosion
- spalling of concrete cover due to reinforcement corrosion
- collapse due to loss of cross section of the reinforcement.

The third step is the calculation of the probability that the limit states defined above occur (determination of the probability of occurance). This will be done by applying the models described in step 1 above. Nowadays it is commonly accepted that the safety of structures should be expressed in terms of reliability (reliability index β). Depending on the type of limit state (SLS, ULS) and the consequences of a failure, values for β are given in EC 0.

The fourth step is the definition of the type of limit state (SLS, ULS) of the limit states described in step 2. Normally depassivation will be classified as a SLS as there is no immediate consequence on structural safety if the reinforcement is depassivated. Therefore β-values in the range of $\beta = 1.0$ to 1.5 may be appropriate for depassivation. However, the owner may require higher β-values for example to safely ensure the aesthetic quality of the structure. For the limit state cracking and spalling the designer has to decide which type of limit state is needed or should be chosen. If, for example, cracking and spalling occurs in anchorage zones without sufficient transversal reinforcement, spalling may lead to collapse. In this case cracking and spalling need to be defined as ULS. In other cases if cracking and spalling does not influence the load bearing capacity of the structural element, cracking and spalling may be defined as SLS.

The service life design approach in this document is elaborated for three different levels. The full probabilistic approach (level 1) will be used only for exceptional structures. Based on the full probabilistic approach a partial safety factor approach comparable to load design is given. The partial safety factor approach (level 2) is a deterministic approach where the probabilistic nature of the problem (scatter of material resistance and environmental load) is taken into account by partial safety factors. Finally the deemed to satisfy approach (level 3), again derived from the full probabilistic approach is

The Model Code is divided into five chapters:

1. General
2. Basis of design
3. Verification of Service Life Design
4. Execution and its quality control
5. Maintenance and condition control

The flow chart in Figure 1.1-1 illustrates the flow of decisions and the design activities needed in a rational service life design process with a chosen level of reliability. Two strategies have been adopted, whereof the first is introduced of three levels of sophistication. In sum 4 options are available.

Strategy 1:

Level 1. Full probabilistic design approach, (option 1)

Level 2. Partial factor design approach, (option 2) elaborated. This type of approach is comparable to the approach which can be found in the standards nowadays. However descriptive rules of today's standards are not based on physically and chemically correct models but more on practical (sometimes bad) experience. In the future currently applied rules urgently have to be calibrated against the full probabilistic approach.

Another option given in this document is the use of non reactive materials (e.g. stainless steel, strategy 2/option 4).

Other methods or levels between the levels chosen for this document may be appropriate for Service Life Design (e. g. the durability factor method approach, [1]).

Level 3.   Deemed to satisfy design approach, (option 3)

Strategy 2: Avoidance of deterioration design approach, (option 4)

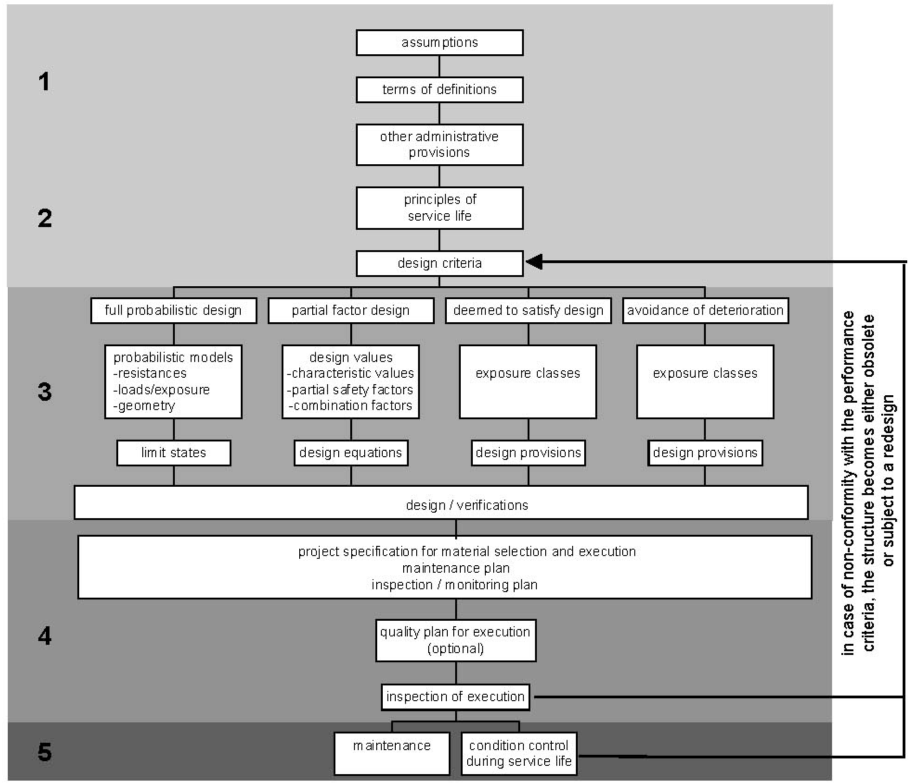

Figure 1.1-1:   Flow chart "service life design"

Within Chapter 3 various deterioration mechanisms are treated:

- carbonation-induced corrosion;:
- chloride-induced corrosion;
- freeze/thaw attack without de-icing agents;
- freeze/thaw attack with de-icing agents.

For these mechanisms broad accepted models exist. Other deterioration mechanisms are not treated, for example alkali silica reaction, and sulfate attack, mainly due to the situation that broad accepted models do not exist so far.

Beside above mentioned mechanisms also fatigue caused by dynamic loading and leading to time dependent material degradation and corrosion fatigue caused by dynamic loading and simultaneous corrosion caused by environmental action is not treated.

Simultaneous dynamic loading and corrosion of steel e. g. in the region of load or restraint induced cracks, will lead to a reduction in the fatigue resistance. The S-N-curves as the basis for fatigue design may be up to 50 % lower related to the stress range compared to S-N-curves of reinforcement without corrosion attack.

To make this document complete, missing models have to be developed which have to respect the general principles of Chapter 2.

The service life design approach described in this document may be applied for the design of new structures, for the update of the service life design if the structure exists and real material properties and/or the interaction of environment and structure can be measured (real concrete covers, carbonation depths) and for the calculation of the residual service life.

Attached to the MC-SLD are 4 informative annexes. These are giving background information as well as examples of procedures and deterioration models for the application in SLD. Other sufficiently validated procedures for reliability management and models for deterioration might be used.

1 General

1.1 Scope

(1) The present Model Code is applicable for Service Life Design (SLD) of plain concrete, reinforced concrete and pre-stressed concrete structures with a special focus on design provisions for managing the adverse effects of degradation. The Model Code provides the basis for service life design of concrete structures. Four different options are offered:

- a full probabilistic approach
- a semi probabilistic approach (partial factor design)
- deemed to satisfy rules
- avoidance of deterioration

The methodology described in this document might also be applied for assessment of remaining service life of existing structures.

Traditionally, national and international concrete standards give requirements to achieve the desired design service life based on the “deemed-to-satisfy” and the “avoidance of deterioration” approach.

Such operative requirements have to be calibrated by the responsible standardization body. This document gives guidance for such calibration.

1.2 Associated codes

(1) The present code is applicable as described under 1.1 together with

- CEN Eurocode 0 (EN 1990:2002) ”Basis for design”
- “Probabilistic Model Code”, Joint Committee on Structural Safety (JCSS PMC:2000), www.jcss.eth.ch
- CEN ENV 13670-1:2000 “Execution of concrete structures”
- ISO 2394:1998 (E), ”General principles on reliability for structures”

CEN EN 1990 “Basis for design” is based on the general principles for the verification of the reliability of structures given in ISO 2394:1998 “General principles on reliability for structures”

CEN ENV 13670-1 is presently the main reference document for ISO TC-71/SC3 when drafting an international standard for the execution of concrete structures.

This CEN standard might be replaced by the coming EN 13670, or by the ISO document when available, or with the execution provisions in the next version of the *fib* Model Code.

1.3 Assumptions

(1) In addition to the general assumptions of EN 1990 the following assumptions apply:

- Structures are designed by appropriately qualified and experienced personnel.
- Adequate supervision and quality control is provided in factories, in plants and on site.

The execution standard assumes that the construction materials brought to the building site comply with relevant product standards defining their minimum performances.

In this respect a “nominal value” means a value fixed on non-statistical bases, for instance on acquired experience or on physical conditions.

– Construction is carried out by personnel having the appropriate skill and experience.
– The construction materials and products are used as specified in the relevant material or product specifications.
– The structure will be adequately maintained according to the options given in this document.
– The structure will be used in accordance with the design brief.
– The minimum requirements for execution and workmanship given in ENV 13670 are complied with.

1.4 Definitions

(1) The terms and definitions given in EN 1990 apply with the following amendments:

1.4.1 Basic variable[1) 8)]

part of a specified set of variables representing physical quantities, which characterise actions and environmental influences, geometrical quantities, and material properties.

1.4.2 Characteristic value (X_k or R_k) [2)]

value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series. This value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product. A nominal value is used as the characteristic value in some circumstance.

1.4.3 Characteristic value of a geometrical property (a_k)[2) 8)]

value usually corresponding to the dimensions specified in the design. Where relevant, values of geometrical quantities may correspond to some prescribed fractiles of the statistical distribution.

1.4.4 Characteristic value of an action (F_k)[2) 8)]

principal representative value of an action.

1.4.5 Design criteria[2)]

quantitative formulations that describe for each limit state the conditions to be fulfilled.

1.4.6 Design service life[3) 8)]

This document applies the term "Design Service Life". The meaning of this term is equivalent to the term "Design working life" as used by CEN.

assumed period for which a structure or a part of it is to be used for its intended purpose.

1.4.7 Design situations[1) 8)]

sets of physical conditions representing the expected conditions occurring during a certain time interval for which the design will demonstrate that the relevant limit states are not exceeded.

1.4.8 Design value of a geometrical property (a_d)[1) 8)]

generally a nominal value. Where relevant, values of geometrical quantities may correspond to some prescribed fractile of the statistical distribution.

Note: The design value of a geometrical property is generally equal to the characteristic value. However, it may be treated differently in cases where the limit state under consideration is very sensitive to the value of the geometrical property. Alternatively, it can be established from a statistical basis, with a value corresponding to a more appropriate fractile (e.g. rarer value) than applies to the characteristic value.

1.4.9 Design value of an action (F_d)[2) 9)]

value obtained by multiplying the representative value by the partial factor γ_f.

Note: The product of the representative value multiplied by the partial factor $\gamma_F = \gamma_{Sd} \cdot \gamma_f$ may also be designated as the design value of the action (See EN 1990 – 6.3.2)

1.4.10 Design value of material or product property (X_d or R_d)[2) 9)]

value obtained by dividing the characteristic value by a partial factor γ_m or γ_M , or, in special circumstances, by direct determination.

1.4.11 Inspection[4)]

conformity evaluation by observation and judgement accompanied as appropriate by measurement, testing or gauging.

1.4.12 Irreversible serviceability limit states[2) 8)]

serviceability limit states where some consequences of actions exceeding the specified service requirements will remain when the actions are removed.

1.4.13 Limit states[2) 8)]

states beyond which the structure no longer fulfils the relevant design criteria.

1.4.14 Maintenance[5)]

set of activities that are planned to take place during the service life of the structure in order to fulfil the requirements for reliability.

1.4.15 Project specification[7)]

documents covering technical data and requirements for materials, execution, maintenance and condition control for a particular project prepared to supplement and qualify the requirements of general standards.

1.4.16 Reference period [2) 8)]

chosen period of time that is used as a basis for assessing statistically variable actions, and possibly for accidental actions.

1.4.17 Reliability [1) 8)]

ability of a structure or a structural member to fulfil the specified requirements, including the design service life, for which it has been designed. Reliability is usually expressed in probabilistic terms.

Note: Reliability covers safety, serviceability and durability of a structure.

1.4.18 Reliability differentiation[2)]

measures intended for socio-economic optimisation of the resources to be used to build construction works, taking into account all expected consequences of failures and the cost of the construction works.

1.4.19 Repair[2)]

activities performed to preserve or to restore the function of a structure that fall outside the definition of maintenance.

1.4.20 Representative value of an action (F_{rep})[2) 8)]

value used for the verification of a limit state. A representative value may be the characteristic value (F_k) or an accompanying value (ψF_k).

Note: The accompanying value of a variable action may be the combination value, the frequent value or the quasi permanent value.

1.4.21 Resistance[1)]

capacity of a member or component, or a cross-section of a member or component of a structure, to withstand actions due to deterioration.

1.4.22 Serviceability limit states (SLS)[2) 9)]

states that correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met.

SLS is this document only treated in its narrow sense, i.e. durability related limit states, and not in its general wider sense, e.g. to cover deflection.

SLS might be associated with any durability related condition beyond which the owner feels uncomfortable and which are included in the design criteria.

1.4.23 Serviceability criterion[2)]

design criterion for a serviceability limit state.

1.4.24 Ultimate limit state (ULS)[2) 9)]

states associated with collapse or with other similar forms of structural failure

Note: They generally correspond to the maximum load-carrying resistance of a structure or structural member

[1)] The definition is based on that in EN 1990

[2)] The definition is identical to that in EN 1990

[3)] CEN documents are using the term “Design working life” where this document is applying “Design service life”

[4)] The definition is identical to that in ISO 9000

[5)] Based on ISO 15686-1:2000 “Building and construction assets – Service life planning, Part 1: General principles” clause 6.7

[6)] The definition is in accordance with JCSS “Probabilistic Model Code – Part 1”

[7)] Based on CEN ENV 13670-1

[8)] The definition is based on that in ISO 2394

[9)] The definition is identical to that in ISO 2394

1.5 Symbols

(1) For the purpose of this document, the following symbols apply:

F	Action
F_d	Design value of action
R	Resistance
SLS	Serviceability limit state
ULS	Ultimate limit state
a	Distance, age exponent
t	Thickness, time being considered

γ	Partial factor
γ_c	Partial factor for concrete
γ_f	Partial factor for actions without taking account of model uncertainties
γ_F	Partial factor for action, also accounting for model uncertainties and dimensional variations
γ_m	Partial factors for material property, taking account only of uncertainties in the material property
γ_M	Partial factors for material property, taking account of uncertainties in the material property itself and in the design model used
γ_{Sd}	Partial factor associated with the uncertainty of the action and/or action effect model
γ_{Rd}	Partial factor associated with the uncertainty of the resistance model, plus geometric deviations if these are not modelled explicitly

2 Basis of design

2.1 Requirements

2.1.1 Basic requirements

(1) The SLD of concrete structures shall be in accordance with the general rules given in EN 1990.

(2) The supplementary provisions for concrete structures given in this document shall also be applied.

(3) The basic requirements of EN 1990 Section 2 are deemed to be satisfied for concrete structures when SLD is carried out according to the requirements given in section 2.1.2 (2).

2.1.2 Reliability management

(1) Reliability management shall follow the rules given in EN 1990 Section 2.

(2) The service life design shall either:

– follow the general principles for probabilistic service life design of concrete structures outlined in the JCSS PMC, ISO 2394:1998 (E), respectively.

– use the partial factor method given in this document

– use the deemed-to-satisfy method given in this document

– be based on the avoidance-of-deterioration method given in this document

2.1.3 Design service life, durability and quality management

(1) The rules for design of service life, durability and quality management are given in EN 1990 Section 2.

(2) The design service life is the assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance

but without major repair being necessary.

The design service life is defined by:

– A definition of the relevant limit state
– A number of years
– A level of reliability for not passing the limit state during this period

(3) Durability of the structure in its environment shall be such that it remains fit for use during its design service life. This requirement can be considered in one, or a combination, of the following ways:

– By designing protective and omitigating systems
– By using materials that, if well maintained, will not degenerate during the design service life
– By giving such dimensions that deterioration during the design service life is compensated
– By choosing a shorter lifetime for structural elements, which may be replaced one or more times during the design life

in combination with appropriate inspection at fixed or condition dependant intervals and appropriate maintenance activities.

In all cases the reliability requirements for long and short-term periods should be met.

(4) The serviceability criteria shall be specified for each project and agreed with the client.

Guidance for the choice of serviceability criteria combined with appropriate target values of reliability are given in Annex A.

The “Consequence classes”, “Reliability classes” and “Design supervision levels” are identical to those defined in Annex B of EN 1990, while the “Inspection levels during execution” of EN 1990 are only one element in the “Execution classes” defined in the present document.

(5) As a guidance to reliability differentiation, Annex A to this document defines the following general classifications:

– Consequence class CC3, CC2 and CC1
– Reliability class RC3, RC2 and RC1
– Design supervision level DSL3, DSL2 and DSL1
– Execution class EXC1, EXC2 and EXC3

- Robustness Class ROC1, ROC2 and ROC3

For service life design, Annex A, in addition, classify 4 levels of condition control during the service life:

- CCL3, CCL2, CCL1 and CCL0

2.2 Principles of limit state design

(1) The rules for limit state design are given in EN 1990 Section 3.

The performance of a whole structure or part of it should be described with reference to a specified set of limit states and associated levels of reliability which separate desired states of the structure from undesired states.

It shall be verified that none of these limit states are exceeded with a less degree of reliability than given in the design criteria.

The definitions of SLS and ULS are given in 1.4.22 and 1.4.24.

SLS represents all limit states except that associated with collapse or other similar forms of structural failure.

Examples of limit states associated with SLS and dealt with in this document might be: depassivation of reinforcement, cracking, spalling of cover, erosion of surface due to freeze-thaw, etc.

2.3 Basic variables

2.3.1 Actions and environmental influences

(1) Actions specific to SLD are given in relevant sections.

Rules for actions and environmental influences are also given in EN 1990, Section 4.

Characteristic values of actions for use in SLD shall either be

- based on data derived for the particular project or
- from general field-experience
- from relevant literature

Other actions, when relevant, shall be defined in the design specification for a particular project.

2.3.2 Material and product properties

(1) The rules for material and product properties are given in EN 1990 Section 4.

(2) Characteristic values of materials and product properties for use in SLD shall either be

- based on data derived for the particular project or
- from general field-experience
- from relevant literature

Materials and product properties to be determined will depend on the deterioration model used. If different models with different basic assumptions are offered, a checking process should be established, to avoid an incorrect mixture of data.

(3) Material property values shall be determined from test procedures performed under specified conditions. A conversion factor shall be applied, when necessary, to convert the test results of laboratory cast specimens into values, which can be assumed to represent the behaviour of the material or product in the structure.

2.3.3 Geometric data

(1) The rules for geometrical data are given in EN 1990 Section 4.

(2) Design values of geometrical data for SLD shall be in accordance with EN 1990 clause 6.3.4 or according to measurements on the completed structure or element.

Of particular relevance for service life design (SLD) are ENV 13670-1 clause 10.6, figure 3 b and 3 d concerning location of ordinary and prestressed reinforcement.

(3) ENV 13670-1 “Execution of concrete structures” specifies permitted geometrical deviations. If the design assumes stricter tolerances, the design assumptions shall be verified by measurements on the completed structure or element.

For practical reasons, a simplified statistical approach based on “maximum permitted deviation” is often used in project specifications. This is often the case for the concrete cover to reinforcement. This is normally given as a nominal value (target value) and maximum permitted minus and plus deviations.

When performing a full probabilistic SLD, this maximum permitted deviation has to be transformed to a given fractile of an assumed statistical distribution (see clause 4.5 (2)).

Material parameters derived from accelerated short-time tests might have an inherent uncertainty concerning their application for long-term modelling.

The relevance of such material characteristics should therefore be calibrated to long-term infield performance.

The uncertainty of models and parameters will normally influence the result of the SLD to a greater degree when used for design of new structures than when assessing remaining service life of existing structures.

2.4 Verification

2.4.1 Verification by full probabilistic method

(1) The general principles for probabilistic service life design of concrete structures outlined in the JCSS PMC shall be followed.

In particular the following four principles shall be considered:

– Probabilistic models shall be applied that are sufficiently validated to give realistic and representative results.

– The parameters of the models applied and their associated uncertainty shall be quantifiable by means of tests, observations and/or experience.

– Reproducible and relevant test methods shall be available to assess the action- and material-parameters.

Uncertainties associated with models and test methods shall be considered.

2.4.2 Verification by the partial factor method

(1) The rules for the partial factor method are given in EN 1990 Section 6 and can be used for SLD without the limitations given in EN 1990 clause 6.2.

(2) The same models as for the full probabilistic method, based on design values, shall be used for the partial factor method. Simplifications on the safe side are possible.

(3) The partial factor format separates the treatment of uncertainties and variabilities originating from various causes. In the verification procedure defined in this document the design values of the fundamental basic variables are expressed as follows:

– Design values of actions are generally expressed as

$$F_d = \gamma_f \cdot F_{rep} \tag{2.4-1}$$

where F_{rep} are representative values of action

γ_f are partial safety factors

- Design values of material or product property are generally expressed as

$$R_d = R_k / \gamma_m \qquad (2.4\text{-}2)$$

Or, in case uncertainty in the design model is taken into account by:

$$R_d = R_k/\gamma_M = R_k/(\gamma_m \cdot \gamma_{Rd}) \qquad (2.4\text{-}3)$$

where R_k are characteristic values of resistance

γ_m is the partial factor for material property

γ_{Rd} is the partial factor associated with the uncertainty of the resistance model plus geometric deviations if these are not modelled explicitly.

$\gamma_M = \gamma_m \cdot \gamma_{Rd}$ is the partial factor for material property also accounting for the model uncertainties and dimensional variations.

- Design values of geometrical quantities to be considered as fundamental basic variables are generally directly expressed by their design values a_d.

The target reliability level used for the calibration shall be in accordance with Chapter 2.1.3. (4)

(4) When using the partial factor method, it shall be verified that the target reliability for not passing the relevant limit state during the design service life is not exceeded when design values for actions or effects of actions and resistance are used in the design models.

The partial factors shall take into account:

- The possibility of unfavourable deviations of action values from the representative values
- The possibility of unfavourable deviations of materials and product properties from the representative values
- Model uncertainties and dimensional variations

The numerical values for the partial factors shall be determined in either of two ways:

- On the basis of statistical evaluation of experimental data and field observations according to requirements of clause "Verification by full probabilistic method”

– On the basis of calibration to a long term experience of building tradition

2.4.3 Verification by the deemed-to-satisfy method

(1) The deemed-to-satisfy method is a set of rules for

– dimensioning,
– material and product selection and
– execution procedures

that ensures that the target reliability for not passing the relevant limit state during the design service life is not exceeded when the concrete structure or component is exposed to the design situations.

Exposure conditions in the design situations might be classified in ”exposure classes”.

Traditionally, deemed-to-satisfy provisions include requirements to the workmanship, concrete composition, possible air entrainment, cover thickness to the reinforcement, crack width limitations and curing of the concrete.

However, other provisions might also be relevant.

Examples of the calibration of deemed-to-satisfy criteria based on a “close-to” full probabilistic method and data derived from 10 – 15 years old structures are given in [2].

(2) The specific requirements for design, materials selection and execution for the deemed-to-satisfy method shall be determined in either of two ways:

– On the basis of statistical evaluation of experimental data and field observations according to requirements of clause “Verification by full probabilistic method”
– On the basis of calibration to a long term experience of building tradition

The limitations to the validity of the provisions, e.g. the range of cement types covered by the calibration, shall be clearly stated.

2.4.4 Verification by the avoidance-of-deterioration method

(1) The avoidance-of-deterioration method implies that deterioration process will not take place due to for instance:

– Separation of the environmental action from the structure or component by e.g. cladding or membranes
– Using non-reacting materials, e.g. certain stainless steels or alkali-non-

reactive aggregates

- Separation of reactants, e.g. keeping the structure or component below a critical degree of moisture.
- Suppressing the harmful reaction e.g. by electrochemical methods

(2) The specific requirements for design, materials selection and execution for the avoidance-of-deterioration method can in principle be determined in the same way as for the deemed-to-satisfy method.

The limitations to the validity of the provisions shall be clearly stated.

The assumed effectiveness of the actual concept shall be documented, for instance for products by complying with relevant minimum requirements in product standards.

3 Verification of Service Life Design

3.1 Carbonation induced corrosion – uncracked concrete

3.1.1 Full probabilistic method

3.1.1.1 Limit state: depassivation

(1) The following requirement needs to be fulfilled:

$$p\{\} = p_{dep.} = p\{a - x_c(t_{SL}) < 0\} < p_0 \qquad (3.1\text{-}1)$$

$p\{\}$:	probability that depassivation occurs
a:	concrete cover [mm]
$x_c\,(t_{SL})$:	carbonation depth at the time t_{SL} [mm]
t_{SL}:	design service life [years]
p_0:	target failure probability, cp. Annex A, Table A2-2

To get corrosion an environment that is wet enough is needed. For structural elements solely exposed to relative dry indoor environment, a limit state 'depassivation' may not be relevant as no significant corrosion will develop.

(2) The variables a and $x_c(t_{SL})$ need to be quantified in a full probabilistic approach.

(3) To exemplify the design procedure and the quantification of above given quantities, an applicable design method is given in Chapter B1, Annex B. Other methods may be used, provided that the basic principles formulated in Chapter 2.4.1 are fulfilled.

3.1.1.2 Limit states: corrosion-induced cracking, spalling and collapse

(1) Exemplified with regard to cracking, the following basic limit state function needs to be fulfilled:

$$p\{\} = p_{crack} = p\{\Delta r_{(R)} - \Delta r_{(S)}(t_{SL}) < 0\} < p_0 \qquad (3.1\text{-}2)$$

Reinforcement corrosion leading to cracking, spalling and collapse depend to a high extent on the environment at the concrete surface. The micro environment may vary considerable along the concrete surface of structural elements. Most unfavourable micro environmental conditions are frequent wetting and drying and/or accumulation of aggressive agents (chlorides originating from seawater or de-icing salts). Macro-cell corrosion effects may trigger high corrosion rates in areas with less severe micro environmental condition. For given degrees of corrosion the risk for cracking and spalling depends on the geometry of the cross section. Most vulnerable cross sectional areas, e. g. the edges of beams, should be chosen as decisive for design.

First approaches exist to quantify the variables $\Delta r_{(S)}(t_{SL})$ and $\Delta r_{(R)}$. Most of the corresponding models are empirically derived, often based on very limited, in consequence insufficient data basis. The correlation between corrosion rates/concrete quality/micro environment is not yet quantified in detail. The same applies to the limit states spalling and collapse. To get first impressions on the propagation period TG 5.6 organised a Delphic oracle. One result of the exposure dependent output of this Delphic oracle is given in Annex R. Together with existing models describing the initiation period and the herewith overall quantified propagation period, fully-probabilistic calculations with regard to corrosion induced cracking, spalling and collapse of concrete structures can be performed, see Equation 3.1-3.

$p\{\}$: probability that carbonation-induced cracking occurs

$\Delta r_{(R)}$: maximal corrosion induced increase of the rebar radius which can be accommodated by the concrete without formation of cracks at the concrete surface [μm]

$\Delta r_{(S)}(t_{SL})$: increase of the rebar radius due to reinforcement corrosion [μm]

t_{SL}: design service life [years]

p_0: target failure probability, cp. Annex A, Table A2-2

An alternative design approach is:

$$p\{\} = p_{crack} = p\{t_{SL} - t_{ini} - t_{prop} < 0\} < p_0 \qquad (3.1\text{-}3)$$

$p\{\}$: probability that carbonation-induced cracking occurs

t_{SL}: design service life [years]

t_{ini}: initiation period [years]

t_{prop}: propagation period [years]

p_0: target failure probability, cp. Annex A, Table A2-2

(2) The variables $\Delta r_{(R)}$ and $\Delta r_{(S)}(t_{SL})$ or the variables t_{ini} and t_{prop} need to be quantified in a full probabilistic approach.

(3) To exemplify the design procedure and the quantification of above given quantities, an applicable design method is given in Annex R. Other methods may be used, provided that the basic principles formulated in Chapter 2.4.1 are fulfilled.

3.1.2 Partial factor method

3.1.2.1 Limit state: depassivation

(1) The following limit state function needs to be fulfilled:

$$a_d - x_{c,d}(t_{SL}) \geq 0 \qquad (3.1\text{-}4)$$

a_d: design value of the concrete cover [mm]

$x_{c,d}(t_{SL})$: design value of the carbonation depth at time t_{SL} [mm]

(2) The design value of the concrete cover ad is calculated as follows:

$$a_d = a_k - \Delta a \qquad (3.1\text{-}5)$$

a_k: characteristic value of the concrete cover [mm]

Δa: safety margin of the concrete cover [mm]

(3) The design value of the carbonation depth at a time t_{SL} $x_{c,d}(t_{SL})$ is calculated as follows:

$$x_{c,d}(t_{SL}) = x_{c,c}(t_{SL}) \cdot \gamma_f \qquad (3.1\text{-}6)$$

$x_{c,c}(t_{SL})$: characteristic value of the carbonation depth at a time t_{SL} [mm], e.g. mean value of the carbonation depth

γ_f: partial safety factor of the carbonation depth [-]

(4) To exemplify the design procedure and the quantification of above given quantities, an applicable design method is given in Annex C. Other methods may be used, provided that the basic principles formulated in Chapter 2.4.2 are fulfilled.

Basic requirements with regard to cover, CO_2-diffusion and binding characteristics as well as executional requirements will be given comparable as already given in EC 2 (cover: c_{min}, EC 2, Table 4.4 and 4.5; diffusion and binding characteristics: indirectly by strength class, minimum requirements with regard to concrete composition, EC 2, Table E.1N; execution….).

3.1.3 Deemed-to-satisfy method

(1) Within this approach a trading-off of geometrical (concrete cover), material (diffusion and binding characteristics) and executional (curing) variables can be established.

3.1.4 Avoidance-of-deterioration method

(1) Generally, avoidance is achieved if depassivation cannot take place due to infinite material resistance or zero environmental load.

3.2 Chloride induced corrosion – uncracked concrete

3.2.1 Full probabilistic method

3.2.1.1 Limit state: depassivation

(1) The following limit state function needs to be fulfilled:

$$p\{\} = p_{dep.} = p\{C_{Crit.} - C(a,t_{SL}) < 0\} < p_0 \qquad (3.2\text{-}1)$$

$p\{\}$: probability that depassivation occurs

$C_{Crit.}$: critical chloride content [wt.-%/binder content]

$C(a,t_{SL})$ chloride content at depth a and time t [wt.-%/binder content]

a: concrete cover [mm]

t_{SL}: design service life [years]

p_0: target failure probability, cp. Annex A, Table A2-2

(2) The variables a, $C_{crit.}$ and $C(a,t_{SL})$ need to be quantified in a full probabilistic approach.

(3) To exemplify the design procedure and the quantification of above given quantities, an applicable design method is given in Chapter B2, Annex B2. Other methods may be used, provided that the basic principles formulated in Chapter 2.4.1 are fulfilled.

3.2.1.2 Limit states: corrosion-induced cracking, spalling and collapse

See Chapter 3.1.1.2

3.2.2 Partial factor method

See Chapter 3.1.2

3.2.3 Deemed-to-satisfy method

See Chapter 3.1.3

3.2.4 Avoidance-of-deterioration method

See Chapter 3.1.4

3.3 Influence of cracks upon reinforcement corrosion

(1) The minimum structural reliability of a cracked reinforced concrete structure has to be of comparable magnitude as the minimum reliability of a comparable exposed uncracked structure.

The corrosion rates in the region of cracks crossing the reinforcement are extremely dependent on the micro climatic conditions at the concrete surface and the orientation of the concrete surface. Most severe conditions occur in case of horizontal concrete surfaces and both cracks and chloride attack from the top. For usual service lives more than 10 years and frequent chloride attack (e. g. parking decks in regions where de-icing salts are used) special protective measures are necessary to avoid the rapid penetration of chlorides to the reinforcement (e. g. linings or crack-bridging coatings). In case of vertical surfaces and horizontal surfaces with chloride spray from the bottom side and chloride containing water not leaking through cracks high quality of concrete cover (cover thickness ≥ 50 mm, low permeability concrete, $w/c \leq 0.5$) and ordinary crack width limitation ($w_{k,cal} \leq 0.3$ mm) ensures sufficiently long service life (≥ 50 years) without extra protection.

In case of carbonation induced corrosion adequate quality of concrete cover and ordinary crack width limitation ensures sufficiently long service life (≥ 50 years) without extra protection.

(2) Similar to the procedure given in Chapters 3.1 and 3.2, unwanted events with regard to serviceability/ functionality have to be identified (SLS). In addition, it has to be checked whether ultimate limits are affected by continuously corroding reinforcement within the cracked zone or not.

(3) If functionality is affected, an avoidance of deterioration approach is recommended.

(4) If structural integrity is affected, an avoidance of deterioration approach have to be applied.

3.4 Risk of depassivation with respect to pre-stressed steel

fib Bulletin 33 "Durability of post-tensioning tendons" (December 2005) describes multi-barrier systems for the protection of pre-stressing systems. These systems are supposed to satisfy the design criteria with ample margin and might be classified according to Chapter 2.4 as between the "deemed-to-satisfy" and the "avoidance-of-deterioration" method.

(1) Apply relevant application rules given in Chapters 3.1, 3.2 and 3.3 and avoid depassivation of pre-stressed steel on an ULS reliability level, cp. Annex A, Table A2-2.

3.5 Freeze/thaw attack –without de-icing agents

3.5.1 Full probabilistic method

3.5.1.1 Limit state: freeze/thaw damage causing local loss of mechanical properties, cracking, scaling and loss in cross-section

(1) The following limit state function needs to be fulfilled:

$$p\{\} = p_{\text{freeze/thaw damage}} = p\{S_{CR} - S_{ACT}(t < t_{SL}) < 0\} < p_0 \quad (3.5\text{-}1)$$

$p\{\}$: probability that freeze/thaw damage occurs

S_{CR}: critical degree of saturation [-]

$S_{ACT}(t)$: actual degree of saturation at the time t [-]

t_{SL}: design service life [years]

p_0: target failure probability, cp. Annex A, Table A2-2

(2) The variables S_{CR} and $S_{ACT}(t)$ need to be quantified in a full probabilistic approach.

(3) To exemplify the design procedure and the quantification of above given quantities, an applicable design method is given in Chapter B3, Annex B. Other methods may be used, provided that the basic principles formulated in Chapter 2.4.1 are fulfilled.

3.5.1.2 Limit states: freeze/thaw-induced deflection and collapse

(1) With regard to load-carrying capacity and deformations, the traditional design must include the localized changes in mechanical properties due to frost damage.

3.5.2 Partial factor method

(1) The following limit state function needs to be fulfilled:

$$S_{CR,d} - S_{ACT,d}\,(t < t_{SL}) \geq 0 \qquad (3.5\text{-}2)$$

$S_{CR,d}$: design value of the critical degree of saturation [-]

$S_{ACT,d}(t < t_{SL})$: design value of the actual degree of saturation at time t [-]

t_{SL}: design service life [years]

(2) The design value of the critical degree of saturation is calculated as follows:

$$S_{CR,d} = S_{CR,min} - \Delta S_{CR} \qquad (3.5\text{-}3)$$

$S_{CR,\,min}$: characteristic value of the critical degree of saturation (minimum value) [-]

ΔS_{CR}: margin of the critical degree of saturation [-]

(3) The design value of the actual degree of saturation at a time t $S_{ACT,d}$ (t) is calculated as follows:

$$S_{ACT,d}(t) = S_{ACT}(t) + \Delta S_{ACT} \qquad (3.5\text{-}4)$$

$S_{ACT,d}$: characteristic value of the actual degree of saturation at a time t [-]

ΔS_{ACT}: margin of the actual degree of saturation (load) [-]

(4) To exemplify the design procedure an applicable design method is given in Annex C3. Other methods may be used, provided that the basic principles formulated in Chapter 2.4.2 are fulfilled.

3.5.3 Deemed-to-satisfy method

(1) Within this approach a trading-off of available space for expansion (air entrainment), material (non-freezable water characteristics) and aging (carbonation) variables can be established.

3.5.4 Avoidance-of-deterioration method

(1) Generally, avoidance is achieved if frost deterioration cannot take place due to infinite material resistance or zero environmental load.

3.6 Freeze/thaw attack – with de-icing agents

3.6.1 Full probabilistic method

3.6.1.1 Limit state equation for the salt-freeze/thaw induced surface scaling

(1) The following limit state function needs to be fulfilled:

$$p\{\} = p_{scaling} = p\{T(t \leq t_{SL}, Cl^-) - T_R(RH(T), T(t), ...) < 0\} < p_0 \qquad (3.6\text{-}1)$$

$p\{\}$: probability that scaling occurs

$T(t,\ldots)$: concrete temperature in [K]

$T_R(t,\ldots)$: critical freezing temperature for scaling to occur at the time t

t_{SL}: design service life [years]

p_0: target failure probability, cp. Annex A, Table A2-2

(2) The variables T and T_R need to be quantified in a full probabilistic approach.

(3) To exemplify the design procedure and the quantification of above given quantities, an applicable design method is given in Chapter B4, Annex B. Other methods may be used, provided that the basic principles formulated in Chapter 2.4.1 are fulfilled.

See Chapter 3.5.2 and Chapter 3.6.1.1

3.6.1.2 Limit states: freeze/thaw-induced deflection and collapse

4 Execution and its quality management

4.1 General

(1) The SLD according to this document assumes that the minimum requirements for execution and its quality management given in ENV 13670-1 “Execution of concrete structures – Part 1: Common rules”, included the amendments given under, are met.

“Those quality management and control measures in design, detailing and execution which are given in Chapter B4 and B5 of this annex (Remark: of annex B of EN 1990:2002) aim to eliminate failures due to gross errors, and ensure the resistance assumed in design”. From Note under EN 1990:2002, B1 (2) b).

To define a set of minimum requirements to the execution, an execution standard prepared according to the principles given in this document is needed as a reference. Since ENV 13670-1 fulfils this role, and since it is chosen by ISO TC-71 as the basis for the coming ISO-standard on execution of concrete structures, ENV 13670-1 is also chosen by *fib* TG 5.6 as the reference.

ENV 13670-1 is expected to be replaced by EN 13670 in 2007.

4.2 Project specification

(1) The project specification shall cover technical data and requirements for a particular project prepared to supplement and qualify the requirements of ENV 13670-1.

CEN ENV 13670-1 further refers to product and component standards for concrete, reinforcement, prestressing systems, prefabricated elements etc.

The specification of the properties of relevance to the design of these materials and components shall be included in the project specification.

(2) It is assumed that the project specification includes all necessary information and technical requirements for execution of the works and agreements made during the execution.

The project specification shall therefore comprise all the assumptions to materials, execution and condition control made in the specific SLD.

Depending on the method used in the SLD, the project specification will give requirements for the materials selection, the execution and the condition control during the service life of the structure.

4.3 Quality management

(1) The quality management for the execution:

– might involve a quality plan

– shall include inspection of the completed work

“Quality Plan” and “Inspection” are defined in:

– ISO 9000/3.7.5 Quality plan: “Document specifying which procedures and associated resources shall be applied by whom and when to a specific project, product or process.”

– ISO 9000/3.8.2 Inspection: “Conformity evaluation by observation and judgment accompanied as appropriate by measurement, testing and gauging.”

4.3.1 Quality plan

(1) If the project specification requires a quality plan, the project specification shall define what elements it shall comprise.

(2) A quality plan might include elements like competence and appropriate training of personnel, the organization of the project and procedures for the execution.

ISO 10005:2005” Quality management – Guidelines for quality plans” gives further advice for the development, acceptance, application and revision of quality plans.

4.3.2 Inspection

(1) The needed inspection to perform a conformity evaluation of the completed work shall be carried out and the results documented.

(2) The project specification might give requirements for the “as-built-documentation” depending on the specifics of the actual SLD.

“as-built-documentation” of the direct input parameters to the SLD models might confirm the design assumptions and possible give the basis for corrective measures. It might also serve as a basis for the condition control of the structure during its service life. Such an extract of the “as-built-documentation” is sometimes named the structure’s “Birth Certificate”.

Such specifics might be the documentation of the achieved direct input parameters applied in the SLD models like for instance diffusion coefficients, cover thickness to the reinforcement etc.

4.3.3 Action in the event of non-conformity

(1) If the inspection reveals that the original SLD assumptions are not met during the construction, actions as given in 5.4 shall be taken.

4.4 Materials

4.4.1 Formwork

(1) Possible other requirements than those listed in ENV 13670-1 shall be stated in the project specification.

4.4.2 Reinforcement

(1) Requirements to possible other types of reinforcement than ordinary steel according to prEN 10080 (for instance galvanized, stainless, coated, non-metallic, etc.) shall be stated in the project specification.

4.4.3 Pre-stressing

(1) Requirements to possible other post-tensioning systems than those referred to in ENV 13670-1 (for instance plastic sheaths, non-metallic strands etc) shall be stated in the project specification.

4.4.4 Concrete

(1) The concrete shall be specified according to, and comply with EN 206-1.

The project specification shall state possible additional requirements to be met depending on the specific SLD models applied.

To define a set of minimum requirements to the performance of concrete, a product standard prepared according to the principles given in this document is needed as a reference. Since EN 206-1 fulfils this role, and since it is chosen by ISO TC-71 as the basis for the coming ISO-standard on concrete – specification, performance, production and conformity, EN 206-1 is also chosen by *fib* TG 5.6 as the reference.

If the SLD is based on performance characteristics of the concrete, these might be replaced by requirements to mix composition either in the design phase based on previous experience, or by initial testing during the construction phase. It shall be stated if the operational requirements for mix composition are target values or characteristic values.

(2) If test methods not referred to in EN 206-1 are to be applied, the sampling, these test methods, and the statistical interpretation of their results, shall be stated in the project specification.

If the SLD is based on other material characteristics than those dealt with in traditional concrete standards like EN 206-1 (i.e. cement type, water-binder ratio, cement content, aggregate property etc), and the SLD depends on a verification of these material characteristics during construction, the project specification shall refer to the relevant test methods and the statistical interpretation of the results (for instance characteristic values or target values).

Such additional material characteristics might for instance be the chloride diffusion coefficient or the inverse carbonation resistance.

4.5 Geometry

The geometrical tolerances given in ENV 13670-1, clause 10 achieves the design assumptions in the European design standard EN 1992 and the required level of safety. These are related to both SLD and the given partial factors for materials used in load bearing design.

(1) The requirements to geometrical tolerances given in class 1 in ENV 13670-1 clause 10 are assumed to have direct relevance to the design assumptions, while those given in ENV 13670-1, Annex F do not.

The tolerances given in ENV 13670-1 annex F are considered to have small structural influence.

(2) The term “permitted deviation” in ENV 13670-1 on geometrical tolerances might be interpreted as the 5 % percentile.

(3) Possible other assumptions on geometrical tolerances applied in the SLD than those given in ENV 13670-1 shall be stated in the project specification.

5 Maintenance and condition control

5.1 General

(1) This chapter provides the general basis for maintenance and condition control during the service life for concrete structures.

(2) Chapter A6, Annex A gives advice for the extent of inspection / monitoring of the structure during its service life.

Table A6-1 defines 4 "Condition Control Levels" as a guidance to the reliability differentiation to be used in the SLD.

5.2 Maintenance

Based on § 6.7 of ISO 15686-1:2000.

(1) In this document the term "maintenance" is used on activities that are planned to take place during the service life of the structure in order to ensure the fulfilment of the assumptions in the SLD.

The maintenance plan might comprise activities like general cleaning, drainage, addition of sealants, replacement of components etc.

(2) A maintenance plan shall state type and frequency of the foreseen activities.

5.3 Condition control during service life

Guidance might be found in ISO/DIS 15686-7:2004 "Buildings and construction assets – Service life planning – Part 7 : Performance evaluation for feedback of service life data from practice".

Chapter A6, Annex A, proposes 4 classes for "Condition Control" giving guidance for the type and extent of inspection/monitoring during the service life.

According to Chapter A6, Annex A, the lowest level of condition control is "No systematic monitoring nor inspection". In every-day construction, this is often the most appropriate level, and its consequences shall be taken into account for the reliability management for the SLD.

5.3.1 Inspection and monitoring during service life

(1) In this document "inspection" means activities to evaluate conformity with the design data for actions and/or material and/or product properties used in the SLD on a periodic basis during the service life of the structure, while "monitoring" means the same activities, but on a continuous basis.

The conformity evaluation might be done by visual observations and judgment accompanied as appropriate by measurements, testing and gauging.

5.3.2 Condition control plan

(1) The plan shall state:

- What types of inspection / monitoring that shall take place
- What components of the structure to be inspected / monitored
- The frequency of the inspections
- The performance criteria to be met
- Possible documentation of the results
- Action in the event of non-conformity with the performance criteria

The service life of a component or structure is always related to one, or a few required functions of that component or structure.

The planned activities on inspection / monitoring shall therefore focus on the evaluation of the design data applied in these deterioration models.

5.4 Action in the event of non-conformity

(1) If the inspection/monitoring reveals that the original SLD assumptions are not met, one or more of the following actions shall be taken:

- Widening the scope of the performance survey to improve the quality and representativeness of the data.
- Performing a recalculation of the original SLD to assess the residual service life of the structure. The new calculation shall be supplemented with the data for action, materials and products derived from the field-exposed structure. The redesign shall conform to the requirements given in Chapter 2 of this document.
- The structure shall be repaired or strengthened to bring its performance back to the agreed design assumptions. The repair shall be based on a partial or full recalculation of the original SLD as stated under 2.
- The structure shall be protected to reduce the action. The protection shall be based on a recalculation of the original SLD as stated under 2.
- The structure shall become obsolete.

For existing structures the costs of achieving a higher reliability level are usually high compared to structures under design.

For this reason the target level of reliability for redesign of service life of existing structures usually should be lower.

(Based on 7.2.1 of JCSS PMC:2000 “Probabilistic Model Code”, Joint Committee on Structural Safety)

(2) According to Chapter 2.1.3 (4) of this document, the serviceability criteria to be applied during the assessment shall be specified for the project and agreed with the owner.

This annex is based on EN 1990, Annex B “Management of Structural Reliability for Construction Works”

Annex A (informative)

Management of reliability for Service Life Design of concrete structures

A1 Scope and field of application

(1) This annex provides additional guidance to 2.1.2 (reliability management).

(2) The approach given in this annex recommends the following procedures for the management of reliability of SLD for concrete structures:

- In relation to 2.1.2 (1), classes are introduced and are based on the assumed consequences of failure and the exposure of the construction works to hazard. A procedure for allowing moderate differentiation in the partial factors for actions and resistance corresponding to the classes is given in A2.

  Note: Reliability classification can be represented by β indexes, which take account of accepted or assumed statistical variability in action effects and resistance and model uncertainties.

- In relation to 2.1.2 (1), a procedure for allowing differentiation between various types of construction works in the requirements for quality levels of design and execution process, as well as the extent of condition control during the service life, are given in A3, A4 and A5.

  Note: Those quality management and control measures in design, detailing and execution which are given in A3 and A4 aim to eliminate failures due to gross errors, and to ensure the resistance assumed in the design.

(3) The procedure has been formulated in such a way so to produce a framework to allow different reliability levels to be used, if desired.

A2 Reliability differentiation

A2.1 Consequences classes

Table A2-1 is identical to table B1 of EN 1990.

(1) For the purpose of reliability differentiation, consequences classes (CC) may be established by considering the consequences of failure or malfunction of the structure as given in Table A2-1.

Table A2-1: Definition of consequences classes

Consequences Classes	**Description**	**Examples of building and civil engineering works**
CC3	High consequence for loss of human life, or economic, social or environmental consequences very great	Grandstands, public buildings where consequences of failure are high (e. g. a concert hall)
CC2	Normal consequence for loss or human life, economic or environmental consequences considerable	Residential and office buildings, public buildings where consequences of failure are medium (e. g. an office building)
CC1	Low consequence for loss of human life, and economic, social or environmental consequences are small or negligible	Agricultural buildings where people do not normally enter (e.g. storage buildings), green houses

(2) The criterion for classification of consequences is the importance, in the terms of consequences of failure, of the structure or structural member concerned. See A2.3.

(3) Depending on the structural form and decisions made during design, particular members of the structure may be designed in the same, higher or lower consequences class than for the entire structure.

The normal consequence by passing a SLS (for instance depassivation of surface reinforcement), is that possible protective measures / repair become more expensive.

In any case, a ULS design has to be made. It is assumed, that the usual design of reinforced and pre-stressed structures is made in that way, that the ULS requirements of Table A2-2 are fulfilled exactly. Corrosion of reinforcement (pre-stressing steel) and/or deterioration of concrete (bond failure, lack of sufficient compressive cross section) will decrease the reliability. If corrosion can not be excluded at a ULS reliability and inspection/maintenance/repair that means "intervention" can not be executed, this will lead to the need of extra reinforcement (sacrificial cross section) and/or special detailing in order to avoid bond failure within the bonding zone. The dimension of this extra cross section highly depends on the reliability, depassivation is excluded. That means, the higher the reliability with regard to depassivation the lower the need of extra reinforcement.

A2.2 Differentiation by β values

(1) The reliability classes (RC) may be defined by the β reliability index concept.

(2) Three reliability classes RC1, RC2 and RC3 may be associated with the three consequences classes CC1, CC2 and CC3.

(3) Table A2-2 gives recommended minimum values for the reliability index associated with the reliability classes.

Table A2-2: Recommended minimum values for reliability index ß for use in SLD (intended for the design life time)

Exposure Class – Eurocode 2	**Description**	**Reliability Class**	SLS[1]	ULS
			Depassivation[2,3]	Collapse
XC[3]	Carbonation	RC1	1.3 ($p_f \approx 10^{-1}$)	3.7 ($p_f \approx 10^{-4}$)
		RC2	1.3 ($p_f \approx 10^{-1}$)	4.2 ($p_f \approx 10^{-5}$)
		RC3	1.3 ($p_f \approx 10^{-1}$)	4.4 ($p_f \approx 10^{-6}$)
XD[3]	Deicing salt	RC1	1.3 ($p_f \approx 10^{-1}$)	3.7 ($p_f \approx 10^{-4}$)
		RC2	1.3 ($p_f \approx 10^{-1}$)	4.2 ($p_f \approx 10^{-5}$)
		RC3	1.3 ($p_f \approx 10^{-1}$)	4.4 ($p_f \approx 10^{-6}$)
XS[3]	Seawater	RC1	1.3 ($p_f \approx 10^{-1}$)	3.7 ($p_f \approx 10^{-4}$)
		RC2	1.3 ($p_f \approx 10^{-1}$)	4.2 ($p_f \approx 10^{-5}$)
		RC3	1.3 ($p_f \approx 10^{-1}$)	4.4 ($p_f \approx 10^{-6}$)

[1] A SLS reliability of $\beta = 1.3$ in consequence could lead to lower ULS reliabilities than usually required by the codes, cp. ISO 2394. That means for very aggressive climates, higher values for β_{SLS} are required, cp. Annex R, in order to fulfil the ULS requirements.

[2] Depassivation of the surface reinforcement in the area exposed to the design environmental load.

[3] In cases with sufficient access of oxygen and moisture to support corrosion.

A2.3 Differentiation by measures relating to the partial factors

(1) One way of achieving reliability differentiation is by distinguishing classes of γ_F factors to be used in fundamental combinations for persistent design situations. For example, for the same design supervision and execution inspection levels, a multiplication factor K_{FI} , see Table A2-3, may be applied to the partial factors.

Table A2-3: K_{FI} factors

	Reliability Class		
	RC1	RC2	RC3
K_{FI}	So far no quantified number available (< 1)	1.0	So far no quantified number available (> 1)

Note: In particular, for class RC3, other measures as described in this annex are normally preferred to using K_{FI} factors. K_{FI} should be applied only to unfavourable actions.

(2) Reliability differentiation may also be applied through the partial factors on resistance γ_M. However, this is not normally used.

(3) Accompanying measures, for example the level of quality control for the design and execution of the structure, may be associated to the classes of γ_F. In this annex, a three level system for control during design and execution has been adopted. Design supervision levels and inspection levels associated with the reliability classes are suggested.

(4) There can be classes (e.g. lighting poles, masts, etc.) where, for reasons of economy, the structure might be in RC1, but be subjected to higher corresponding design supervision and inspection levels.

Structural failure caused by corrosion of reinforcement may be due to loss of cross section of bars or due to spalling of concrete cover and loss of anchorage.

Spalling of concrete cover in anchorage zones without confinement may lead to sudden failure.

Reliable findings for limit values of corrosion intensities causing spalling do not exist. Limit values will depend on bar diameter and bar spacing and on environmental conditions (volume of rust products). The given values in Table A3-1 are rough estimates and need to be confirmed by further research. Being rough estimates the given values in Table A3-1 can be taken as mean values providing the cross section contains more than three single bars.

A3 Robustness of sections related to corrosion

(1) The service life of a structure susceptible to rebar corrosion depends on the length of the initiation period and the length of the propagation period. That means the structural ULS reliability can either be achieved by excluding corrosion at a ULS reliability, or by adding needed extra reinforcement (sacrificial cross section). In most cases both design elements will be taken into account. The dimension of this extra cross section highly depends on the reliability, depassivation is excluded. That means, the higher the reliability with regard to depassivation the lower the need of extra reinforcement.

A critical loss of extra cross section of bars caused by corrosion leading to structural failure need to be defined for ULS. To differentiate different failure modes, robustness classes may be defined.

Table A3-1: Robustness Classes (ROC)

Robustness Class	**Characteristics**	**Characteristics Loss of Cross sections (rough estimates) ΔA_s [%]**
ROC 3	bending reinforcement outside of anchorage and laps	25
ROC 2	shear reinforcement, anchorage zones with confinement	15
ROC 1	anchorage zones without confinement	5

In dependency of ROC's it might be necessary to fulfil ULS-requirements by excluding depassivation on a higher reliability level as recommended in Table A2-2.

A4 Design, quality management differentiation

(1) Design supervision differentiation consists of various organisational quality control measures, which can be used together. For example, the differentiation of design supervision level (A4(2)) may be used together with other measures such as classification of designers and checking authorities (A4(3)).

Minimum levels for the quality management regime are often given in national legislation.

(2) Three possible design supervision levels (DSL) are shown in Table A4-1. The design supervision levels may be linked to the reliability class selected or chosen according to the importance of the structure and in accordance with national requirements or the design brief, and implemented through appropriate quality management measures. See 2.1.2 (1).

Table A4-1: Design supervision levels (DSL)

Design Supervision Levels	**Characteristics**	**Minimum recommended requirements for the checking of calculations, drawings and specifications**
DSL3 Relating to RC3	Extended supervision	Third party checking: Checking performed by an organisation different from that which has performed the design
DSL2 Relating to RC2	Normal supervision	Checking by different persons than those originally responsible and in accordance with the procedure of the organisation
DSL1 Relating to RC1	Normal supervision	Self-checking: Checking performed by the person who has prepared the design

(3) Design supervision differentiation may also include a classification of designers and/or design inspectors (checkers, controlling authorities, etc.), depending on their competence and experience, their internal organisation for

the relevant type of construction works being designed.

Note: The type of construction works, the materials used and the structural forms can affect this classification.

(4) Alternatively, design supervision differentiation can consist of a more refined detailed assessment of the nature and magnitude of actions to be resisted by the structure, or of a system or design load management to actively or passively control (restrict) these actions.

A5 Execution, quality management differentiation

(1) Three execution classes (EXC) may be introduced as shown in Table A5-1. The execution classes may be linked to the quality management classes selected and implemented through appropriate quality management measures. See 2.1.2 (1).

CEN ENV 13670-1 refers to “inspection classes”.

“Inspection” is defined by ISO 9000 as “Conformity evaluation by observation and judgment accompanied as appropriate by measurement, testing or gauging”.

The “Execution classes” might also comprise other elements of the quality management regime at the construction site.

Minimum levels for the quality management regime are often given in national legislation.

Table A5-1: Execution Classes (EXC)

Execution Class	**Characteristics**	**Requirements**
EXC3 Relating to RC3	Extended inspection	Third party inspection
EXC2 Relating to RC2	Normal inspection	Inspection in accordance with the procedures of the organisation
EXC1 Relating to RC1	Normal inspection	Self inspection

A6 Condition control during service life, quality management differentiation

(1) For service life design, the level of supervision during the use of the structure or component is also decisive for the appropriate level of reliability. For this use the following condition control levels (CCL) during the service life might be applied:

A proper inspection during the service life of a structure will give the owner a possibility to apply protective measures in case the expectations for the service life design are not met.

The consequences of unacceptable performance are thus reduced. This opens then for applying a more liberal reliability class and associated β -value.

Table A6-1: Conditions Control Levels (CCL)

Condition Control Levels	Characteristics	Requirements
CCL3	Extended inspection	Systematic inspection and monitoring of relevant parameters for the deterioration process(es) that is (are) critical in the SLD
CCL2	Normal inspection	Regular visual inspection by qualified personnel
CCL1	Normal inspection	No systematic monitoring nor inspection
CCL0	No inspection	No possible inspection, for instance due to lack of access

A7 Relative cost of measures

Guidance might be found in "Probabilistic Model Code", Joint Committee on Structural Safety (JCSS PMC:2000).

(1) As serviceability failures by definition are not associated with loss of human life or limb, the cost of measures to achieve a higher reliability level should influence the choice of consequence class and thus reliability index (cp. Table A2-1 and Table A2-2).

(2) For existing structures the costs of achieving a higher reliability level are usually high compared to structures under design. For this reason the target level for existing structures usually should be lower.

A8 Partial factors for resistance properties

(1) A partial factor for a material or product property or a member resistance may be reduced if an inspection class higher than that required according to Table A5-1 and/or more severe requirements are used.

Note: Such a reduction, which allows for example for model uncertainties and dimensional variations, is not a reliability differentiation measure: it is only a compensating measure in order to keep the reliability level dependent on the efficiency of the control measures.

Annex B (informative)

Full probabilistic design methods

B1 Full probabilistic design method for carbonation induced corrosion – uncracked concrete

B1.1 Limit state equation for the depassivation of the reinforcement

(1) A full probabilistic design approach for the modelling of carbonation induced corrosion of uncracked concrete has been developed within the research project DuraCrete and slightly revised in the research project DARTS, each project was funded by the European Union. It is based on the limit-state Equation B1.1-2, in which the concrete cover a is compared to the carbonation depth $x_c(t)$ at a certain point of time t.

The original DuraCrete model is described in more detail in [3], the DARTS model (revised DuraCrete model) is described in [4], [5].

$$g\left(a, x_c(t)\right) = a - x_c(t) = a - \sqrt{2 \cdot k_e \cdot k_c \cdot (k_t \cdot R_{ACC,0}^{-1} + \varepsilon_t) \cdot C_S} \cdot \sqrt{t} \cdot W(t) \tag{B1.1-2}$$

a:	concrete cover [mm], cp. B1.2.1
$x_c(t)$:	carbonation depth at the time t [mm]
t:	time [years], cp. B1.2.2
k_e:	environmental function [-], cp. B1.2.3
k_c:	execution transfer parameter [-], cp. B1.2.4
k_t:	regression parameter [-], cp. B1.2.5
$R_{ACC,0}^{-1}$:	inverse effective carbonation resistance of concrete [(mm²/years)/(kg/m³)], cp. B1.2.5
ε_t:	error term, cp. B1.2.5

While assessing existing structures, the constants in Equation B1.1-1 might be combined to achieve a simplified expression:

$$g\left(a, x_c(t)\right) = a - k\sqrt{t} \tag{B1.1-1}$$

In view of *fib* TG 5.6, published models of [6] or others are useful as well, if validated according to the principles given in Chapter 2.

C_S: CO_2-concentration [kg/m³], cp. B1.2.6

W(t): weather function [-], cp. B1.2.7

(2) Equation B1.1-2 is based on diffusion as the prevailing transport mechanism within the concrete (Fick's 1st law of diffusion). It is assumed that the diffusion coefficient for carbon dioxide through the material is a constant material property, although the CO_2-diffusion coefficient for a concrete during service life may be a function of numerous variables.

B1.2 Quantification of parameters

B1.2.1 Concrete cover a

B1.2.1.1 General

(1) The concrete cover a is chosen during the design phase. Due to construction practices the actual concrete cover does vary and therefore has to be considered as a stochastic variable rather than a constant value. The following distribution types are in principle appropriate for the description of the concrete cover a and its variability:

– Normal distribution
– Beta-distribution
– Weibull(min)-distribution
– Lognormal distribution
– Neville distribution

When choosing a distribution function for the description of the concrete cover, it has to be considered that values of the varying concrete cover including the scatter are positive defined values ($p_f = p\{a < 0\} = 0$). Only due to bad workmanship defects, if for example the reinforcement is being pushed into the formwork, it is theoretically possible that the concrete cover may take negative values. Also considerations targeted on restricting the upper value of the concrete cover are possible ($p_f = p\{a > d\} = 0$, d = dimension of the structural element).

(2) By applying a Beta, Weibull(min), Lognormal and Neville distribution, negative values for the concrete cover are excluded due to the characteristics of these types of distributions. If a normal distribution is considered, one has to be aware that negative values for the concrete cover are not excluded by the characteristics of the normal distribution. Especially for concrete covers with a small mean value, this can lead to unrealistic results, since a high probability of negative values for the concrete cover may exist from a statistical point of view. When the mean value becomes larger (hereby assuming a steady

standard deviation) this effect becomes neglegible. For a statistic description of low concrete covers (e.g. nom a = 20 mm) in particular, the right-skewed lognormal distribution, Neville distribution and beta-distribution (with a lower bound of a = 0 mm) are considered to be appropriate.

B1.2.1.2 Quantification of a

(1) Distribution function: For large concrete covers all of the distribution types discussed before can be applied. In this case a normal distribution is very common. If a rather small concrete cover has to be described, distributions excluding negative values should be chosen, as for instance the Lognormal, Beta-, Weibull(min)- or the Neville distribution. Especially if due to the application of quality control action a small standard deviation is expected, a Neville distribution shows good fitting characteristics.

As the nominal cover is secured by spacers of corresponding dimension, the designer can expect, that the achieved mean value of the concrete value may equal to the nominal value.

mean value of a:

m = nom a [mm]

From field investigations it turned out, that the observed standard deviations of the concrete cover were in the range of 2 mm $\leq$ s $\leq$ 15 mm. In most cases, the given recommendation of chapter B1.2.1.2, (1) in regard to s can be taken.

standard deviation of a:

s = 8 - 10 mm	without particular execution requirements
s = 6 mm	with additional execution requirements targeted

for restricted distributions:

lower limit: 0 mm

upper limit: 5 · nom a < d, d: width of the structural element [mm]

B1.2.2 Design service life t_{SL}

(1) Indicative values for the design service life t_{SL} are given in Table B1-1:

Table B1-1: Indicative values for the design service life t_{SL}

design service life t_{SL} [years]	**Examples**
10	Temporary structures (structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary)
10 - 25	Replaceable structural parts, e. g. gantry girders, bearings
15 – 30	Agricultural and similar structures
50	Building structures and other common structures
100	Monumental buildings structures, bridges, and other civil engineering structures

Compare EN 1990:2002, Table 2.1.

B1.2.3 Environmental function k_e

B1.2.3.1 General

(1) The environmental function k_e takes account of the influence of the humidity level on the diffusion coefficient and hence on the carbonation resistance of the concrete. The reference climate is T= +20°C/ 65% RH.

(2) The environmental function k_e can be described by means of Equation B1.2-1, cp. also [4]:

$$k_e = \left(\frac{1 - \left(\frac{RH_{real}}{100} \right)^{f_e}}{1 - \left(\frac{RH_{ref}}{100} \right)^{f_e}} \right)^{g_e} \tag{B1.2-1}$$

Carbonation measurements on concrete and mortar specimens exposed to various values of relative humidity show that up to approx. RH = 60 % the carbonation depth increases, which is followed by decreasing carbonation depths for an increasing relative humidity. Since for instance in European climates a relative humidity below 60 % is less common, Equation B1.2-1 appears to be sufficient. For lower values of the relative humidity the model of k_e is on the safe side.

RH_{real}:	relative humidity of the carbonated layer [%]
RH_{ref}:	reference relative humidity [%]
f_e:	exponent [-]
g_e:	exponent [-]

B1.2.3.2 Relative humidity RH_{real}

(1) Data of the nearest weather station may be used as an input for RH_{real}. For quantification, the weather station data (daily mean value) has to be evaluated.

(2) Due to the fact that the relative humidity varies by definition utmost in a range of $0\ \% < RH \leq 100\ \%$, restricted distributions with an upper limit should be used to describe this variable. For instance in European climate conditions, a right-skewed distribution is in general appropriate to describe RH_{real}. Depending on the region the lower limit of RH might be significantly different from zero. In such a case it seems reasonable to describe the data set by means of a distribution function with an upper and a lower limit, as for example:

- Beta-distribution
- Weibull(max)-distribution

Strictly speaking, the relative humidity of the carbonated layer has to be taken into account. Since it is very difficult to obtain such data and due to the fact that the carbonation process takes place in the outer parts of the concrete it seems justifiable to use relative humidity data (e.g. mean daily values) derived from the ambient air of the structure. However result of further research on this factor may also, that mean yearly values may be sufficient as well. For the time being statistically described data of mean daily values are recommended.

B1.2.3.3 Reference relative humidity RH_{ref}

(1) The reference relative humidity has to be chosen in accordance with the test conditions for determining the carbonation resistance of the concrete. For the recommended ACC-test method, which is described in Chapter B1.2.5.2, the reference climate T= +20°C/ 65% RH. Therefore, RH_{ref} is quantified as follows:

RH_{ref} [%]: constant parameter, value: 65

B1.2.3.4 Exponents g_e, f_e

(1) The parameters g_e and f_e have been determined by means of a curve-fitting procedure with the actual test data. The best results were gained with the following set of parameters, cp. [4] and [5]:

g_e [-]: constant parameter, value: 2.5

f_e [-]: constant parameter, value: 5.0

The exponents are independent of exposure conditions and management phases.

B1.2.4 Execution transfer parameter k_c

B1.2.4.1 General

(1) The execution transfer parameter k_c takes the influence of curing on the effective carbonation resistance into account. In this context, all measures which are targeted on preventing premature desiccation of concrete close to the surface are being considered as curing measures.

Measures such as water cured, air cured but sealed with sheets in order to prevent desiccation, casted/moulded are being considered as curing measures.

(2) Figure B1.2-1 illustrates the influence of the duration of curing on the curing effect. The statistical quantification of k_c has been carried out by means of a linear regression (double logarithmic scale) according to Bayes, cp. [5].

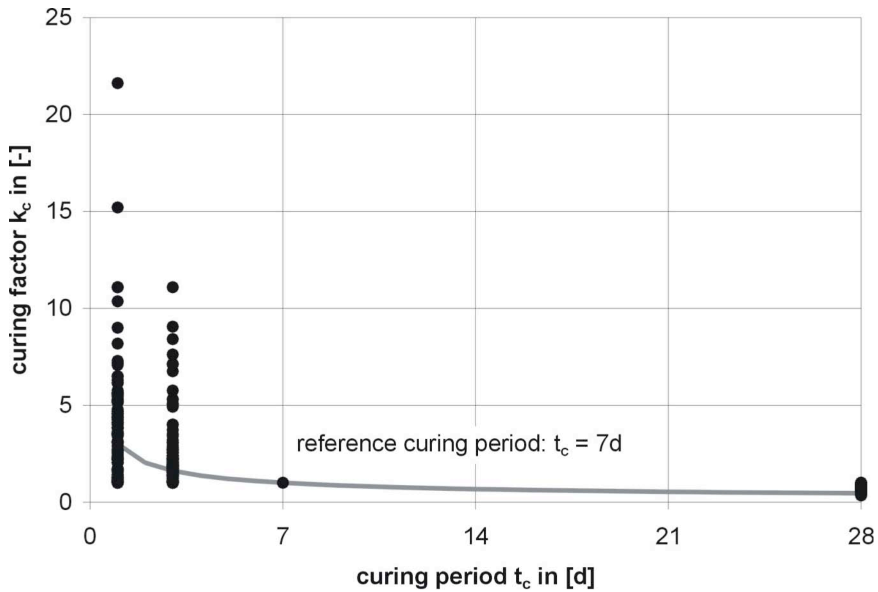

Figure B1.2-1: curing variable versus curing period (n = 312), [5]

(3) By means of Bayesian regression, the following Equation B1.2-2 has been determined, [5]:

$$k_c = \left(\frac{t_c}{7}\right)^{b_c} \qquad \text{(B1.2-2)}$$

k_c: execution transfer parameter [-]

b_c: exponent of regression [-]

t_c: period of curing [d]

B1.2.4.2 Quantification of k_c

(1) The variables b_c and t_c have been quantified as follows, [5]:

b_c [-]:	normal distribution,	m:	-0.567
		s:	0.024
t_c [d]:	constant, parameter,	value: period of curing	

B1.2.5 Inverse Carbonation Resistance $R_{ACC,0}^{-1}$

B1.2.5.1 General

(1) For the model introduced above, it has been agreed upon that the inverse effective carbonation resistance is to be determined by accelerated carbonation tests (ACC-test method) in which laboratory (20/65) pre-stored concrete specimens are tested under defined conditions at a reference time t_0.

(2) The relationship between the inverse carbonation resistances obtained under natural conditions (NAC) and in an accelerated test (ACC) is illustrated in Figure B1.2-2, [5].

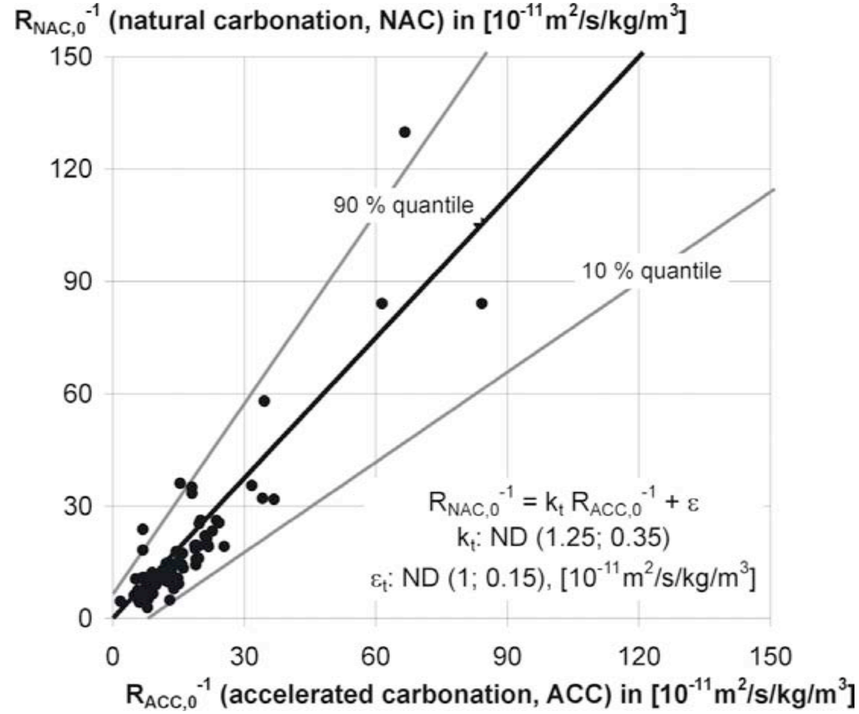

Figure B1.2-2: Relationship of the inverse carbonation resistances, obtained under natural conditions (NAC) and in an accelerated test (ACC, [5])

(3) Inverse carbonation resistances $R_{NAC,0}^{-1}$ determined under natural carbonation conditions will be larger by an average factor of A = 1.25. This may be explained by the fact that in an accelerated test the drying front does not penetrate as deep as under natural conditions (though testing under the same climatic conditions being 20°C/65 RH). This will slightly retard the carbonation process under ACC conditions. For very dry concrete this theoretically implies values of $R_{ACC,0}^{-1} = 0$. As concrete has no infinite resistance, the so-called error term $\varepsilon_t > 0$ (y-intercept) has been introduced.

B1.2.5.2 Performance tests for the determination of $R_{ACC,0}^{-1}$

(1) For measuring the carbonation resistance different direct and indirect testing methods can be used. The benefits of the ACC test method are:

- the binding capacity of the concrete does not have to be considered additionally
- changes of the carbonation resistance due to carbonation do not have to be considered additionally
- good reproducibility of the test results
- short duration

Short duration is important to get as early as possible information about the material performance.

(2) For these reasons the ACC-test method with the following procedure has been chosen as the reference test method, [5].

The presented procedure is open for further more detailed specifications, e. g. tolerances on RH_{ref}, T_{ref}, C_S etc.

- Production of concrete specimens with the following dimensions: height/width/length = 100/100/500 [mm].
- After removing of the formwork the specimens have to be stored in tap water with a temperature of T_{ref} = 20°C for overall seven days (reference curing).
- Subsequent to the water storage described above, the specimens are removed from the water and stored for 21 further days in a standardised laboratory climate (T_{ref} 20°C, RH_{ref} = 65 %).
- At the age of 28 days (t_{ref} = 28 d) the specimens are placed in a carbonation chamber with the standardised laboratory climate (T_{ref} 20°C, RH_{ref} = 65 %). In the chamber the specimens are exposed to a CO_2 concentration of C_S = 2.0 vol.-% during 28 days.
- After removal the concrete specimens are split and the carbonation depth is measured at the plane of rupture with an indicator solution consisting of 1.0g phenolphthalein per litre.
- By evaluation of the measured carbonation depth according to Equation B1.2-3, the mean value of the reference inverse effective carbonation resistance can be determined.

Test duration in total: Δt = 56 days. ‘ACC-conditions’ with regard to CO_2-concentration were set to a maximum of C_S = 2.0 vol.-% to avoid as much as possible the formation of phases which normally are not formed under natural carbonation conditions, e. g. vaterite.

$$R^{-1}_{ACC,0} = \left(\frac{x_c}{\tau}\right)^2 \qquad \text{(B1.2-3)}$$

$R_{ACC,0}^{-1}$: inverse effective carbonation resistance of concrete $[(m^2/s)/(kg/m^3)]$

τ: 'time constant' in $[(s/kg/m^3)^{0.5}]$, for described test conditions: $\tau = 420$

x_c: measured carbonation depth in the compliance test [m]

B1.2.5.3 Quantification of $R_{ACC,0}^{-1}$

(1) $R_{ACC,0}^{-1}$ shows a normal distribution with mean values which can be calculated by means of Equation B1.2-3. The relationship between mean value and standard deviation of $R_{ACC,0}^{-1}$ is illustrated in Figure B1.2-3, cp. [5].

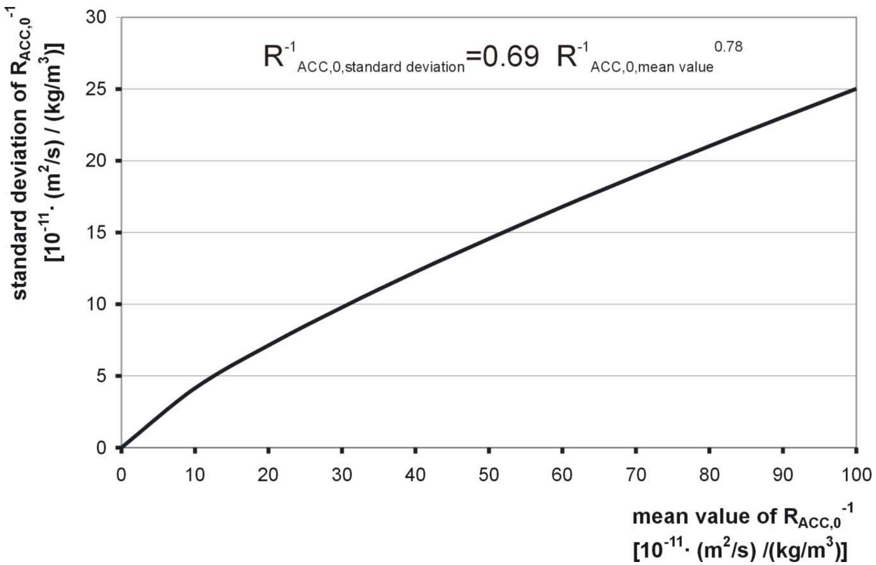

Figure B1.2-3: Quantification of $R_{ACC,0}^{-1}$; determination of the standard deviation based on the mean value

(2) The inverse carbonation resistance should be quantified as shown in this chapter. If no test data is available, the following literature data can be used for orientation purposes, cp. Table B1-2, cp. [5].

Table B1-2: Quantification of $R_{ACC,0}^{-1}$

$R_{ACC,0}^{-1}$ [10^{-11} (m^2/s)/(kg/m^3)] / cement type	$w/c_{eqv.}$ [1]					
	0.35	0.40	0.45	0.50	0.55	0.60
CEM I 42.5 R	n.d.[2]	3.1	5.2	6.8	9.8	13.4
CEM I 42.5 R + FA (k = 0.5)	n.d.[2]	0.3	1.9	2.4	6.5	8.3
CEM I 42.5 R + SF (k = 2.0)	3.5	5.5	n.d.[2]	n.d.[2]	16.5	n.d.[2]
CEM III/B 42.5	n.d.[2]	8.3	16.9	26.6	44.3	80.0

[1] equivalent water cement ratio, considering FA (fly ash) or SF (silica fume) with the respective k-value (efficiency factor).

The considered contents were: FA: 22 wt.-%/cement; SF: 5 wt.-%/cement.

[2] n.d. – inverse effective carbonation resistance $R_{ACC,0}^{-1}$ has not been determined for these concrete mixes.

(3) Considerable attention has to be paid to the units, as $R_{ACC,0}^{-1}$ has been determined by now within the unit [$10^{-11} \cdot$ (m^2/s)/(kg/m^3)]. For translation of $R_{ACC,0}^{-1}$ into the respective unit for the deterioration model [(mm^2/years)/(kg/m^3)], a multiplication factor has to be applied.

$R_{ACC,0}^{-1}$ [(m^2/s)/(kg/m^3)]: normal distribution,

m = according to Equation B1.2-3

(values for orientation purpose: Table B1-2)

s = according to Figure B1.2-3

B1.2.5.4 Test method factors

(1) The factors k_t and ε_t have been introduced in order to transform the results gained under "accelerated carbonation" conditions $R_{ACC,0}^{-1}$ into an inverse carbonation resistance $R_{NAC,0}^{-1}$ under "natural carbonation" conditions (NAC), cp. Equation B1.2-4, cp [5].

$$R_{NAC,0}^{-1} = k_t \cdot R_{ACC,0}^{-1} + \varepsilon_t \qquad \text{(B1.2-4)}$$

$R_{ACC,0}^{-1}$: inverse effective carbonation resistance of dry concrete, determined at a certain point of time t_0 on specimens with the accelerated carbonation test ACC [(mm²/years)/(kg/m³)]

$R_{NAC,0}^{-1}$: inverse effective carbonation resistance of dry concrete (65% RH) determined at a certain point of time t_0 on specimens with the normal carbonation test NAC [(mm²/years)/(kg/m³)]

k_t: regression parameter which considers the influence of test method on the ACC-test [-]

ε_t: error term considering inaccuracies which occur conditionally when using the ACC test method [(mm²/years)/(kg/m³)]

Factors k_t and ε_t will cover all the differences between specimens tested at ACC-conditions and the 'structure' tested under 'natural carbonation' conditions (climate 20/65). Differences in compaction and water-movements due to vibration between test specimens and structure are not quantified so far as compared specimens tested at 'ACC-conditions' and 'natural conditions', see Figure B1.2-2, were compacted identically.

(2) The test method factors for the accelerated carbonation test have been quantified as follows, cp. [5]:

k_t [-]:	normal distribution,	m = 1.25 s = 0.35
ε_t [(mm^2/years)/(kg/m^3)]:	normal distribution,	m = 315.5 s = 48

B1.2.6 Environmental impact C_s

B1.2.6.1 General

(1) The CO_2 concentration of the ambient air represents the direct impact on the concrete structure. The impact can be described by the following Equation B1.2-5, cp. [5]:

$$C_S = C_{S,atm.} + C_{S,emi.} \qquad \text{B(1.2-5)}$$

C_S: CO$_2$ concentration [kg/m³]

$C_{S,atm.}$: CO_2 concentration of the atmosphere [kg/m³]

$C_{S,emi.}$: additional CO_2 concentration due to emission sources [kg/m³]

Increased CO_2 concentrations can be applied e. g. to road tunnels or when combustion engines are used. For usual structures**,** Equation B1.2-5 can be reduced to Equation B1.2-6:

$$C_S = C_{S,atm.} \qquad \text{(B1.2-6)}$$

B1.2.6.2 CO_2 concentration of the atmosphere $C_{S,atm.}$

(1) The actual CO_2 content in the atmosphere has been detected to be in a range of 350-380 ppm (parts per million). This corresponds with a concentration of 0.00057 up to 0.00062 kg/m^3. The standard deviation of the CO_2 content is almost constant with a maximum value of 10 ppm. By extrapolating the mean CO_2 concentration in the atmosphere based on Figure B1.2-4, the CO_2 concentration will increase by about 1.5 ppm per year.

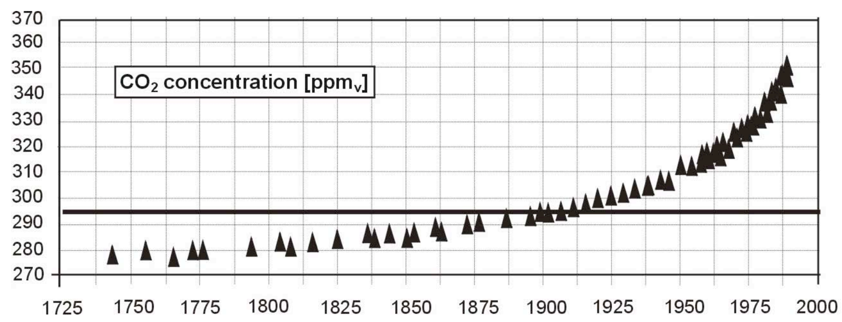

Figure B1.2-4: Progress of carbon dioxide concentration in the atmosphere, [7]

(2) Based on this estimated trend the atmospheric concentration of CO_2 can be quantified for simplification reasons as follows:

$C_{S,atm.}$ [kg/m^3]: normal distribution, m = 0.00082

s = 0.0001

B1.2.7 Weather function

B1.2.7.1 General

(1) The weather function W takes the meso-climatic conditions due to wetting events of the concrete surface into account, cp. Equation B1.2-7, cp. [5].

$$W = \left(\frac{t_0}{t}\right)^{\frac{(p_{SR} \cdot ToW)^{b_w}}{2}} = \left(\frac{t_0}{t}\right)^{w} \qquad \text{(B1.2-7)}$$

t_0: time of reference [years]

w: weather exponent [-]

ToW time of wetness [-], cp. Equation B1.2-8

$$ToW = \frac{\text{days with rainfall } h_{Nd} \geq 2.5 \text{ mm per year}}{365} \qquad \text{(B1.2-8)}$$

p_{SR}: probability of driving rain [-]

b_w: exponent of regression [-]

B1.2.7.2 Parameters describing rain events

(1) The effect of rain events on the concrete with respect to its carbonation resistance depends on the orientation and the geometrical characteristics of the structure. The following variables have to be quantified:

ToW (time of wetness)

p_{SR} (probability of driving rain)

(2) ToW **(T**ime **o**f **W**etness) is the average number of rainy days per year. A rainy day is defined by a minimum amount of precipitation water of h_{Nd} = 2.5 mm per day. The data for the quantification of ToW can be obtained by evaluation of data from the nearest weather station.

According to the explanation above the quantification of the variable ToW can be given as:

ToW [-]:	constant parameter, value:	to be evaluated from weather station data

(3) p_{SR} (probability of driving rain) is the average distribution of the wind direction during rain events. An evaluation can be carried out by determining the wind direction during rain events, based on data from the nearest weather station.

The quantification of the variable p_{SR} can be given as:

p_{SR} [-]:	constant parameter, value:	if vertical elements are treated p_{SR} has to be evaluated from weather station data
	constant parameter, value:	if horizontal elements are treated p_{SR} is equal to 1
	constant parameter, value:	if interior structural elements are treated p_{SR} is equal to 0

B1.2.7.3 Model variables b_W and t_0

(1) The weather function contains two model variables. One is the exponent of regression b_w and the other is the time of reference, t_o. These variables have been quantified as follows, cp. [5]:

b_w [-]:	normal distribution,	m =	0.446
		s =	0.163
t_o [years]:	constant parameter, value:		0.0767

B2 Full probabilistic design method for chloride induced corrosion – uncracked concrete

B2.1 Limit state equation for the depassivation of the reinforcement

(1) A full probabilistic design approach for the modelling of chloride induced corrosion in uncracked concrete has been developed within the research project DuraCrete and slightly revised in the research project DARTS, each project was funded by the European Union. It is based on the limit-state Equation B2.1-1, in which the critical chloride concentration C_{crit} is compared to the actual chloride concentration at the depth of the reinforcing steel at a time t $C(x = a, t)$.

$$C_{crit.} = C(x = a, t) = C_0 + (C_{S,\ddot{A}x} - C_0) \cdot \left[1 - \operatorname{erf} \frac{a - \ddot{A}x}{2 \cdot \sqrt{D_{app,C} \cdot t}}\right] \qquad \text{(B2.1-1)}$$

$C_{crit.}$: critical chloride content [wt.-%/c], cp. Chapter B2.2.6

$C(x,t)$: content of chlorides in the concrete at a depth x (structure surface: x = 0 m) and at time t [wt.-%/c]

C_0: initial chloride content of the concrete [wt.-%/c], cp. Chapter B2.2.4

$C_{S,\Delta x}$: chloride content at a depth Δx and a certain point of time t [wt.-%/c], cp. Chapter B2.2.5

x: depth with a corresponding content of chlorides C(x,t) [mm]

a: concrete cover [mm], cp. Chapter B1.2.1

Δx: depth of the convection zone (concrete layer, up to which the process of chloride penetration differs from Fick's 2nd law of diffusion) [mm], cp. (2) and Chapter B2.2.5

$D_{app,C}$: apparent coefficient of chloride diffusion through concrete [mm²/years], cp. Equation B2.1-2

The DuraCrete model is described in more detail in [3], the DARTS model (revised DuraCrete model) is described in [4], [5]. These models are in principle applicable both for marine environment and for de-icing salts on roads/bridges.

Fick's 2nd law for diffusion was first proposed for application in chloride exposed structures by M. Collepardi [8] in 1970.

In the early 1990s parallel efforts in different research communities did take place to improve this model. Such improved models are, in addition to the DuraCrete/DARTS model [3], [4], respectively, models developed by [6] and [9]. By the committee of this document, these models are regarded as useful as well.

As the details within this family of models slightly differ (e.g. in respect to the treatment of the surface layer), data derived by the use of one model is not directly applicable for use in the other models without a recalculation according to these differences.

At the time of publishing this document, alternative models for chloride ingress are under development and are expected to form alternatives to the above-mentioned models as soon as they are sufficiently validated against in-field performance.

Usually $D_{app,C}$ is determined by use of the “Chloride profiling method”. The determined $D_{app,C}$ is a constant average value representing the period from start of exposure to the moment of inspection when the profile is taken (time of interest). Chloride profiles can either be taken from existing structures or from test samples stored under conditions which are expected in practise. As the determination of $D_{app,C}$ on test samples (for the design of new structures) is very time consuming, a second, empirically derived approach is offered, cp. Equation B2.1-2.

The hereby applied rapid test methods are methods of convenience and should always be calibrated against the “chloride profiling method (under natural conditions)”, cp. [5].

For this reason hundreds of chloride profiles were collected from different sources and separated into different datagroups.

Profiles were collected separately. Profiles of cements mixed with different binders, different water/binder ratios were collected separately. In addition to that, the collected data was further separated into four exposure groups: Structures submerged, structures exposed to tidal action, structures exposed to chloride containing splash water and structural parts solely exposed to salt fog (spray zone).

t: time [years], cp. Chapter B1.2.2

erf: error function

(2) The model is based on Fick’s 2nd law of diffusion, taking into account that most observations indicate that transport of chlorides in concrete is diffusion controlled. Often the surface is often exposed to a frequent change of wetting and subsequent evaporation. This zone is usually referred to as the “convection zone”. As the transport mechanisms in this convection zone are not mainly diffusion controlled, the approach of Fick’s 2nd law of diffusion yields no satisfactory approximation for the chloride penetration inside the convection zone. In order to still describe the penetration of chlorides for an intermittent load using Fick’s 2nd law of diffusion, the data of the convection zone, which may deviate considerably from ideal diffusion behaviour, is neglected and Fick’s 2nd law of diffusion is applied starting at a depth Δx with a substitute surface concentration $C_{s,\Delta x}$. Δx marks the depth of the convection zone. With this simplification, Fick’s 2nd law of diffusion yields a good approximation of the chloride distribution at a depth $x \geq \Delta x$.

(3) The apparent coefficient of chloride diffusion of concrete can be determined by means of Equation B2.1-2, cp. [5]:

$$D_{app,C} = k_e \cdot D_{RCM,0} \cdot k_t \cdot A(t) \qquad \text{(B2.1-2)}$$

k_e: environmental transfer variable [-], cp. Equation B2.1-3

$$k_e = \exp\left(b_e\left(\frac{1}{T_{ref}} - \frac{1}{T_{real}}\right)\right) \qquad \text{(B2.1-3)}$$

b_e: regression variable [K], cp. Chapter B2.2.3

T_{ref}: standard test temperature [K], cp. Chapter B2.2.3.4

T_{real}: temperature of the structural element or the ambient air [K], cp. Chapter B2.2.3.3

$D_{RCM,0}$: chloride migration coefficient [mm²/a], cp. Chapter B2.2.1

k_t: transfer parameter [-], cp. Chapter B2.2.2

$A(t)$: subfunction considering the ‘ageing’ [-], cp. Equation B2.1-4

1. From these separately collected profiles $D_{app,C}$ was calculated back representing the time period between start of exposure and the moment of inspection with the corresponding local temperature regime, respectively.

2. The influence of the local temperature was considered by means of Equation B2.1-3. All collected apparent diffusion coefficients were converted to those based on T = 20°C. This step was made to make the data comparable.

3. Comparable cement mixes were tested by means of the rapid chloride migration test ($D_{RCM,0}$)

4. Separate regression analysis were made, each forced to the initial value of $D_{RCM,0}$ by best fitting through the collected apparent diffusion coefficients. The outcome of these regression analysis were quantified stochastic variables (ageing exponent).

5. Backwards taking the initial value of $D_{RCM,0}$ the environmental transfer variable k_c and the quantified ageing exponent into account (for the time being, the transfer variable k_t is set to $k_t = 1$) an apparent diffusion coefficient $D_{RCM,0}$ be calculated ($D_{RCM,0}$, k_e, a are stochastic values, $D_{app,C}$ is a stochastic value). This calculated value of $D_{app,C}$ again represents the time period between start of exposure and time of interest as a stochastic value, but constant in time.

$$A(t) = \left(\frac{t_0}{t}\right)^a \qquad \text{(B2.1-4)}$$

a: ageing exponent [-], cp. Chapter B2.2.2

t_0: reference point of time [years], cp. Chapter B2.2.2

B2.2 Quantification of parameters

B2.2.1 Chloride migration coefficient $D_{RCM,0}$

B2.2.1.1 General

While assessing existing structures the D_{app} might be derived directly from chloride profiles taken from the chloride exposed structure at different times.

When the D_{app} is derived from "chloride profiling method" (diffusion under "natural conditions"), the length of the exposure should be sufficiently long to get reliable data. A minimum duration of several months is recommended.

By profiling at different ages, information on the time-dependency ($D_{app,C}$) might also be obtained.

(1) The Chloride Migration Coefficient is one of the governing parameters for the description of the material properties in the chloride induced corrosion model. Suitable data for $D_{RCM,0}$ may be obtained from literature to be used as starting variables in a service life design calculation. When working with special concrete mixes with very low water/binder ratios and high contents of plasticiser, quantitative results from literature are usually not available. Therefore, it is essential to determine the efficiency of the materials to be used through basic tests, e.g. in order to identify the suitability of the designed concrete mix.

Various methods to assess the diffusion characteristics of concrete could be recommended, cp. [13]. The model described here has been developed upon the basis of the Rapid Chloride Migration Method (RCM).

(2) Among different rapid test methods, the Rapid Chloride Migration method (RCM) revealed to be theoretically clear, experimentally simple and related to precision (repeatability) promising tool.

B2.2.1.2 Rapid chloride migration method

See NT Build 492, [21].

B2.2.1.3 Quantification of the chloride migration coefficient $D_{RCM,0}$

(1) $D_{RCM,0}$ is a normally distributed variable with a mean value to be calculated according to Equation B2.2-3. The standard deviation of $D_{RCM,0}$ can be calculated according to Equation B2.2-6, [5].

$$s = 0.2 \cdot m \qquad \text{(B2.2-1)}$$

s: standart deviation of $D_{RCM,0}$

m: mean value of $D_{RCM,0}$

(2) $D_{RCM,0}$ should be quantified according to Chapter B2.2.1.2. If no test data is available, the following literature data can be used for orientation purposes, cp. Table B2-1, [5].

Table B2-1: Quantification of $D_{RCM,0}$ for different concrete mixtures, [5]

$D_{RCM,0}$ [m^2/s] / cement type	$w/c_{eqv.}$ [1]					
	0.35	0.40	0.45	0.50	0.55	0.60
CEM I 42.5 R	n.d.[2]	$8.9\cdot10^{-12}$	$10.0\cdot10^{-12}$	$15.8\cdot10^{-12}$	$19.7\cdot10^{-12}$	$25.0\cdot10^{-12}$
CEM I 42.5 R + FA (k = 0.5)	n.d.[2]	$5.6\cdot10^{-12}$	$6.9\cdot10^{-12}$	$9.0\cdot10^{-12}$	$10.9\cdot10^{-12}$	$14.9\cdot10^{-12}$
CEM I 42.5 R + SF (k = 2.0)	$4.4\cdot10^{-12}$	$4.8\cdot10^{-12}$	n.d.[2]	n.d.[2]	$5.3\cdot10^{-12}$	n.d.[2]
CEM III/B 42.5	n.d.[2]	$1.4\cdot10^{-12}$	$1.9\cdot10^{-12}$	$2.8\cdot10^{-12}$	$3.0\cdot10^{-12}$	$3.4\cdot10^{-12}$

[1] equivalent water cement ratio, hereby considering FA (fly ash) or SF (silica fume) with the respective k-value (efficiency factor). The considered contents were: 22 wt.-%/cement; SF: 5 wt.-%/cement.

[2] n.d. – chloride migration coefficient $D_{RCM,0}$ has not been determined for these concrete mixes

B2.2.1.4 Quantification of $D_{RCM,0}$ for orientation purposes

(1) The quantification of $D_{RCM,0}$ can be summarised as given below. Considerable attention has to be paid to the units, as until now $D_{RCM,0}$ has been determined using the unit [m^2/s]. When translating $D_{RCM,0}$ into the appropriate unit for the deterioration model [mm^2/years], a multiplication factor has to be applied.

$D_{RCM,0}$ [m^2/s]: normal distribution, m = values for orientation purpose: Table B2-1

s = m · 0.2 (cp. Equation B2.2-1)

B2.2.2 Transfer parameter k_t and ageing exponent a

B2.2.2.1 General

(1) The apparent diffusion coefficient $D_{app,C}$ is subject to considerable scatter and tends to reduce with increasing exposure time.

(2) Taking this into account when modelling the initiation process, a transfer parameter k_t in combination with a so-called ageing exponent a has been introduced.

B2.2.2.2 Quantification of a, k_t and t_0

(1) The functional relationship between exposure period t and diffusion coefficient $D_{app,C}$ for three different types of cement is illustrated in Table B2-2. Table B2-2 was derived for the exposure conditions ”splash zone”, ”tidal zone” and ”submerged zone”, but as an assumption on the safe side it can also be applied for “spray zone” and “atmospheric zone” exposure, [5].

The statistical quantities of the ageing exponent were determined as follows, for example for Portland cement concretes:

1. Published chloride profiling data ($D_{app,C}(t_i)$) of existing Portland cement concrete structures (comparable w/c ratio, e. g. $0.40 \leq w/c \leq 0.60$) exposed in conditions submerged/splash/tidal were collected (among others also data of [6]) and plotted vs. exposure time (temperature adjusted to reference temperature: T = 20 °C)

2. New concrete mixes (Portland cement, $0.40 \leq w/c \leq 0.60$) of comparable quality were tested with the RCM-method at the reference time t_0.

3. The spread of the RCM-test-results at the age t_0 was determined, RCM-results were plotted into the diagramme of published results.

4. A regression analysis was performed. The regression line was forced (boundary) through the data of new concretes, determined at time t_0.

The ageing exponent a corresponding to Equation B2.1-4 and Table B2-4 cannot be measured by the rapid test method RCM. RCM results of concretes tested at different ages will give an ageing exponent which do not fit to Equation 2.1-4 and Table B2-4, cp. [11]. Ageing determined with the RCM method represents only a certain portion of the total effect (increase of chloride penetration resistance due to ongoing hydration of concrete)

Table B2-2: Result of the statistical quantification of the variable a

concrete	ageing exponent a[5] [-]
Portland cement concrete CEM I; $0.40 \leq w/c \leq 0.60$	Beta (m^1=0.30; s^2= 0.12; a^3= 0.0; b^4= 1.0)
Portland fly ash cement concrete $f \geq 0.20 \cdot z$; $k = 0.50$; $0.40 \leq w/c_{eqv.} \leq 0.62$	Beta (m^1=0.60; s^2= 0.15; a^3= 0.0; b^4= 1.0)
Blast furnace slag cement concrete CEM III/B; $0.40 \leq w/c \leq 0.60$	Beta (m^1=0.45; s^2= 0.20; a^3= 0.0; b^4= 1.0)

[1] m: mean value

[2] s: standard deviation

[3] a: lower bound

[4] b: upper bound

[5] quantification can be applied for the exposure classes: splash zone, tidal zone and submerged zone

(2) To carry out the quantification of a, the transfer variable k_t was set to $k_t = 1$:

k_t [-]: constant, value: 1

Material performance can additionately be tested at a higher degree of maturity (i.e. $t_0 = 56$ d or $t_0 = 90$ d) to verify the positive age effect of puzzolanic additions on the penetration resistance of concrete. One have to keep in mind, that the time dependent decrease of D_{RCM} will only represent a certain portion (hydration) of the total ageing effect.

(3) The reference point of time was chosen to be $t_0 = 0.0767$ years ($t_0 = 28$ d).

t_0 [years]: constant, value: 0.0767

B2.2.3 Environmental transfer variable k_e

B2.2.3.1 General

(1) The environmental transfer variable k_e has been introduced in order to take the influence of T_{real} on the diffusion coefficient $D_{Eff,C}$ into account. The influence of T_{real} on the chloride diffusion coefficient is described by the Arrhenius-equation (Equation B2.2-7), cp. [5]

$$k_e = \exp(b_e(\frac{1}{T_{ref}} - \frac{1}{T_{real}})) \qquad (2.2\text{-}2)$$

k_e: environmental transfer variable [-]

b_e: regression variable [K]

T_{ref}: reference temperature [K]

T_{real}: temperature of the structural element or the ambient air [K]

(2) In order to determine the environmental transfer variable k_e mathematically according to Equation B2.2-7, T_{real}, T_{ref} (standard test temperature, T_{ref} = 293 K (20°C)) and the parameter b_e have to be determined.

B2.2.3.2 Regression variable b_e

Data was taken from Page [14] to quantify the regression variable b_e.

(1) The values of the regression varible b_e vary between b_e = 3500 K and b_e = 5500 K. b_e can be described as follows, cp. [5]:

b_e [K]: normal distribution, m = 4800
s = 700

B2.2.3.3 Temperature T_{real}

(1) The temperature of the structural element or the ambient air is described by means of the variable T_{real}. T_{real} can be determined by using available data from a weather station nearby.

T_{real} [K]: normal distribution, m = evaluated weather station data
s = evaluated weather station data

B2.2.3.4 Standard test temperature T_{ref}

(1) The standard test temperature T_{ref} has been defined as 293 K (= 20°C) and can be considered as constant.

T_{ref} [K]: constant parameter, value: 293

B2.2.4 Initial chloride content of the concrete C_0

(1) The chloride content in the concrete is not only caused by chloride ingress from the surface, but can also be due to chloride contaminated aggregates, cements or water used for the concrete production. Especially

when building in marine environment, the chloride content of fine and coarse aggregates and water can be considerable.

(2) In contrast to the chloride profiles resulting from chloride ingress from the surface, the distribution of the initial chloride content can be assumed to be uniform over the whole cross section

B2.2.5 Content of chlorides at the substitute surface $C_{s,\Delta x}$

B2.2.5.1 General

(1) The chloride content C_S at the concrete surface as well as the substitute surface content $C_{S,\Delta x}$ at a depth Δx are variables that depend on material properties and on geometrical and environmental conditions.

When assessing existing structures exposed to a chloride rich environment, the chloride concentration on the surface (or the substitute surface) might be derived directly from chloride profiles from the structure.

(2) Material properties that need to be taken into account are primarily the type of binder and the concrete composition itself.

(3) The most important variable describing the environmental impact is the equivalent chloride concentration of the ambient solution. Besides, the geometry of the structural element and the distance to the chloride source can be of significance.

In consequence the chloride content C_S at the concrete surface as well as the substitute surface chloride content $C_{S,\Delta x}$ are time dependent as well. However there are indications that these built-up periods are often relatively short. For long term predictions this time dependency is for practical reasons not included.

(4) All the variables mentioned above have a direct impact on the chloride content at the concrete surface and on the substitute surface content $C_{S,\Delta x}$. The information needed to determine C_S and $C_{S,\Delta x}$ is illustrated in the flow chart given in Figure B2.2-1, cp. [5].

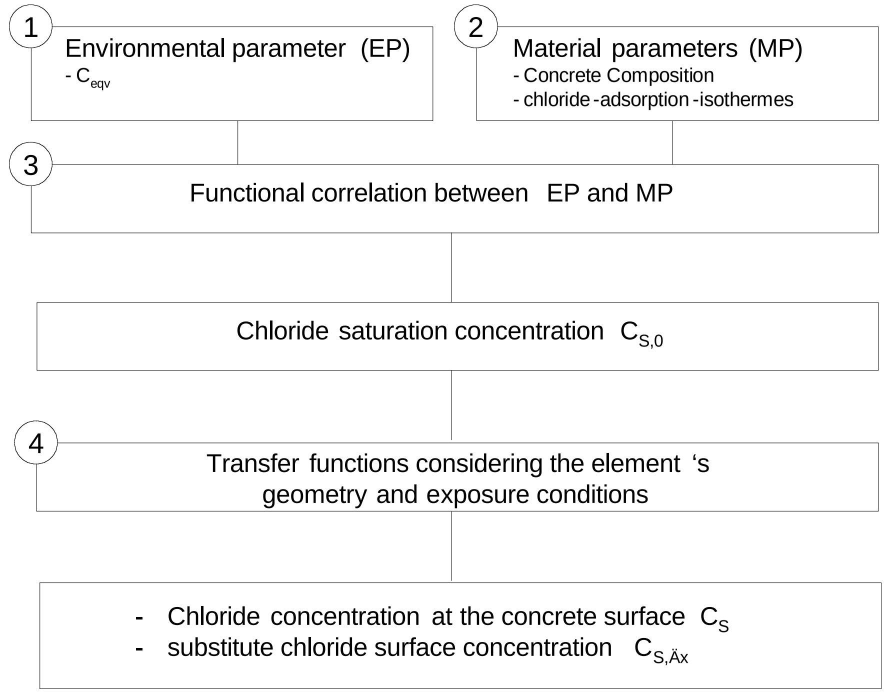

Figure B2.2-1: Information needed to determine the variables C_S and $C_{S,\Delta x}$

B2.2.5.2 Potential chloride impact C_{eqv}

(1) The potential chloride impact depends on the chloride content of the chloride source. For **marine** or **coastal structures** the potential chloride impact C_{eqv} is identical with the natural chloride content of sea water $C_{0,M}$, cp. Equation B2.2-3.

$$C_{eqv} = C_{0,M} \quad \text{(B2.2-3)}$$

C_{eqv}: potential chloride impact [g/l]

$C_{0,M}$: natural chloride content of sea water [g/l]

(2) The chloride concentration of chloride contaminated water due to **de-icing salt** $C_{0,R}$ presents a significantly larger variation than sea water with a comparable natural chloride content $C_{0,M}$. An adequate quantification of C_{eqv} turns out to be very complex, as for structures that are subjected to chloride impact due to de-icing salt, the variables describing the amount of de-icing salt applied are hard to quantify, cp. Equation B2.2-4.

$$C_{eqv} = C_{0,R} = \frac{n \cdot c_{R,i}}{h_{S,i}} \qquad \text{(B2.2-4)}$$

$C_{0,R}$: average chloride content of the chloride contaminated water [g/l]

n: average number of salting events per year [-]

$c_{R,i}$: average amount of chloride spread within one spreading event [g/m²]

$h_{S,i}$: amount of water from rain and melted snow per spreading period [l/m²]

B2.2.5.3 Material parameters

(1) The following material characteristics have to be determined in order to calculate the chloride saturation content $C_{S,0}$:

– chloride adsorption isotherms for the type of cement to be used

– concrete composition

(2) These characteristics have a pronounced influence on both the physical and the chemical binding capacity of the material and the pore volume that has to be saturated to the point where the chloride concentration in the pore solution is balanced with the exposure environment.

B2.2.5.4 Chloride saturation concentration $C_{S,0}$

Calculation is according Tang [10].

(1) Once the binder-specific chloride-adsorption-isotherms, the concrete composition and the order of magnitude of the impact level (potential chloride impact $C_{eqv.}$ [g/l]) are known, the chloride saturation concentration $C_{S,0}$ can be calculated.

(2) Figure B2.2-2 shows the correlation between $C_{S,0}$ and C_{eqv} for a Portland cement concrete (c = 300 kg/m³, w/c = 0.50). For C_{eqv} = 30 g/l, the chloride saturation concentration $C_{S,0}$ was determined to be $C_{S,0}$ = 2.78 wt.-%/cement.

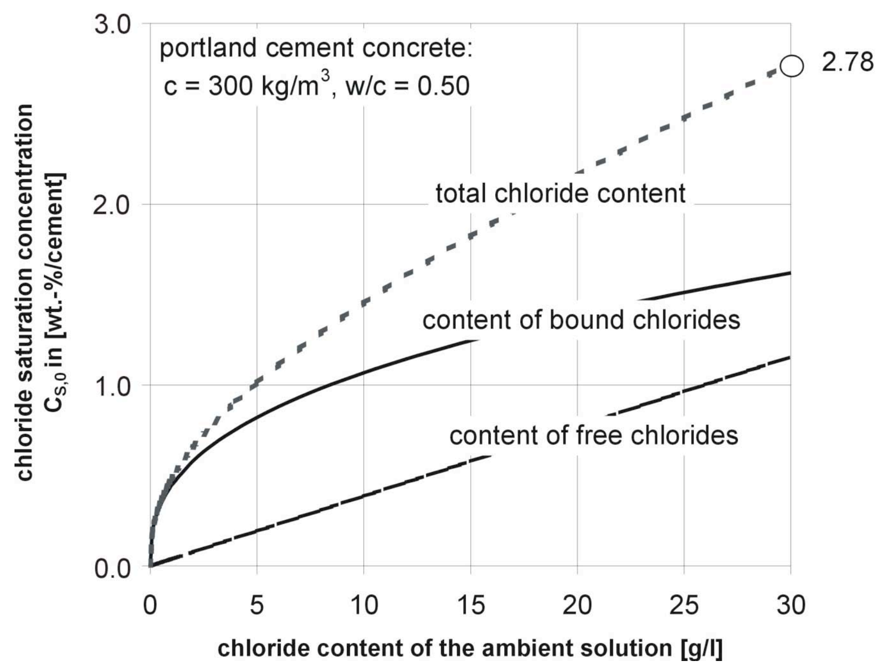

Figure B2.2-2: Surface chloride concentration $C_{S,0}$ in dependency on C_{eqv} for a Portland cement concrete

Especially in exposure environments where chlorides are not continuously affecting the structure time dependencies of C_S are observed. To get a non realistic view, a consideration of a time dependent variable C_S may be appropriate as soon as the corresponding modelling and quantification is validated.

(3) Under a continuous chloride impact of constant concentration, the chloride saturation concentration $C_{S,0}$ on the concrete surface is reached often in relative short time periods compared to the design service life ($C_{S,0} = C_S$) see for example [22]. Based on these results, the simplification that the variable C_S is from the beginning constant with time can be concluded e. g. for concrete continuously exposed to sea water. This simplification is on the safe side.

B2.2.5.5 Transfer function Δx

(1) If structural elements are intermittently exposed to a solution of constant or varying chloride concentration, transfer functions have to be formulated. A structural element which is intermittently loaded with a chloride-contaminated solution, interrupted by dry periods of air storage during which the water in the concrete close to the surface evaporates, any

subsequent re-wetting provokes a process of capillary suction. Compared to diffusion processes, capillary action leads to a rapid transport of chlorides into the concrete up to a depth Δx where the chlorides can accumulate with time until they create a saturation concentration $C_{S,\Delta x} = C_{S,0}$.

(2) The variable Δx can be described by a beta-distribution. Under **splash-conditions**, the average depth Δx up to which chlorides can rapidly penetrate can be limited to $6.0 \text{ mm} \leq \Delta x \leq 11.0 \text{ mm}$.

(3) In a distance to the road surface larger than 1.5 m (spray zone) the formation of a convection zone cannot be detected any more, $\Delta x = 0$.

(4) For parts of a structure which are constantly **submerged** the chloride surface concentration C_S is equal to the chloride saturation concentration which is developed rather spontaneously. Thus, for this special case no transfer function or transfer parameter is needed. In case the structure is exposed to **tidal conditions**, the depth Δx up to which a deviation from the diffusion behaviour according to Fick's solution exists has to be quantified.

(5) To summarise, for the different types of exposure conditions Δx can be quantified as follows:

Δx [mm]:	beta distributed	s = 5.6 m = 8.9 a = 0.0 b = 50.0	- for splash conditions (splash road environment, splash marine environment)
Δx [mm]:	constant parameter, value: 0		- for submerged marine structures - for leakage due to seawater and constant ground water level - for spray conditions (spray road environment, spray marine environment)
Δx [mm]:	beta distributed, m, s, a and b to be determined		- for leakage due to varying groundwater level - for tidal conditions

B2.2.5.6 Chloride surface content C_S resp. substitute chloride surface content $C_{S,\Delta x}$

(1) The chloride contamination of a structural element in the splash zone or in the spray zone increases with decreasing distance to the chloride source. This has been verified for both horizontal and vertical distances.

(2) Although $C_{S,\Delta x}(t)$ theoretically is a time-dependent variable, for simplification purposes it is going to be considered as time independent.

(3) For a structure of the following characteristics,

- location: urban and rural areas in Germany
- time of exposure of the considered structure: 5-40 years
- concrete: CEM I, w/c = 0.45 up to w/c = 0.60,

the maximum chloride content in the concrete C_{max} can be determined according to Equation B2.2-5, cp. [15]:

$$C_{max}(x_a, x_h) = 0.465 - 0.051 \cdot \ln(x_a + 1) - \left(0.00065 \cdot (x_a + 1)^{-0.187}\right) x_h \quad \text{(B2.2-5)}$$

C_{max}: maximum content of chlorides within the chloride profile, [wt.-%/concrete]

x_a: horizontal distance from the roadside [cm]

x_h: height above road surface [cm]

(4) Equation B2.2-5 was derived empirically for the conditions given above. For structures of different exposure or concrete mixes, an equivalent equation has to be determined.

(5) For structures under **splash** conditions, $C_{S,\Delta x}$ is defined as the maximum chloride content C_{max}. As tests yielded that for concrete at a height of more than 1.50 m above the road (**spray zone**) no Δx develops, C_{max} equals the chloride content at the concrete surface C_S. For these exposures $C_{S,\Delta x}$ resp. C_S can be quantified as follows:

$C_{S,\Delta x}$ resp. C_S [wt.-%/cement]: normal distribution,

m = cp. Equation B2.2-10 or equivalent

s = 0.75 m

For **submerged** structures, the surface content C_S is equal to the chloride saturation content $C_{S,0}$

B2.2.6 Critical chloride content C_{crit}

(1) In this context, the critical chloride content C_{crit} is defined as follows:

“The total chloride content which leads to the depassivation of the reinforcement surface and initiation of iron dissolution, irrespective of whether it leads to visible corrosion damage on the concrete surface.”

(2) The lower boundary of the variable C_{crit} has been specified as $C_{Crit,min}$ = 0.20 wt.-%/cement. As the lower boundary is known and differs from 0, it seems advisable to use a restricted distribution for the description of the critical chloride content causing corrosion. A beta-distribution with a lower boundary of $C_{Crit,min}$ = 0.20 wt.-%/cement yields a sufficiently good description of the test results, cp. [16]. The mean value of $C_{crit.}$ was set to $C_{Crit,m}$ = 0.60 wt.-%/c.

This value is recommended for ordinary mild steel. If another steel quality is used (e. g. stainless steel), mean value, standard deviation, lower and upper boundary of C_{crit} usually are on a higher level.

(3) The critical chloride content $C_{Crit.}$ can be quantified as follows:

C_{Crit} [wt.-%/cement]:	beta distributed,	m =	0.6
		s =	0.15
		a =	0.2
		b =	2.0

B3 Full probabilistic design method for frost induced internal damage – uncracked concrete

B3.1 Limit state equation for the frost damage of a unit cell

(1) A full probabilistic design approach for the modelling of frost induced internal damage of uncracked concrete has been developed within a series of research project. It is based on the limit-state Equation B3.1-1, in which the critical degree of saturation S_{CR} is compared to the actual degree of saturation $S_{ACT}(t)$ at a certain point of time t, during a certain target service life t_{SL}.

$$g\left(S_{CR}, S_{ACT}(t < t_{SL})\right) = S_{CR} - S_{ACT}(t < t_{SL}) \quad \text{(B3.1-1)}$$

S_{CR}: critical degree of saturation [-], cp. B3.2.1

t_{SL}: design service life [years], cp. B3.2.2

$S_{ACT}(t)$: actual degree of saturation at the time t [-], cp. B3.2.3

t: time [years]

(2) Equation B3.1-1 is based on water absorption into the air-void system as the prevailing transport mechanism within the concrete. It is assumed that the critical degree of saturation through the material is a constant material property, although the critical degree of saturation for a concrete during service life may be a function of numerous variables.

B3.2 Quantification of parameters

B3.2.1 Critical degree of saturation S_{CR}

B3.2.1.1 General

(1) The critical degree of saturation S_{CR} is determined from a laboratory test for the actual concrete

The critical degree of saturation S_{CR} for a particular concrete cannot be estimated from S_{CR} for another concrete. The absolute levels of S_{CR} for different concrete cannot be compared. S_{CR} can only be compared to the actual degree of saturation S_{ACT} for the same concrete.

In the test a series of specimens are vacuum saturated and dried to various degrees of saturation between 0.7 and 1.0. The specimens are sealed and frozen, once or with several freeze-thaw cycles. The dynamic E-modulus is determined for each specimen before and after the freeze-thaw cycles. Alternatively, the dilatation during one freeze-thaw cycle is measured for the series of specimens with different degrees of saturation. From the changes in E-modulus or dilatation as a function of degree of saturation, the critical degree of saturation is determined, where the frost damage starts to occur, cp. Figure B3.2-1.

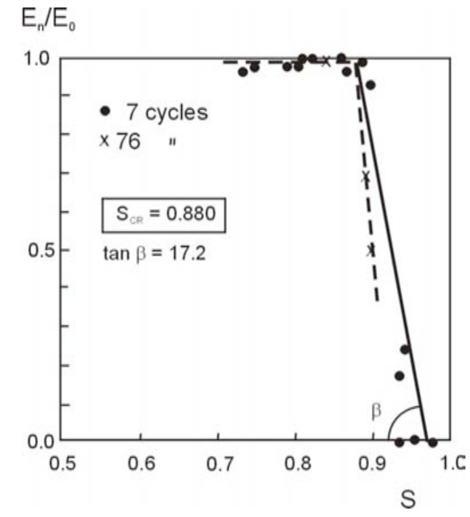

Figure B3.2-1: Example of determination of the critical degree of saturation by measuring the change in dynamic E-modulus after 7 or 76 freeze-thaw cycles,[17]

(2) The concrete composition, including the air-void system, is chosen during the design phase. Due to construction practices the actual concrete composition does vary and therefore has to be considered as a stochastic variable rather than a constant value. The following distribution types are in principle appropriate for the description of the critical degree of saturation and its variability:

- Normal distribution
- Beta-distribution
- Weibull(min)-distribution
- Lognormal distribution
- Neville distribution

B3.2.1.2 Quantification of S_{CR}

(1) distribution function:

Normal distribution

See Chapter B1.2.2.

B3.2.2 Design service life t_{SL}

B3.2.3 Actual degree of saturation S_{ACT}

B3.2.3.1 General

(1) The environmental action S_{ACT} considers the water absorption of the concrete, including the water absorption in the air-void system.

(2) The environmental action S_{ACT} can be described by means of Equation B3.2-1:

$$S_{ACT}(t < t_{SL}) = S_n + e \cdot t_{eq}^{d} \tag{B3.2-1}$$

t_{eq}: equivalent time of wetness [days], cf. B3.2.3.2

S_n, e, d: material parameters, exponents, respectively, cp. B3.2.3.3

B3.2.3.2 Equivalent time of suction t_{eq}

(1) The equivalent time of suction is completely dependent on the micro climate at the concrete surface, see Figure 3.2-2. Decisive parameters are how the surface is exposed to rain or splash, the frequency and duration and the conditions for drying. Table B3-1 gives provisional times of wetness for some important cases.

Table B3-1: Provisional equivalent times of wetness, [17]

Exposure	Equivalent time of wetness	Comments
Submerged surface	t_{SL}	
Horizontal surface	4 months	Surfaces, wet during a winter
Vertical surface[1]	1 week	Rain exposed surfaces that can dry out

[1] Orientation against prevailing driving rain direction and sunshine must be considered.

(2)

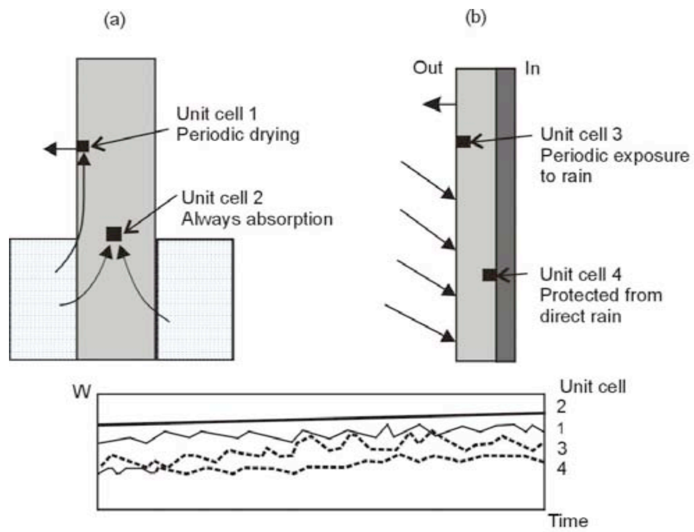

Figure B3.2-2: The actual moisture level in unit cells within the structure is different in different cells depending on their location and it varies over time. (a) Hydraulic structure constantly sucking water. (b) Façade element periodically exposed to rain, [17]

B3.2.3.3 Material parameters S_n, e and d

(1) The material parameters S_n, e and d describes the water absorption characteristics of the concrete when exposed to water. The parameters are determined for the actual concrete with a long term capillary suction test

S_n is the degree of saturation at the knick point in a √t-scale diagram.

Parameters e and d describes the slope of the water absorption after the knick point, in a log-scale diagram.

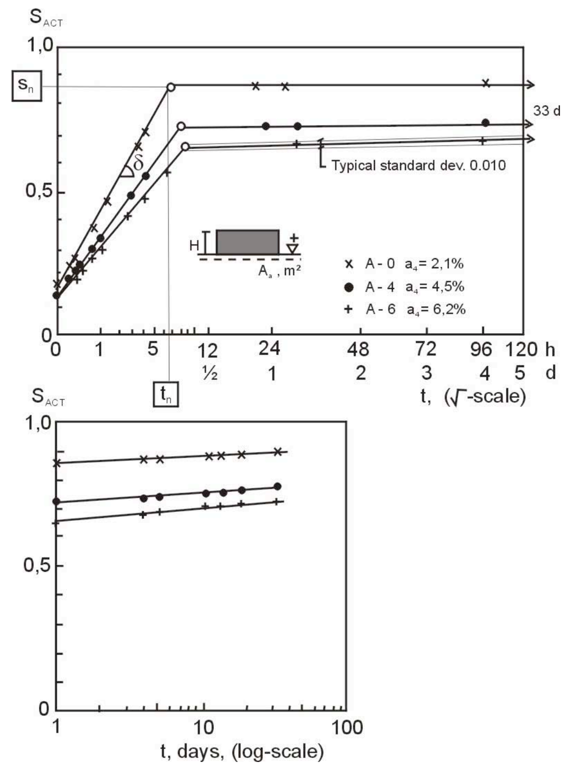

Figure B3.2-3: Results from a test of short and long term water absorption, [17]

B3.2.4 Loss of mechanical properties due to internal frost damage

B3.2.4.1 General

(1) For the model introduced above, the consequences of internal frost damage is included in the traditional design process, with changes of mechanical material properties, such as elastic modulus, strength and bond strength between concrete and reinforcement.

B4 Full probabilistic design method for salt-frost induced surface scaling – uncracked concrete

B4.1 Limit state equation for the salt-frost induced surface scaling

(1) A full probabilistic design approach for the modelling of salt-frost induced surface scaling of uncracked concrete is based on the limit-state Equation B4.1-1, in which the concrete temperature T(t) is compared to the scaling resistance $T_R(t)$ at a certain point of time t, during a certain design service life t_{SL}.

$$g\left(T, T_R\,(t < t_{SL})\right) = T(t \leq t_{SL}, Cl) - T_R\,(RH(T), T(t),..) \qquad \text{(B4.1-1)}$$

T(t): concrete temperature [K], cp. B4.2.1

t_{SL}: design service life [years], cp. B4.2.2

$T_R(t)$: critical freezing temperature for scaling to occur at the time t [-], cp. B4.2.3

t time [years]

(2) Equation B4.1-1 is based on the assumption that scaling occurs in the same moment as the concrete surface temperature falls below a certain, critical level, the scaling resistance T_R. It is assumed that this critical level of scaling resistance changes with age, depending on exposure and type of concrete.

B4.2 Quantification of parameters

B4.2.1 Scaling resistance T_R (The critical freezing temperature for scaling to occur)

(1) The critical freezing temperature for scaling to occur, the scaling resistance T_R, is determined from a laboratory test for the actual concrete, at an age of 28 days. The accepted degree of scaling must be defined before the test.

The scaling test is performed at three temperature levels.

(2) The concrete composition, including the air-void system, is chosen during the design phase. Due to construction practices the actual concrete composition does vary and therefore has to be considered as a stochastic variable rather than a constant value. The following distribution types are in principle appropriate for the description of the scaling resistance and its variability:

- Normal distribution
- Beta-distribution
- Weibull(min)-distribution
- Lognormal distribution
- Neville distribution

The scaling resistance T_R will change with age and exposure. This change with time will be different for different types of concrete, cp. Figure B4.2-1.

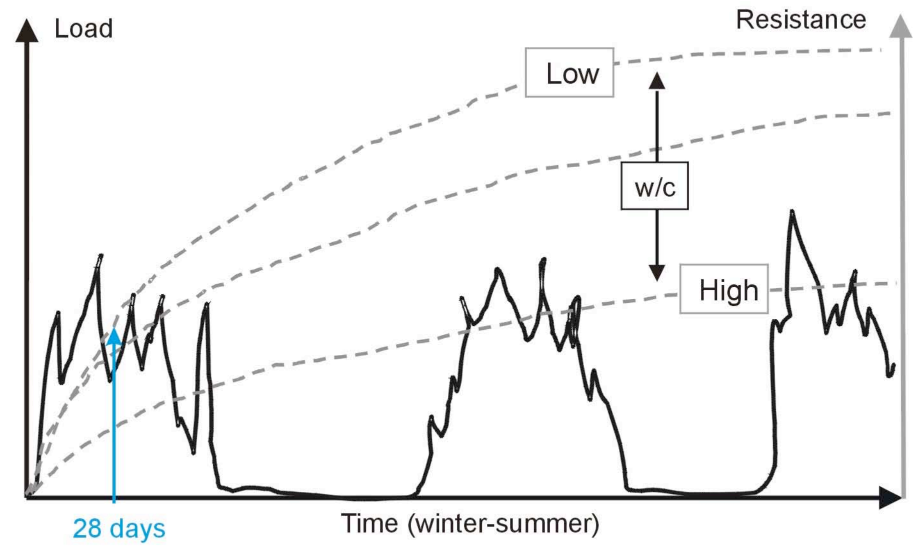

Figure B4.2-1: Principle sketch of the concrete temperature ("load"), vertical scale with negative temperatures upwards, during three winters, compared to the true scaling resistance ("resistance") as a function of time for three different concretes, [18]

B4.2.2 Design service life t_{SL}

See Chapter B1.2.2.

B4.2.3 Actual concrete temperature T(t)

B4.2.3.1 General

(1) The environmental action T(t), the actual concrete temperature mainly during clear winter nights, considers the air temperature, convection due to wind and the long wave radiation during clear nights. The decisive concrete temperature is for nights only when salt is present at the concrete surface.

(2) The environmental action, the concrete surface temperature T(t) can be described by means of Equation B4.2-1:

$$T(t < t_{SL}) = T_{air} + \frac{\alpha_r}{\alpha_r + \alpha_{cv}} \left(T_{sky} - T_{air}\right) \qquad \text{(B4.2-1)}$$

T_{air}:	air temperature [K], cp. B4.2.3.2
α_r:	surface heat conductance due to radiation [W/(m^{2}K)], cp. B4.2.3.3
α_{cv}:	surface heat conductance due to convection[W/(m^{2}K)], cp. B4.2.3.3
T_{sky}:	corresponding temperature of space [K], cp. B4.2.3.4

B4.2.3.2 Air temperature T_{air}

(1) Data of the nearest weather station may be used as an input for T_{air}. For quantification, the extreme weather station data (cold, clear winter nights) has to be evaluated.

(2) For European climate conditions, a normal distribution is in general appropriate to describe T_{air}.

B4.2.3.3 Surface heat conductance α_r and α_{cv}

(1) The surface heat conductance α_r for radiation depends on the concrete surface temperature and the corresponding temperature of space and the emissivity ε of the concrete surface.

$$\alpha_r = 4 \cdot \varepsilon \cdot \sigma \frac{(T_{sky} - T_{air})^3}{2} \qquad [W/(m^2K)] \qquad \text{(B4.2-2)}$$

where $\sigma = 5.67 \cdot 10^{-8} [W/(m^2K^4)]$ (the Stefan-Bolzmann number). For concrete the emissivity $\varepsilon = 0.9$

(2) The surface heat conductance α_{cv} for convection depends on the wind velocity close to the concrete surface. For cases with wind speed **u** below 5 m/s, a value can be

$$\alpha_{cv} = 6 + 4 \cdot u \qquad [W/(m^2K)] \qquad \text{(B4.2-3)}$$

B4.2.3.4 Corresponding sky temperature T_{sky}

(1) The corresponding temperature of the sky for the long-wave radiation from a concrete surface depends on the orientation of the surface, cloudiness and “shadows” from other buildings, cp. Figure B4.2-2.

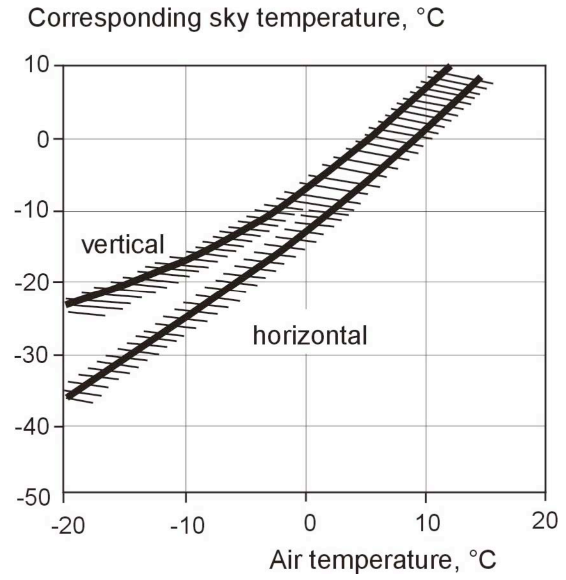

Figure B4.2-2: The corresponding sky temperature for different surfaces, depending on the air temperature.

(2) The corresponding temperature of the sky for the long-wave radiation from a concrete surface in Figure B4.2-2 can be estimated from Equation B4.2-4.

$$T_{sky} = \begin{cases} 1.2 \cdot T_{air} - 14 & \text{for horizontal surfaces – clear sky} \\ 1.1 \cdot T_{air} - 5 & \text{for vertical surfaces – clear sky} \\ T_{air} & \text{for a cloudy sky} \end{cases} \qquad \text{(B4.2-4)}$$

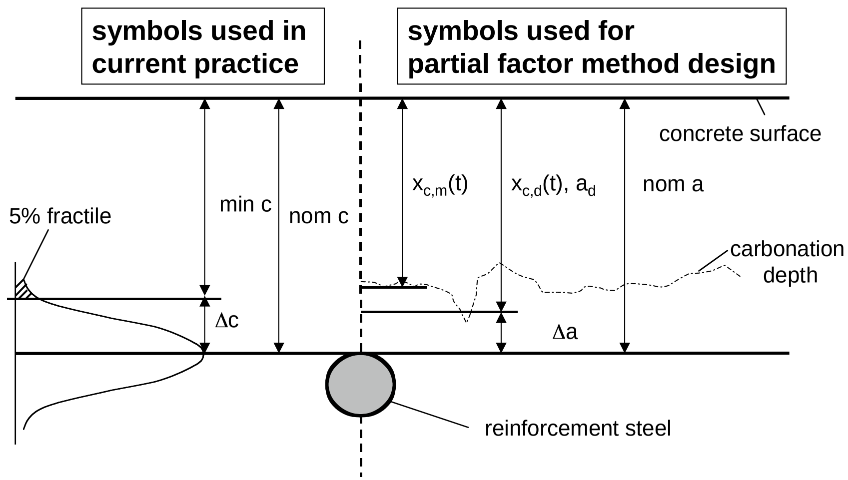

Figure C1.1-1: Symbols used in current practice (left hand side) and used within the partial factor method design (right hand side)

min c: minimum concrete cover [mm]

nom c: nominal concrete cover [mm]

Δc: margin between minimum and nominal concrete cover [mm]

$x_{c,m}(t)$ mean value of the carbonation depth at the time t [mm]

$x_{c,d}(t)$ design value of the carbonation depth at the time t [mm]

Annex C (informative)

Partial factor methods

C1 Partial factor method for carbonation induced corrosion - uncracked concrete

C1.1 Limit state equation including partial factors for the depassivation of the reinforcement

(1) The partial factor method for carbonation induced corrosion in uncracked concrete introduced in this chapter is based on the full probabilistic design approach presented in Chapter B1, Annex B.

a_d: design value of the concrete cover [mm]

Δa: safety margin of the concrete cover [mm]

nom a: nominal concrete cover [mm]

To determine the partial safety factors according ISO 2394 the governing variables have to determined. For this reason a parameter study including three different design examples was carried out by means of the software package STRUREL, [12]. The influence of all variables on the calculated reliability for carbonation induced corrosion is given by the corresponding α_i value. Figure C1.1-2 shows the result.

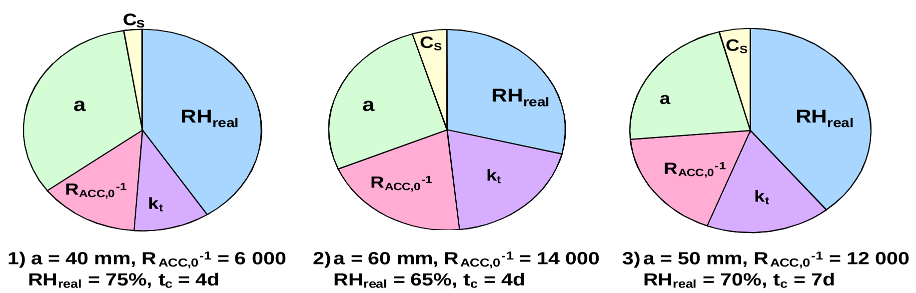

Figure C1.1-2: α_i values for the variables to calculate the limit state of depassivation due to carbonation

(2) The aim of the partial factor method is to enable a durability design for carbonation induced corrosion that can be carried out as a simple calculation without additional considerations concerning the probabilistic distributions of input parameters.

(3) A structural element meets the requirements concerning its durability with respect to carbonation induced corrosion if limit state Equation C1.1-1 is fulfilled:

$$a_d - x_{c,d}(t_{SL}) \geq 0 \quad \text{(C1.1-1)}$$

$$x_{c,d}(t_{SL}) = \sqrt{2 \cdot k_{e,d} \cdot k_{c,d} \cdot (k_{t,d} \cdot R^{-1}_{ACC,0,k} \cdot \tilde{a}_R + \mathring{a}_{t,d}) \cdot C_{S,d}} \cdot \sqrt{t_{SL}} \cdot W(t_{SL}) \quad \text{(C1.1-2)}$$

a_d: design value of the concrete cover [mm], cp. Chapter B1.2.1

$$a_d = \text{nom a} - \Delta a = \text{nom c} - \Delta a \quad \text{(C1.1-3)}$$

nom c: nominal concrete cover [mm]

nom a: nominal concrete cover [mm]

Δa: safety margin of the concrete cover [mm]

$\Delta a = 10$ mm

t_{SL}: design service life [years], cp. Chapter B1.2.2

$x_{c,d}(t_{SL})$ design value of the carbonation depth at the time t_{SL} [mm]

$k_{e,d}$: design value of the environmental function [-], cp. Chapter B1.2.3

It can be seen that the three governing parameters of Equation B1.1-2 are the following:

1. RH_{real} (k_e)
2. a
3. $R_{ACC,0}^{-1}$

For these three parameters partial safety factors γ_{RH}, γ_R and the safety margin Δa are introduced.

The partial safety factors given in this paragraph are to be considered as preliminary and will probably be changed. They are linked to a service life of t_{SL} = 50 years and to a reliability index of β = 1.3.

$$k_{e,d} = \left(\frac{1 - \left(\frac{RH_{real,k}}{\gamma_{RH} \cdot 100} \right)^{f_e}}{1 - \left(\frac{RH_{ref}}{100} \right)^{f_e}} \right)^{g_e} \quad \text{(C1.1-4)}$$

$RH_{real,k}$: characteristic value of relative humidity of the carbonated layer [%], cp. Chapter B1.2.3, here: mean value of RH_{real}

RH_{ref}: reference relative humidity [%], cp. Chapter B1.2.3
RH_{ref} = 65 %

f_e: exponent [-],cp. Chapter B1.2.3
f_e = 5.0

g_e: exponent [-],cp. Chapter B1.2.3
g_e = 2.5

γ_{RH}: partial safety factor for the relative humidity RH_{real} [-],
γ_{RH} = 1.3

$k_{c,d}$: design value of the execution transfer parameter [-], cp. Chapter B1.2.4 and Table C1-1, here: mean value of k_c

Table C1-1: Execution transfer parameter $k_{c,d}$ for different curing periods t_c

curing period d_c [d]	1	2	3	4	5	6	7	8	9	10	11	12	13	14
$k_{c,d}$	3.00	2.03	1.61	1.37	1.20	1.09	1.00	0.92	0.86	0.81	0.77	0.73	0.70	0.67

$k_{t,d}$: design value of the regression parameter [-], cp. Chapter B1.2.5, here: mean value of k_t; $k_{t,d}$ = 1.25

$R_{ACC,0,k}^{-1}$: characteristic value of the inverse effective carbonation resistance of concrete [(mm²/years)/(kg/m³)], cp. Chapter B1.2.5; here: mean value of $R_{ACC,0}^{-1}$

γ_R: partial safety factor for the inverse carbonation resistance of concrete $R_{ACC,0,k}^{-1}$ [-] $\gamma_R = 1.5$

$\varepsilon_{t,d}$: design value of the error term, cp. Chapter B1.2.5, here: mean value of ε_t, $\varepsilon_{t,d} = 315.5$

$C_{S,d}$: design value of the CO_2-concentration [kg/m³], cp. B1.2.6, here: mean value of C_S; $C_{S,d} = 0.00082$

W(t): weather function [-], cp. Chapter B1.2.7 and Equation C1.1-4

$$W = \left(\frac{t_0}{t}\right)^{\frac{(p_{SR} \cdot ToW)^{b_{w,d}}}{2}} \qquad \text{(C1.1-5)}$$

t_0: time of reference [years], $t_0 = 0.0767$

ToW time of wetness [-], cp. Equation C1.1-5

$$ToW = \frac{\text{days with rainfall } h_{Nd} \geq 2.5 \text{ mm per year}}{365} \qquad \text{(C1.1-6)}$$

p_{SR}: probability of driving rain [-]

$b_{w,d}$: design value of the exponent of regression [-],

here: mean value of b_W; $b_{W,d} = 0.446$

As soon as ultimate limit states (ULS) are assessed, the propagation period has to be taken into account, other resistance variables become dominant, e. g. an added sacrificial cross section and a higher reliability is required (ULS-level).

(5) The partial safety factors γ_R and γ_{RH} and the safety margin Δa have been quantified for a SLS reliability of $\beta = 1.3$ with respect to the limit state "depassivation of reinforcement due to carbonation, SLS". If a higher reliability is desired, the partial safety factors have to be modified accordingly.

(6) The partial factor method includes simplifications of the full probabilistic approach on the safe side. Therefore, the use of the full probabilistic method can lead to more economical solutions, but it requires considerably larger expenses for the quantification of the input parameters and the calculation itself.

C1.2 Example

The following input data with regard to environment, concrete diffusion characteristics, and curing was collected.

Table C1-2: Input data, partial safety factor approach

Parameter	**Unit**	**Input data**
$RH_{real,k}$	[% rel. humidity]	80
γ_{RH}	[-]	1.3
$k_{c,d}$	[-]	1.61
$R_{ACC,0,k}^{-1}$	[(mm?/year)/(kg/m?)]	4500
γ_R	[-]	1.5
$C_{S,d}$	[kg/m?]	$8.2 \cdot 10^{-4}$
t_{SL}	[years]	1-50 (parameter study)
p_{SR}	[-]	0.1
ToW	[-]	0.27
Δa	[mm]	10

With these data, equation (C1.1-2) can be solved:

$$x_{c,d}(t_{SL}) = \sqrt{\left(2 \cdot \left(\frac{1-\left(\frac{80}{1.3 \cdot 100}\right)^5}{1-\left(\frac{65}{100}\right)^5}\right)^{2.5} \cdot 1.61 \cdot (1.25 \cdot 1.5 \cdot 4500 + 315.5) \cdot 8.2 \cdot 10^{-4}\right)} \cdot \sqrt{t_{SL}} \cdot \left(\frac{0.0767}{t_{SL}}\right)^{\frac{(0.1 \cdot 0.27)^{0.446}}{2}}$$

With the time dependent design value of the carbonation depth $x_{c,d}(t_{SL})$ the nominal concrete cover nom a can be calculated, cp. Figure C1.2-1.

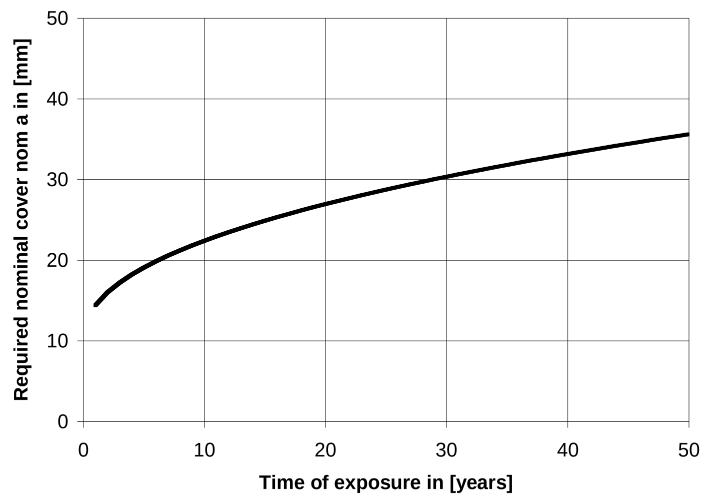

Figure C1.2-1: Required nominal concrete cover nom a with time of exposure, exposure carbonation, middle European climate, cyclic wet and dry, exposed to driving rain (vertical reinforced concrete fassade), CEM I - concrete, w/c = 0.60

C2 Partial factor method for frost induced damage - uncracked concrete

(1) The partial factor method for frost induced damage in uncracked concrete introduced in this chapter is based on the full probabilistic design approach presented in Chapter B3, Annex B.

(2) The aim of the partial factor method is to enable a durability design for frost induced damage that can be carried out as a simple calculation without additional considerations concerning the probabilistic distributions of input parameters.

(2) The following limit state function needs to be fulfilled:

$$S_{CR,d} - \Delta S_{cr} - (S_{ACT,d}(t < t_{SL}) + \Delta S_{ACT}) \geq 0 \qquad \text{(C1.2-1)}$$

$S_{CR,d}$:	design value of the critical degree of saturation [-]
$S_{ACT,d}(t < t_{SL})$:	design value of the actual degree of saturation at time t [-]
t_{SL}:	service life [years]
ΔS_{CR}:	margin of the critical degree of saturation [-]
ΔS_{ACT}:	margin of the critical degree of saturation [-]

Annex R (informative)
Reliability management: from SLS to ULS

R1 General

According to Tuutti [19] the process of reinforcement corrosion can be roughly divided into two time periods, cp. Figure R1.1-1:

- Initiation period
- Propagation period

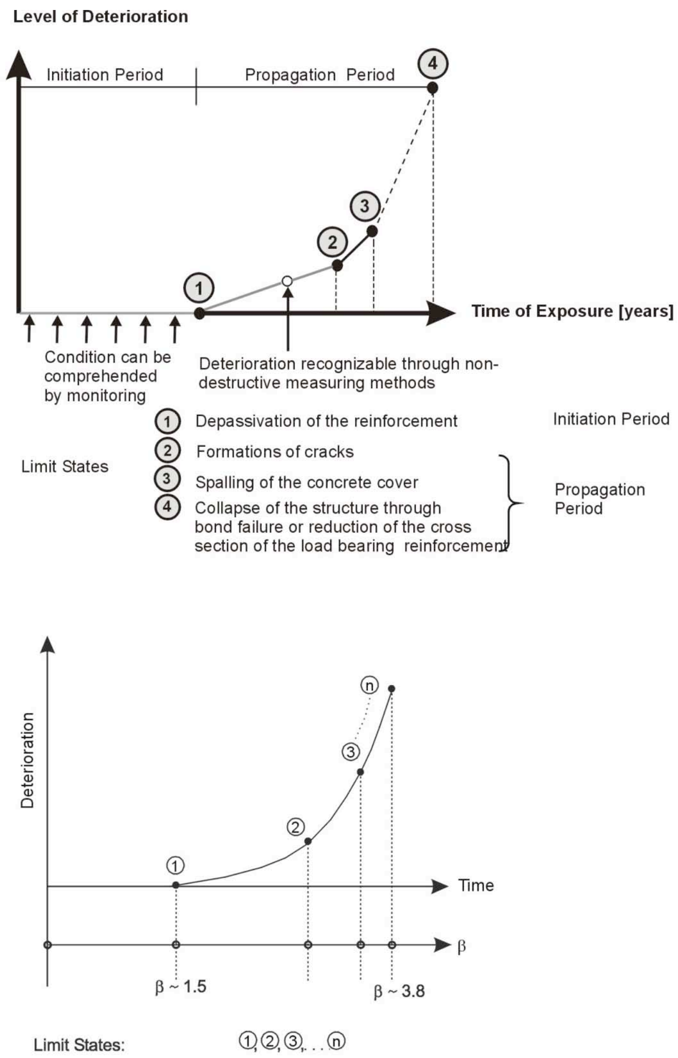

Figure R1.1-1: Deterioration process of reinforcement corrosion and definition of limit states for basic scheme of the service life design

The initiation period is defined as the time until the reinforcement becomes depassivated either by carbonation or by penetration of chlorides. This period does not harm the concrete and/or the reinforcement itself. As soon as the concrete at the depth of steel (outer reinforcement) is carbonated or containing a certain amount of free chlorides the reinforcement becomes depassivated. The end of the initiation period is reached and corrosion (under certain circumstances) is possible, cp. Figure R1.1-1.

During the propagation period the reinforcement itself is affected which may lead to deterioration of the concrete as well. In case of expanding corrosion products of the reinforcement cracks along the reinforcing element are provoked which subsequently leads to spalling of the concrete cover. Finally the loss of cross section of the reinforcement may lead to reduction of the load bearing capacity. ULS is defined by the relevant failure mode of the section and may be reached by cracking or spalling (failure of anchorage) or by inadmissible loss of cross section. This has been also illustrated right hand side in Figure R1.1-1.

For the assessment of a structure towards a certain event (limit state) the respective time periods can be added up and compared with the time period of interest, which is in most cases the service life t_{SL}, cp. Equation R1.1-1.

$$t_{SL} = t_{ini} + t_{prop,i} \qquad \text{(R1.1-1)}$$

t_{SL}: service life [years]

t_{ini}: time period of initiation [years]

$t_{prop,i}$: time period of propagation till the treated event i(cracking, spalling, collapse) occurs [years]

R2 Reliability management

It is assumed, that the usual design of reinforced and pre-stressed structures is made in that way, that the ULS requirements of Annex A, Table A2-2 are fulfilled exactly. Corrosion of reinforcement (pre-stressing steel) and/or deterioration of concrete (bond failure, lack of sufficient compressive cross section, will decrease the reliability.

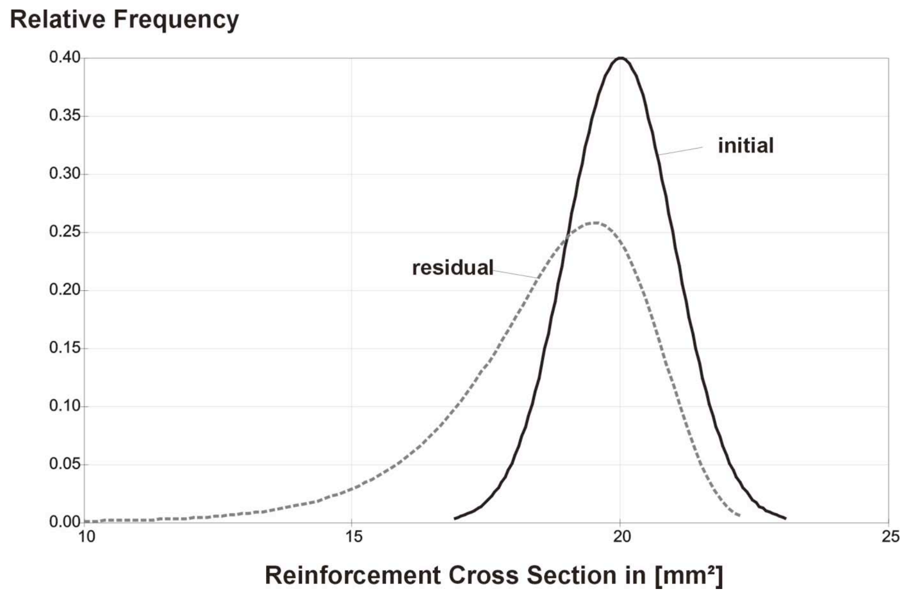

Figure R1.1-1: Density function of initial and residual (after corrosion) reinforcement cross section

If corrosion of reinforcement can not be excluded at a ULS reliability and inspection/maintenance/repair that means “intervention” can not be executed, in every case this will lead to the need of extra reinforcement (sacrificial cross section) and, depending on the expected failure mode, special detailing in order to avoid bond failure within the bonding zone.

The dimension of this extra cross section which influences the length of the propagation period decisively highly depends on the reliability depassivation is excluded. The length of the period until the event depassivation occurs (initiation period) is decisively influenced by the concrete quality and the magnitude of concrete cover. As both periods participates to the service life both influencing variables (extra reinforcement on the one hand side and concrete quality and concrete cover on the other side) can be traded off. That means, the higher the reliability with regard to depassivation the lower the need of extra reinforcement.

In every case, without any exception, to balance the effect of corrosion the extra cross section is equal to the corroded part of the initial cross section, cp. Figure R1.1-1, $\Delta A_{S,Corr}$.

However in some cases it is not sufficient to balance only the effect of corrosion. If spalling is considered as an ULS (Annex A, Table A3-1, ROC1), beside balancing the effect of corrosion spalling have to be avoided. This have to be done by restricting the allowable total loss of cross section and the permissible maximum extra cross section, respectively. This inevitably leads to a restriction of the propagation period. Consequently the initiation period have to be prolonged, that means depassivation have to be avoided on a higher reliability level to avoid ULS at the required reliability.

In the following a procedure is described how to quantify the needed extra reinforcement.

The procedure is as follows:

In a first step the length of an initiation period is calculated on basis of a specific data set. The initiation period was set to be over as soon the minimum SLS reliability of Annex A, Table A2-2 is not fulfilled anymore.

Common agreed models to describe the propagation period do not exist. To gap this problem a Delphic round was organized by the Taskgroup 5.6 and experts all over the world gave their opinion on expected penetration depths and propagation periods in a specific carbonation environment.

This data was evaluated in a second step. In a third step the needed extra cross section was determined which is needed to ensure the required ULS reliability (here: failure mode according to Annex A, Table A3-1, ROC3).

R3 Initiation Period

R3.1 Model to calculate the initiation period t_{ini} (Exposition XC4)

The predicted depth of carbonation at the end of service life $x_c(t_{SL})$ has to be compared with the concrete cover a in order to obtain a prediction about the reliability in case of carbonation induced corrosion. This leads to the following limit state equation for the initiation period:

First Notation for Limit State: Depassivation

$$g(a, x_c(t_{SL})) = a - x_c(t_{SL}) \qquad \text{(R3.1-1)}$$

$$= a - \sqrt{2 \cdot k_e \cdot k_c \cdot (k_t \cdot R_{ACC,0}^{-1} + \mathring{a}_t) \cdot C_S} \cdot \sqrt{t_{SL}} \cdot W(t_{SL})$$

a: concrete cover [mm]

$x_c(t_{SL})$: carbonation depth at the end of service life t_{SL} [mm]

By converting Equation R3.1-1 to compare the time period of initiation with the service life according to Equation R1.1-1 the following notation can be used, cp. Equation R3.1-2. (The first and the second notation are equivalent.)

Second Notation for Limit State: Depassivation

$$g(t_{ini}, t_{SL}) = t_{ini} - t_{SL} \qquad \text{(R3.1-2)}$$

$$= \left(\frac{2 \cdot k_e \cdot k_c \cdot \left(k_t \cdot R_{ACC,0}{}^{-1} + \mathring{a}_t\right) C_S}{a^2} \cdot t_0{}^{2 \cdot w} \right)^{\left(\frac{1}{2 \cdot w - 1}\right)} - t_{SL}$$

t_{ini}: time period of initiation due to carbonation [years]

t_{SL}: service life [years]

R3.2 Evaluation of the initiation period t_{ini} (Exposition XC4)

To calculate the reliability against depassivation of reinforcement for chosen environmental conditions and material properties ($R_{ACC,0}{}^{-1}$; a) the limit state (either Equation R3.1-1 or Equation R3.1-2) has to be evaluated. Hereby the variables describing the environmental conditions have been quantified in accordance to the exposition class XC4. The material properties have been chosen in such a way, that the reliability index linked to depassivation of the reinforcement after 50 years of exposure reaches $\beta_{depassivation} = 1.3$. An illustrative overview of all quantified variables is given in Table R3-1. The evaluation of the limit state function can be carried out with computer programs like e.g. [12].

Table R3-1: Overview of quantified variables to describe the duration of the initiation period for the exposition XC4

No.	Variable	Unit	Distribution	Mean Value	Standard Deviation
1	RH_{real} (k_c)	[%]	beta distribution	m = 80; s = 10 a = 40; b = 100	
1	Rh_{ref} (k_c)	[%]	constant	65	-
1		[-]	constant	2.5	-
1		[-]	constant	5.0	-
2	b_c (k_c)	[-]	normal distribution	-0.567	0.024
2	t_c (k_c)	[d]	normal distribution	4	-
3	k_t	[-]	normal distribution	1.25	0.35
4	$R_{ACC,0}^{-1}$	[(m²/s)/(kg/m³)] ([(mm²/years)/(kg/m³)])	normal distribution	$23 \cdot 10^{-11}$ (7,300)	$8 \cdot 10^{-11}$ (2,500)
5	ε_t	[(m²/s)/(kg/m³)] ([(mm²/years)/(kg/m³)])	normal distribution	$1.0 \cdot 10^{-11}$ (315.5)	$0.15 \cdot 10^{-11}$ (48)
6	C_s	[kg/m³]	normal distribution	$8.2 \cdot 10^{-4}$	$1.0 \cdot 10^{-4}$
7	t	[years]	constant	50	-
8	ToW (W)	[-]	constant	0.2	[-]
8	b_w (W)	[-]	normal distribution	0.446	0.163
8	p_{SR} (W)	[-]	constant	0.1	-
8	t_0 (W)	[years]	constant	0.0767	-
9	a	[mm]	normal distribution	25	8

Furthermore for demonstration reasons also a parameter study over the initiation period $t_{ini,}$ with quantities for the required variables as given in Table R3-1 has been carried out with [12]. The parameter study over the variable t_{ini} came up with the result as outlined in Figure R3.2-1.

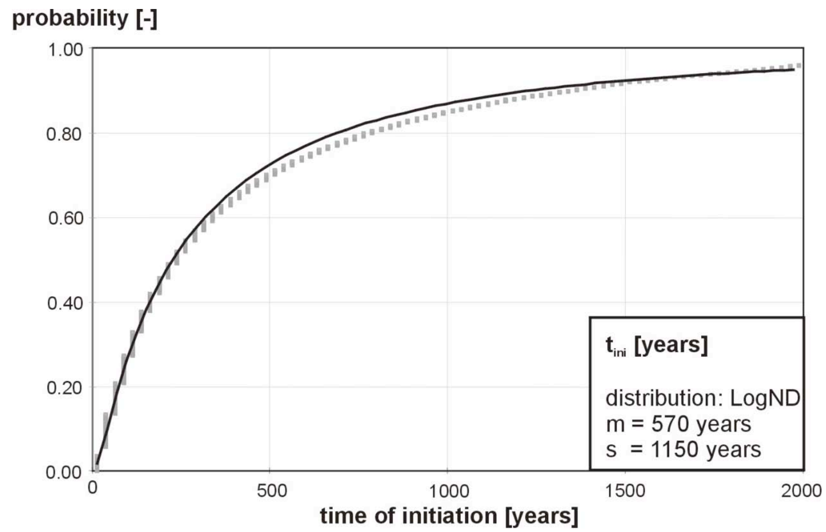

Figure R3.2-1: Cumulative frequency of t_{ini} as a result of the parameter study over the duration of the initiation period (variables according to Table R3-1)

R4 Propagation Period

R4.1 Limit State

To describe the limit states linked to the propagation period according to Equation R1.1-1 the following Equation R4.1-1 can be evaluated in terms of g (…) < 0.

Limit State Equation Based on Second Notation

$$g(t_{ini}, t_{prop,i}, t_{SL}) = t_{ini} + t_{prop,i} - t_{SL} \tag{R4.1-1}$$

with: $$t_{ini} = \left(\frac{2 \cdot k_e \cdot k_c \cdot \left(k_t \cdot R_{ACC,0}^{-1} + \varepsilon_t \right) \cdot C_S}{a^2} \cdot t_0^{2 \cdot w} \right)^{\left(\frac{1}{2 \cdot w - 1}\right)}$$

t_{ini}: duration of initiation period due to carbonation [years]

$t_{prop,i}$: duration of propagation period till the treated event i (spalling, cracking, collapse) occurs [years]

t_{SL}: service life [years]

R4.2 Evaluation of $t_{prop,crack}$ and $t_{prop,spall}$

Within a carried out “Delphic Oracle” experts gave experience based estimations about the duration of the of propagation period till the event of cracking and spalling within given exposure conditions. Whereas the start of the estimated propagation period has been defined as the point in time in which the reinforcement becomes depassivated. Furthermore the following conditions had to be considered:

– exposure class according to the definition of EN 206, prEN 1992-1-1

– concrete cover according to prEN 1992-1-1, Table 4.4, structural class 3.

As the average yearly temperature differed on which the estimations have been based the raw data had to be adapted. In the presented case all data has been transformed to a reference temperature of $T_{ref} = 20°C = 293$ K using the Arrhenius-equation in correspondence to [20], cp. Equation R4.2-1.

$$t_{Prop}(T_{ref}) = \frac{t_{Prop}(T_i)}{k_{T_i}} \qquad \text{(R4.2-1)}$$

$$k_{T_i} = \frac{1}{e^{b \cdot \left(\frac{1}{T_{ref}} - \frac{1}{T_i} \right)}}$$

$t_{prop}(T_{ref})$: duration of propagation period till the treated event based on reference temperature T_{ref} [years]

$t_{prop}(T_i)$:	duration of propagation period till the treated event based on temperature T_i [years]
k_{Ti}:	transfer variable to consider the influence of temperature on the duration propagation period [-]
T_{ref}:	reference temperature, here: 293 [K]
T_i:	temperature on which the estimation about the propagation period has been based [K]
b:	regression parameter, here: 4300 [K]

By treating the estimated minimum values of the experts as 1 % quantiles, the mean values as 50 % quantiles and the maximum values as 99 % quantiles the following cumulative frequencies for the estimated propagation periods can be drawn, cp. Figure R4.2-1. The cumulative frequencies in Figure R4.2-1 take the propagation period based on the reference temperature of T = 293 K (20°C) into account.

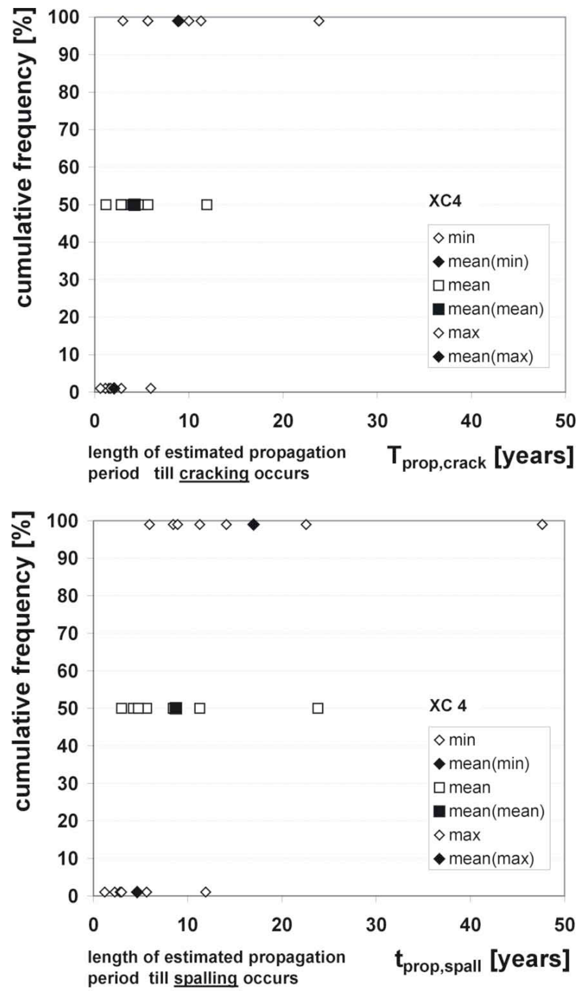

Figure R4.2-1: Cumulative frequencies of the estimated propagation period linked to the event of cracking and spalling

The evaluation based on mean values for each of the treated quantile (cp. Figure R4.2-1) came to the following result for a quantification of $t_{prop,i}$ linked to the events of cracking and spalling, cp. Figure R4.2-2.

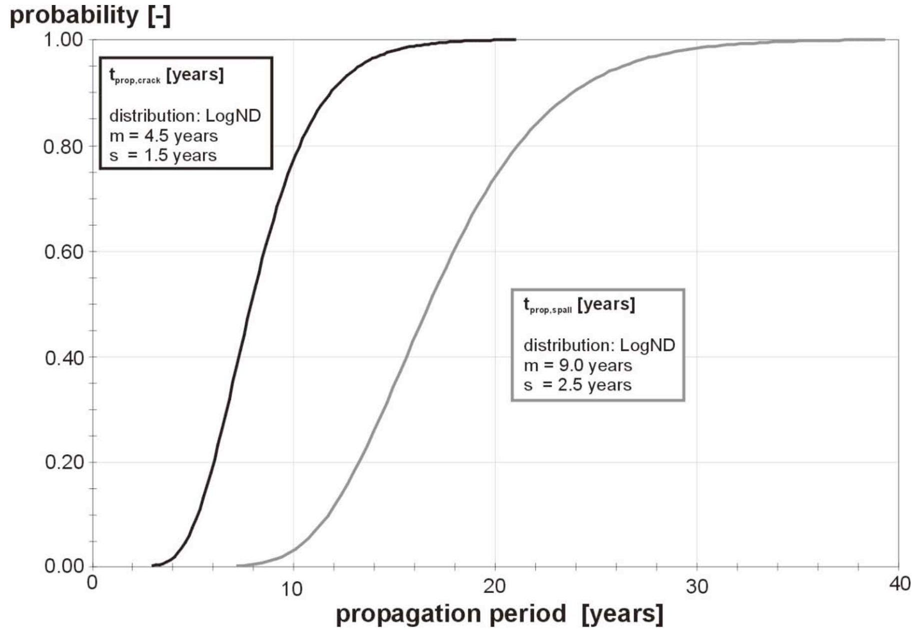

Figure R4.2-2: Cumulative frequency of the evaluated variables $t_{prop,i}$ linked to the event of cracking and spalling (T = 293 K)

R4.3 Evaluation of $t_{prop,collapse}$ (Recalculation of V_{Corr})

Corrosion of reinforcement becomes critical, if required ULS reliability can not be verified at a level, which is required according to Table A2-2 of Annex A. Assumed, that the structural design of reinforced concrete structures is made more or less exactly on a ULS reliability, corrosion of reinforcement will reduce a sufficient ULS-reliability to an insufficient level. In case of no inspection/maintenance/repair extra reinforcement is required which can corrode. This extra cross section has to be derived from the corrosion penetration depth one have to expect during service life.

In order to verify the required reliability for the event of collapse the limit state as given in Equation R4.1-1 has to be evaluated for the event i = collapse.

Following the approach described above the duration of the propagation period linked to the event of collapse has to be determined. The calculation has been carried out by using the following simplified model, cp. Equation R4.3-1.

$$t_{prop,collapse} = \frac{x_{crit,collapse}}{v_{corr}} \tag{R4.3-1}$$

$t_{prop,collapse}$: duration of propagation period till the event of collapse [years]

$x_{crit,collapse}$: penetration depth of the reinforcement linked to corrosion leading to collapse [μm]

v_{corr}: corrosion rate [μm/years]

By using the information of the “Delphic Oracle” (estimated mean value of penetration depth till the treated event and the respective mean value of the duration of the propagation period) an approximate value for the corrosion rate can be derived in a similar way as indicated by Equation R4.3-1. In the considered case the mean value of calculated corrosion rates linked to the event of cracking and spalling has been calculated (38 μm/a) by taking a temperature of T = 293 K (20°C) into account. Furthermore it has been assumed that the corrosion depth is a lognormal distributed variable with a variation of approximately 50 %.

In advance penetration depths $x_{crit,collapse}$ (in magnitude of 25 % loss of cross section, cp. Annex A, Table A3-1, ROC3) have been set as constant parameters in dependency of the reinforcement diameter as 1,000 μm (for diameter 12 mm) or 2,000 μm (for diameter 25 mm). This was made to enable a calculation of corresponding propagation periods ($t_{prop,collapse}$).

A parameter study over the variable $t_{prop,collapse}$ (propagation period till the event of collapse) has been carried out with the with Comrel and Statrel, both belonging to the software package Strurel [12]. The result of this evaluation is outlined in Figure R4.3-1, hereby considering a critical penetration depth of 1,000 μm and 2,000 μm.

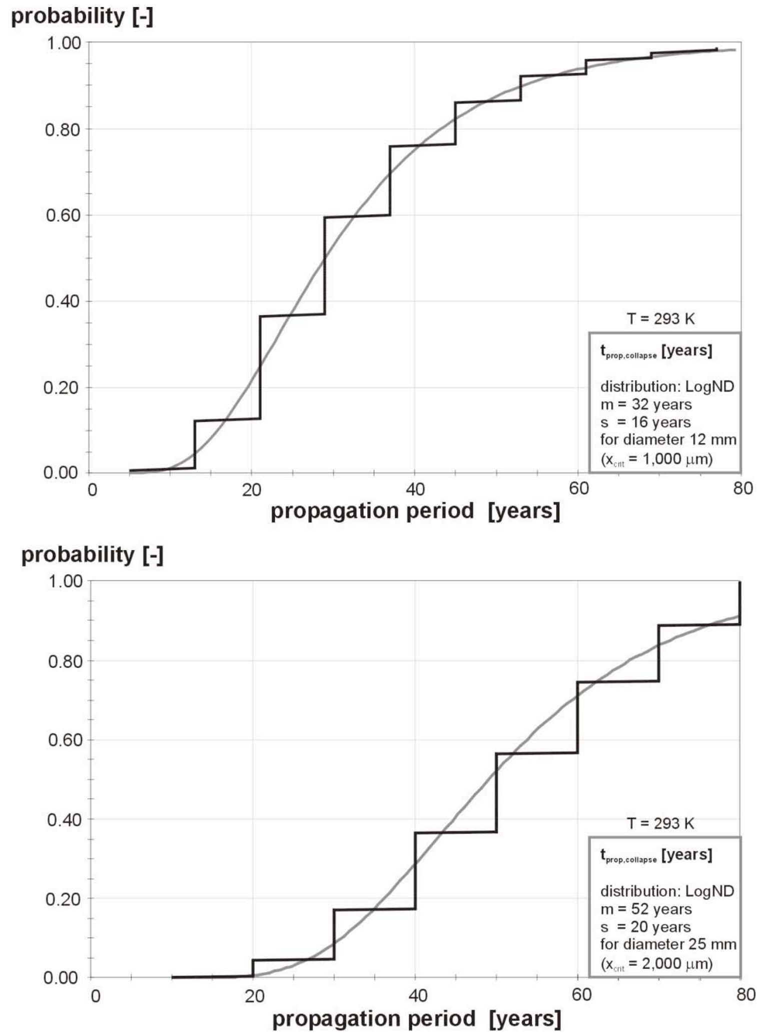

Figure R4.3-1: Parameter study over $t_{prop,collapse}$ (duration of the propagation period till the event of collapse) for a penetration depth of $x_{crit,collapse}$ = 1,000 μm and $x_{crit,collapse}$ = 2,000 μm

R5 Evaluation of Limit States

Calculation no. 1 ("Delphic Oracle", T = 293°K, $\beta_{depassivation}$ = 1.3)

With assumptions for the input data to model the initiation period according to Table R3-1 and information concerning the propagation period derived from the "Delphic Oracle" the following reliabilities after 50 years of exposure (target service life) have been calculated by evaluation of the respective limit state equations and taking a temperature of T = 293 K (20°C) into account, cp. Table R5-1.

Table R5-1: Evaluated reliability indices for the exposition XC4, starting from a reliability index of $\beta_{depassivation} = 1.3$ (reference temperature of T = 293 K)

Limit State	respective time period for the treated event	distr.	m [years]	s [years]	Information derived from	β for t_{SL} = 50 years
depassivation of reinforcement	t_{ini}	LogN	570	1150	Model with quantified input variables according to Table 1	1.3*
cracking of concrete	$t_{prop,crack}$	LogN	4.5	1.5	Delphic Oracle	1.4
spalling of concrete	$t_{prop,spall}$	LogN	9.0	2.5	Delphic Oracle	1.4
collapse of the structural parts	$t_{prop,collapse}$	LogN	32	16	corrosion rate calculated based on information from Delphic Oracle and estimation for $x_{crit,collapse}$	1.8
	$t_{prop,collapse}$		52	20		2.4

*initial reliability index (material variables have been quantified with the aim to reach this reliability index)

According to ISO 2394 a minimum target reliability index for an ultimate limit state, such as collapse, of $\beta_{collapse} = 3.1$ is required, in other standards reliabilities between 3.7 and 4.4, cp. Annex A, Table A2-2 are required. As the corrosion induced reduction of reinforcement is one of various uncertain resistance variables, probably the most dominant resistance variable, the extra depth has to be designed on a reliability level of approx. $\beta_{ULS,corr} = 0.8 \cdot \beta_{collapse}$.

For the beforehand assumed extra depths x_{crit} = 1,000 µm (reinforcement diameter 12 mm) and 2,000 µm (for diameter 25 mm) the calculated reliabilities were 1.8 and 2.4 respectively. The calculated reliabilities of $\beta_{ULS,corr} = 2.4$ (corresponds to $\beta_{collapse} = 3.0$), and 1.8 (corresponds to $\beta_{collapse} = 2.3$) are not sufficient, cp. Table R3-1.

Calculation no. 2 ("Delphic Oracle", T = 293°K, $\beta_{ULS,corr} = 3.1$)

In order to meet the Table A2-2 requirements (RC1: $\beta_{collapse} = 3.7$, RC2: $\beta_{collapse} = 4.2$) a reliability of $\beta_{ULS,corr} = 0.8 \cdot \beta_{collapse}$ of approximately $\beta_{ULS,corr} = 3.1$ have to be confirmed. Due to the calculated reliability index within "calculation no. 1" which were assessed to be not sufficient, further calculations have been carried out aiming to meet the requirement for the event of collapse. This means, for these calculations the material variables (a, $R_{ACC,0}^{-1}$, cp. Table R5-2) have been quantified in such a way, that at the end of service life the calculated reliability index $\beta_{collapse}$ fulfills the requirement.

Table R5-2: Overview of changed material variables compared to Table R3-1 quantified variables (all other variables are unvaried)

Variable		Unit	Distribution	Mean Value	Standard Deviation
4	$R_{ACC,0}^{-1}$	[(m²/s)/(kg/m³)] ([(mm²/years)/(kg/m³)])	normal distribution	$13 \cdot 10^{-11}$ (4,100)	$5 \cdot 10^{-11}$ (1,600)
9	a	[mm]	normal distribution	30	8

The evaluation of limit state based reliability indices shows, that by choosing the material properties is such a way, that at the end of service life a reliability index linked to the event of collapse of $\beta_{ULS,corr} = 3.1$ is reached within a climate with a mean temperature of T = 20°C, the subsequent reliability against depassivation is $\beta_{depassivation} = 2.2$, cp. Figure R6- 1.

Calculation no. 3 ("Delphic Oracle" T = 283°K, $\beta_{depassivation} = 1.3$)

In order to demonstrate the influence of the temperature on the corrosion rate and hence on the calculated reliability indices further calculations have been carried out taking a mean temperature of T = 283 K (10°C) into account, which is a good estimation for the mean temperature for e.g. Germany. The material properties have been chosen in such a way, that the reliability index linked to depassivation of the reinforcement after 50 years of exposure reaches $\beta_{depassivation} = 1.3$, cp. Table R3-1.

The evaluation of limit state based reliability indices show a significant increase of reliability indices compared to the corresponding calculation with a temperature of T = 293 K. By starting with a limit state based reliability for the event of depassivation of $\beta_{depassivation} = 1.3$ and taking the lower mean temperature into account for the event of collapse a reliability index of $\beta_{ULS,corr} = 3.1$ ($x_{crit,collapse} = 2{,}000$ μm) has been calculated, cp. Figure R6- 1

Calculation no. 4 (DuraCrete, T = 293°K, $\beta_{depassivation} = 1.3$)

For orientation purposes additional calculation have been carried out based on corrosion rates as given within [6]. These exposure based data has been collected in southern Europe within in insitu conditions (mean temperature of approximately 20°C).

The time period linked to the event of collapse has been modelled according to Equation R4.3-1, whereat the same estimation for the critical penetration depth as in the other calculation examples has been used ($x_{crit,collapse} = 2{,}000$ μm).

By using this information the respective reliability index linked to the event of collapse can be evaluated according to. Equation R4.1-1, hereby taking the material and environmental variables from Table R3-1 into account to describe the initiation period. By starting with a limit state based reliability for the event of depassivation of $\beta_{depassivation} = 1.3$ and taking the exposure based corrosion rate according [3] into account the calculated reliability index $\beta_{ULS,corr}$ significantly increased ($\beta_{ULS,corr} = 3.8$), cp. Figure R6- 1.

R6 Conclusions

Started with a given reliability $\beta_{SLS} = 1.3$, the evaluation of the corresponding reliability indices β_{SLS} to β_{ULS} is based on information from the “Delphic Oracle” considering unfavourable conditions (small concrete cover) linked to deterioration of reinforcement caused by corrosion.

The calculated reliabilities for the events cracking and spalling are in the range of 1.3 to 1.8 (cp. Figure R6- 1), depending on the temperature regime, the influence of V_{Corr} is from minor importance.

The calculated reliabilities of β_{ULS} are in the range of 2.2 and 3.8 (cp.

Figure R6- 1), temperature and corrosion rate V_{Corr} are from high importance.

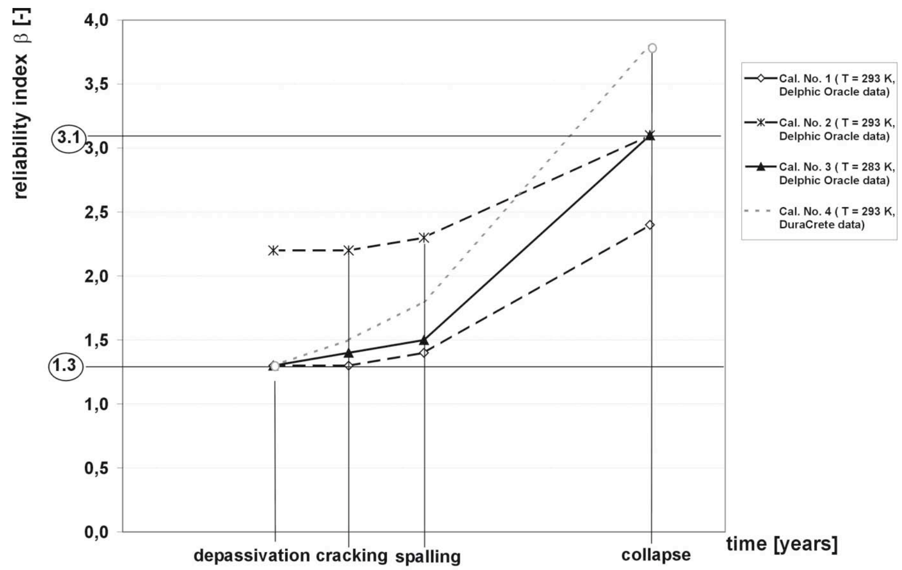

Figure R6- 1: Evaluated reliability indices at the end of service life linked to the limit states of depassivation of reinforcement, cracking and spalling of concrete cover and collapse of the structure. 4 calculations have been carried out considering different boundary conditions ($x_{crit,collapse}$ has been considered as 2,000 μm)

That means, a structure of robustness class 3 (ROC3) exposed to carbonation and middle European average temperatures may also be assessed as sufficient durable with regard to ULS-events if minimum requirements with regard to SLS (depassivation) are fulfilled. As soon as the structure is classified to ROC2 or ROC1 and/or the structure is exposed to higher temperatures (e.g. south Europe), higher reliabilities with regard to the event depassivation should be required.

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