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Durability — Service life design of concrete structures

Durabilité — Conception de la durée de vie des structures en béton

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Contents

Page

Foreword

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The main task of technical committees is to prepare International Standards. Draft International Standards adopted by the technical committees are circulated to the member bodies for voting. Publication as an International Standard requires approval by at least 75 % of the member bodies casting a vote.

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. ISO shall not be held responsible for identifying any or all such patent rights.

ISO 16204 was prepared by Technical Committee ISO/TC 71, *Concrete, reinforced concrete and pre-stressed concrete*, Subcommittee SC 3, *Concrete production and execution of concrete structures*.

Introduction

This International Standard is based on the principles given in ISO 2394, *General principles on reliability for structures*, ISO 13823, *General principles on the design of structures for durability*, and fib[1] “Model Code for Service Life Design” [1] (MC SLD, today implemented in fib Model Code 2010 [2]). The two International Standards were prepared by ISO/TC 98, *Bases for design of structures*.

The limit-states method, as developed in ISO 2394, has been adopted and used for preparing and harmonizing national and regional standards for structural design around the world. The objective of ISO 13823 is to provide a framework for the development of standards to predict the service life of components of a structure and to ensure that these principles are incorporated in the material-specific standards developed by other ISO Technical Committees.

The objective of fib MC SLD is to implement the principles of ISO 2394 in service life design of concrete structures.

This International Standard treats design for environmental actions leading to deterioration of concrete and embedded steel.

The flowchart in Figure 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; in the first, three levels of sophistication are distinguished. In total, four options are available.

Strategy 1: Design to resist deterioration

Level 1 Full probabilistic method (option 1)

Level 2 Partial factor method (option 2)

Level 3 Deemed-to-satisfy method (option 3)

Strategy 2: Avoidance-of-deterioration method, (option 4)

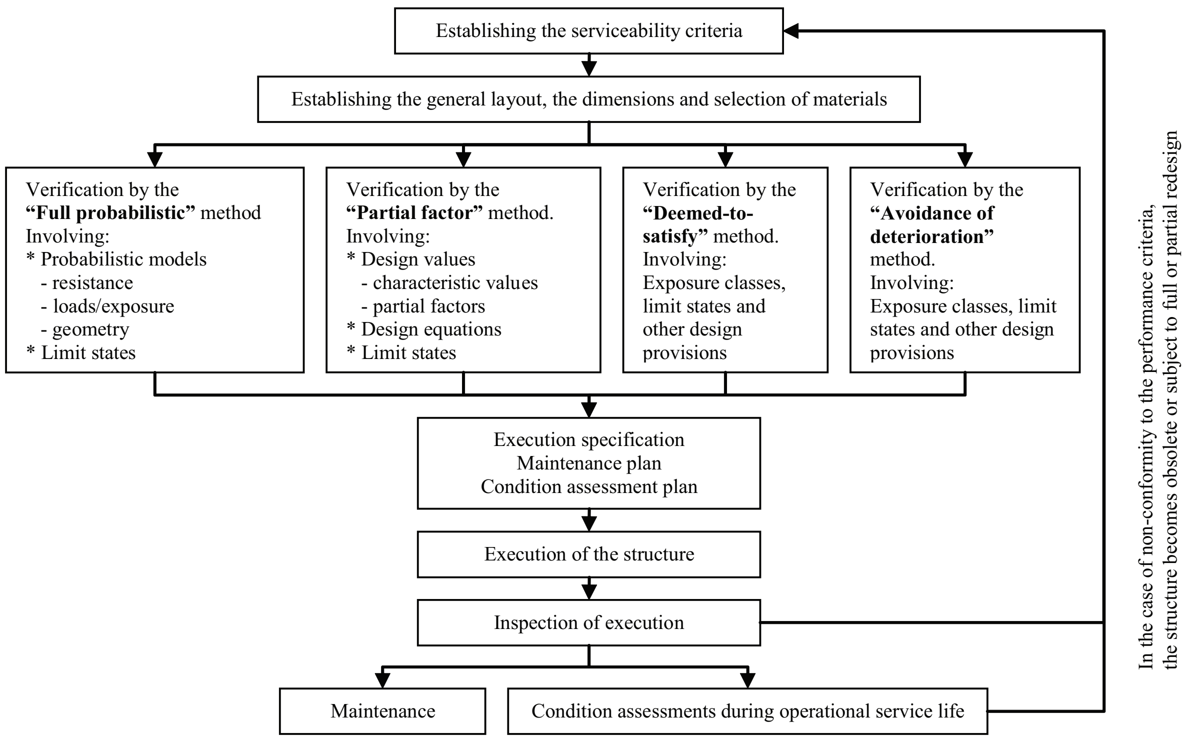

Figure 1 — Flowchart for service life design

1) The International Federation for Structural Concrete.

Within Clause 6 the following deterioration mechanisms are addressed:

— carbonation-induced corrosion;

— chloride-induced corrosion;

— freeze/thaw attack without de-icing agents or sea-water;

— freeze/thaw attack with de-icing agents or sea-water.

For these mechanisms widely accepted mathematical models exist.

The other deterioration mechanisms:

— chemical attack, and

— alkali-aggregate reactions,

are not treated in detail primarily because widely accepted mathematical models do not exist at present.

To make this International Standard complete, the missing models have to be developed and comply with the general principles of Clause 5.

This International Standard includes four informative annexes giving background information for the application in service life design and one informative annex giving guidance for the preparation of a possible national annex.

Durability — Service life design of concrete structures

1 Scope

This International Standard specifies principles and recommends procedures for the verification of the durability of concrete structures subject to:

— known or foreseeable environmental actions causing material deterioration ultimately leading to failure of performance;

— material deterioration without aggressiveness from the external environment of the structure, termed self-ageing.

NOTE The inclusion of, for example, chlorides in the concrete mix might cause deterioration over time without the ingress of additional chlorides from the environment.

This International Standard is intended for use by national standardization bodies when establishing or validating their requirements for durability of concrete structures. It may also be applied:

— for the assessment of remaining service life of existing structures; and

— for the design of service life of new structures provided quantified parameters on levels of reliability and design parameters are given in a national annex to this International Standard.

Fatigue failure due to cyclic stress is not within the scope of this International Standard.

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 2394, *General principles on reliability for structures*

ISO 13823, *General principles on the design of structures for durability*

ISO 22965-1, *Concrete — Part 1: Methods of specifying and guidance for the specifier*

ISO 22965-2, *Concrete — Part 2: Specification of constituent materials, production of concrete and compliance of concrete*

ISO 22966, *Execution of concrete structures*

ISO 6935 (all parts), *Steel for the reinforcement of concrete*

ISO 16311 (all parts), *Maintenance and repair of concrete structures*[2)]

2) To be published. ISO 16311-1, -2, -3 and -4 are under preparation.

3 Terms and definitions

For the purposes of this document, the following terms and definitions apply.

3.1
basic variable
part of a specified set of variables representing physical quantities, which characterize actions and environmental influences, material properties including soil properties, and geometrical quantities

[ISO 2394:1998, 2.2.18]

3.2
characteristic value
X_k **or** R_k
value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series

NOTE 1 This value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product.

NOTE 2 A nominal value is used as the characteristic value in some circumstances.

3.3
characteristic value of a geometrical property
a_k
value usually corresponding to the dimensions specified in the design

NOTE Where relevant, values of geometrical quantities may correspond to some prescribed fractiles of the statistical distribution.

3.4
characteristic value of an action
F_k
principal representative value

NOTE It is chosen:

— on a statistical basis, so that it can be considered to have a specified probability for not being exceeded towards unfavourable values during a reference period;

— on acquired experience; or

— on physical restraints.

[ISO 2394:1998, 2.3.12]

3.5
design criteria
quantitative formulations that describe for each limit state the conditions to be fulfilled

3.6
design service life
assumed period for which a structure or a part of it is to be used for its intended purpose with anticipated maintenance, but without major repair being necessary

3.7
design situation
set of physical conditions representing a certain time interval for which the design demonstrates that the relevant limit states are not exceeded

[ISO 2394:1998, 2.2.1]

3.8
design value of a geometrical property
a_d
generally a nominal value

NOTE 1 Where relevant, values of geometrical quantities may correspond to some prescribed fractile of the statistical distribution.

NOTE 2 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.

3.9
design value of an action
F_d
value obtained by multiplying the representative value by the partial factor γ_f or γ_F.

[Modified from ISO 2394:1998, 2.3.16]

3.10
design value of material or product property
X_d **or** R_d
value obtained by dividing the characteristic value by a partial factor γ_m or γ_M, or, in special circumstances, by direct determination

NOTE See 5.4.2 (3).

[Modified from ISO 2394:1998, 2.4.3]

3.11
execution specification
documents covering all drawings, technical data and requirements necessary for the execution of a particular project

NOTE The execution specification is not one single document but signifies the total sum of documents required for the execution of the work as provided by the designer to the constructor and includes the project specification prepared to supplement and qualify the requirements of this International Standard, as well as referring to the national provisions relevant in the place of use.

[ISO 22966:2009, 3.8]

3.12
inspection
conformity evaluation by observation and judgement accompanied as appropriate by measurement, testing or gauging

[ISO 9000:2005, 3.8.2]

3.13
limit state
state beyond which the structure no longer satisfies the relevant design criteria

NOTE Limit states separate desired states (no failure) from undesired states (failure).

[Modified from ISO 2394:1998, 2.2.9]

3.14
maintenance
set of activities that are planned to take place during the service life of a structure in order to fulfil the requirements for reliability

3.15
project specification
project-specific document describing the requirements applicable for the particular project

[ISO 22966:2009, 3.15]

3.16
reference period
chosen period of time which is used as a basis for assessing values of variable actions, time-dependent material properties, etc.

[ISO 2394:1998, 2.2.8]

3.17
reliability
ability of a structure or a structural member to fulfil the specified requirements, including the design service life, for which it has been designed

NOTE 1 Reliability is usually expressed in probabilistic terms.

NOTE 2 Reliability covers safety, serviceability and durability of a structure.

[Modified from ISO 2394:1998, 2.2.7]

3.18
reliability differentiation
measures intended for socio-economic optimization 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

3.19
repair
activities performed to preserve or to restore the function of a structure that fall outside the definition of maintenance

3.20
representative value of an action
F_{rep}
value used for the verification of a limit state

NOTE Representative values consist of characteristic values, combination values, frequent values and quasi-permanent values, but may also consist of other values.

[ISO 2394:1998, 2.3.11]

3.21
resistance
capacity of a member or component, or a cross-section of a member or component of a structure, to withstand actions that lead to deterioration

3.22
serviceability limit state
state that corresponds to conditions beyond which specified service requirements for a structure or structural element are no longer met

[ISO 2394:1998, 2.2.11]

3.23
serviceability criterion
design criterion for a serviceability limit state

3.24
ultimate limit state
state 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 element, but in some cases to the maximum applicable strain or deformation.

[ISO 2394:1998, 2.2.10]

4 Symbols and abbreviated terms

4.1 Abbreviated terms

SLD	service life design
SLS	serviceability limit state
ULS	ultimate limit state

4.2 Main letters

F	action in general
R	resistance
S	action effect
T	temperature
X	basic variable
a	geometric quantity
p	probability
t	time
x	distance
α	ageing factor
Δ	margin
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 actions, also accounting for model uncertainties and dimensional variations
γ_m	partial factors for a material property, taking account only of uncertainties in the material property
γ_M	partial factors for a material property, taking account of uncertainties in the material property itself and in the design model used
γ_{Rd}	partial factor associated with the uncertainty of the resistance model, plus geometric deviations if these are not modelled explicitly

4.3 Subscripts

app	apparent
crit	critical
d	design value
dep	depassivation
ini	initiation
k	characteristic value
prop	propagation
nom	nominal value
rep	representative value
SL	service life

5 Basis of design

5.1 Requirements

5.1.1 Basic requirements

The service life design (SLD) of concrete structures shall be in accordance with the general principles given in ISO 2394 and ISO 13823.

The supplementary provisions for concrete structures given in this International Standard shall also be applied.

The service life design shall either:

— follow the general principles for probabilistic service life design of concrete structures outlined in ISO 2394 (the full probabilistic method);

— use the partial factor method given in this International Standard;

— use the deemed-to-satisfy method given in this International Standard;

— be based on the avoidance-of-deterioration method given in this International Standard.

The serviceability criteria related to durability shall be specified for each project and agreed with the client.

NOTE Guidance for the choice of serviceability criteria combined with appropriate target values of reliability are given in Annex E of ISO 2394:1998[3], Annex A of fib MC SLD[1] and in JCSS Probabilistic Model Code[6].

5.1.2 Reliability management

Reliability management shall be in accordance with the general principles given in ISO 2394.

NOTE In ISO 2394:1998[3], these provisions are given in Clause 4.

As guidance to reliability differentiation, this International Standard refers to the following general classifications:

— consequence classes CC1, CC2 and CC3;

— reliability classes RC1, RC2 and RC3.

The three consequence classes relate to minor, moderate and large consequences of failure or inadequate serviceability of the structure.

The three reliability classes may be associated with the three consequence classes.

NOTE 1 The three-level differentiation corresponds to that in ISO 2394:1998[3], 4.2

NOTE 2 ISO 13823:2008[7], 8.5 and 8.6 apply a four-level differentiation for consequences of failure.

The required level of reliability may be achieved by measures related to, for example, the robustness of the design and to measures related to quality assurance adopted in the design, execution as well as inspection/maintenance during a structure's service life.

NOTE ISO 22966 defines three execution classes, EXC1, EXC2 and EXC3, for the quality management regime, for which the required strictness increases from class 1 to class 3.

In addition, for service life design, Annex D classifies four levels of condition assessment during the service life:

CAL0, CAL1, CAL2 and CAL3

5.1.3 Design service life, durability and quality management

The design of service life, durability and quality management shall be in accordance with the general principles given in ISO 2394.

NOTE In ISO 2394:1998[3], these provisions are given in Clause 4.

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 states,

— a number of years, and

— a level of reliability for not passing each relevant limit state during this period.

Durability of a structure exposed to its environment shall be such that it remains fit for use during its design service life. This requirement may be satisfied in one, or a combination, of the following ways:

— by designing protective and mitigating systems;

— by using materials that, if well maintained, will not degenerate during the design service life;

— by providing such dimensions that deterioration during the design service life is compensated for;

— by choosing a shorter lifetime for structural elements that when necessary are replaced one or more times during the design life;

— in combination with appropriate inspection at fixed or condition-dependent intervals and appropriate maintenance activities.

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

5.2 Principles of limit state design

The limit state design shall be in accordance with the general principles given in ISO 2394.

NOTE In ISO 2394:1998[3], these provisions are given in Clause 5.

5.3 Basic variables

5.3.1 Actions and environmental influences

Characteristic values of actions for use in SLD shall be

— based on data derived for the particular project, or

— from general field-experience, or

— from relevant literature.

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

Actions specific to SLD may be given in a national annex to this International Standard.

5.3.2 Material and product properties

The material and product properties shall be identified in accordance with the general principles given in ISO 2394.

NOTE In ISO 2394:1998[3], these provisions are given in Clause 6.

Characteristic values of materials and product properties for use in SLD shall be

— based on data derived for the particular project, or

— from general field-experience, or

— 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 applied for mapping the material properties and in the SLD, a checking process shall be established to ensure that the selected model and applied data are compatible.

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 and tested specimens into values that are assumed to represent the behaviour of the material or product in the structure.

5.3.3 Geometric data

Design values of geometrical data for SLD shall be in accordance with ISO 2394 or based on measurements on the completed structure or element.

NOTE In ISO 2394:1998[3], these provisions are given in Clause 6.

ISO 22966 specifies permitted geometrical deviations. If the design assumes stricter tolerances, the design assumptions shall be verified by measurements on the completed structure or element.

5.4 Verification

5.4.1 Verification by the full probabilistic method

The general principles for probabilistic service life design of concrete structures outlined in the ISO 2394 shall be followed.

In particular the following three principles shall be applied:

— 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 taken into account.

5.4.2 Verification by the partial factor method

SLD may be performed in accordance with the general principles of the partial factor method given in ISO 2394.

NOTE In ISO 2394:1998[3], these provisions are given in Clause 9.

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

The partial factor method separates the treatment of uncertainties and variabilities originating from various causes. In the verification procedure defined in this International Standard, the design values of the fundamental basic variables are expressed as follows.

The design values of an action are generally expressed as:

$$F_{\mathrm{d}} = \gamma_{\mathrm{f}} \times F_{\mathrm{rep}} \tag{1}$$

where

F_{rep}	is the representative value of the action;
γ_{f}	is the partial safety factor for the action.

The design values of material or product property is generally expressed as:

$$R_{\mathrm{d}} = R_{\mathrm{k}} / \gamma_{\mathrm{m}} \tag{2}$$

or, where uncertainty in the resistance model is taken into account, by:

$$R_{\mathrm{d}} = R_{\mathrm{k}}/\gamma_{\mathrm{M}} = R_{\mathrm{k}}/(\gamma_{\mathrm{m}} \times \gamma_{\mathrm{Rd}}) \tag{3}$$

where

R_{k}	is the characteristic value of resistance;
γ_{m}	is the partial factor for a 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;
$\gamma_{\mathrm{M}} = \gamma_{\mathrm{m}} \times \gamma_{\mathrm{Rd}}$	is the partial factor for a material property also accounting for the model uncertainties and dimensional variations.

Design values of geometrical quantities to be considered as fundamental basic variables shall, in general, be directly expressed by their design values a_{d}.

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 one of the following two ways:

— on the basis of statistical evaluation of experimental data and field observations according to 5.4.1;

— on the basis of calibration to a long-term experience of building tradition.

5.4.3 Verification by the deemed-to-satisfy method

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.

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 5.4.1;

— 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.

5.4.4 Verification by the avoidance-of-deterioration method

The avoidance-of-deterioration method implies that the deterioration process will not take place due to, for example:

— 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.

The specific requirements for design, materials selection and execution for the avoidance-of-deterioration method may, 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.

6 Verification of service life design

6.1 Carbonation-induced corrosion - uncracked concrete

6.1.1 Full probabilistic method

6.1.1.1 Limit state: Depassivation

The following requirement shall be fulfilled:

$$p\{\} = p_{\text{dep.}} = p\{a - x_{\text{c}}(t_{\text{SL}}) < 0\} < p_0 \tag{4}$$

where

$p\{\}$ is the probability that depassivation occurs;

a is the concrete cover [mm];

$x_c\ (t_{SL})$ is the carbonation depth at the time t_{SL} [mm];

t_{SL} is the design service life [years];

p_0 is the target failure probability.

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

NOTE The limit state “depassivation” is only relevant for structures with sufficient humidity to support a corrosion process.

6.1.1.2 Design model

The ingress of the carbonation front might be assumed to obey the following equation:

$$x_c\left(t\right) = W \times k \times \sqrt{t} \tag{5}$$

k is a factor reflecting the basic resistance of the chosen concrete mix (like water/cement ratio, cement type, additions) under reference conditions and the influence of the basic environmental conditions (like mean relative humidity and CO_2 concentration) on ingress of carbonation. It also reflects the influence of the execution.

W takes into account the varying meso-climatic conditions for the specific concrete member during the design service life, such as humidity and temperature.

For the design of a new structure, the factors W and k might be derived from literature data or existing structures where the concrete composition, execution and exposure conditions have been similar to those expected for the new structure.

When assessing remaining service life of an existing structure, the product of W and k might be derived directly from measurements on the structure.

Other models may be used, provided that the basic principles formulated in 5.4.1 are fulfilled

6.1.1.3 Limit states: corrosion-induced cracking and spalling

Exemplified with regard to cracking, the following basic limit state function shall be fulfilled:

$$p\{\} = p_{crack} = p\{\Delta r_{(R)} - \Delta r_{(S)}(t_{SL}) < 0\} < p_0 \tag{6}$$

where

$p\{\}$ is the probability that carbonation-induced cracking occurs;

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

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

t_{SL} is the design service life [years];

p_0 is the target failure probability.

An alternative design approach is:

$$p\{\} = p_{crack} = p\{t_{SL} - t_{ini} - t_{prop} > 0\} < p_0 \tag{7}$$

where

$p\{\}$ is the probability that carbonation-induced cracking occurs;

t_{SL} is the design service life [years];

t_{ini} is the initiation period (period till depassivation of the reinforcement occurs) [years];

t_{prop} is the propagation period (period of active corrosion) [years];

p_0 is the target failure probability.

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.

Other methods may be used, provided that the basic principles formulated in 5.4.1 are fulfilled.

At the time of publishing this International Standard, no time-dependent model with general international consensus is available for the propagation phase. The time span from initiation to cracking may be estimated from existing structures where the concrete composition, execution and exposure conditions have been similar to those expected for the structure considered.

6.1.2 Partial factor method

6.1.2.1 Limit state: depassivation

The following limit state function needs to be fulfilled:

$$a_d - x_{c,d}(t_{SL}) \geq 0 \tag{8}$$

where

a_d is the design value of the concrete cover [mm];

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

The design value of the concrete cover a_d is calculated as follows:

$$a_d = a_{nom} - \Delta a \tag{9}$$

where

a_{nom} is the nominal value for the concrete cover [mm];

Δa is the safety margin (permitted deviation) of the concrete cover [mm].

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

$$x_{c,d}(t_{SL}) = x_{c,k}(t_{SL}) \times \gamma_f \tag{10}$$

where

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

γ_f is the partial safety factor of the carbonation depth [-].

Other methods may be used, provided that the basic principles formulated in 5.4.2 are fulfilled.

6.1.3 Deemed-to-satisfy method

Within this approach a trading-off of geometrical (concrete cover to reinforcement) material parameters (indirectly linked to diffusion and binding characteristics) and execution aspects (compaction and curing) is applied.

6.1.4 Avoidance-of-deterioration method

Generally, avoidance is achieved if depassivation cannot take place due to infinite resistance of the concrete to carbonation or zero environmental load or infinite corrosion resistance of the reinforcement.

6.2 Chloride-induced corrosion - uncracked concrete

6.2.1 Full probabilistic method

6.2.1.1 Limit state: depassivation

The following limit state function shall be fulfilled:

$$p\{\} = p_{\mathrm{dep.}} = p\{C_{\mathrm{crit}} - C(a,t_{\mathrm{SL}}) < 0\} < p_0 \tag{11}$$

where

- $p\{\}$ is the probability that depassivation occurs;
- C_{crit} is the critical chloride content [% by mass of binder];
- $C(a,t_{\mathrm{SL}})$ is the chloride content at depth a and time t [% by mass of binder];
- a is the concrete cover [mm];
- t_{SL} is the design service life [years];
- p_0 is the target failure probability.

The variables a, C_{crit} and $C(a,t_{\mathrm{SL}})$ shall be quantified in a full probabilistic approach.

NOTE If the binder content of the actual concrete composition is not known, the critical chloride content may be related to the mass of the concrete.

6.2.1.2 Design model

The ingress of chlorides in a marine environment may be assumed to obey the following equation:

$$C(x,t) = C_{\mathrm{s}} - (C_{\mathrm{s}} - C_{\mathrm{i}}) \times \left[\mathrm{erf}\left(\frac{x}{2 \times \sqrt{D_{\mathrm{app}}(t) \times t}} \right) \right] \tag{12}$$

In this modified Fick’s second law of diffusion, the factors are as follows:

- $C(x,t)$ is the content of chlorides in the concrete at a depth x (structure surface: $x = 0$ mm) and at time t [% by mass of binder];
- C_{s} is the chloride content at the concrete surface [% by mass of binder];
- C_{i} is the initial chloride content of the concrete [% by mass of binder];
- x is the depth with a corresponding content of chlorides $C(x,t)$ [mm];
- $D_{\mathrm{app}}(t)$ is the apparent coefficient of chloride diffusion through concrete [mm^2/year] at time t (see Formula 13);
- t is the time [years] of exposure;
- erf is the error function.

$$D_{\text{app}}(t) = D_{\text{app}}(t_0)\left(\frac{t_0}{t}\right)^{\alpha} \tag{13}$$

where

$D_{\text{app}}(t_0)$ is the apparent diffusion coefficient measured at a reference time of t_0;

α is the ageing factor giving the decrease over time of the apparent diffusion coefficient. Depending on the type of binder and the micro-environmental conditions, the ageing factor is likely to lie between 0,2 and 0,8.

NOTE The "apparent" diffusion coefficient after a period t of chloride exposure, $D_{\text{app}}(t)$, represents a constant equivalent diffusion coefficient giving a similar chloride profile to the measured one for a structure exposed to the chloride environment over a period t.

The decrease of the apparent diffusion coefficient is due to several reasons:

— continued reactions of the binder;

— influence of reduced capillary suction of water in the surface zone with time;

— degree of saturation of concrete; and

— effect of penetrated chlorides from sea-water or de-icing salts (leading to ion exchange with subsequent poreblocking in the surface layer).

For the design of a new structure, the parameters C_{S}, C_{i}, α and D_{app} (t_0) may be derived from existing structures where the concrete composition, execution and exposure conditions have been similar to those relevant for the new structure.

When assessing remaining service life of an existing structure, the factors, with the possible exception of α, may be derived directly from measurements on the structure.

For both design of new structures, and for the assessment of remaining service life of existing structures, the ageing factor, α, shall be obtained from in-field observations from structures where the concrete composition, execution and exposure conditions have been similar to those for the actual structure. At least observations at two periods of exposure (with a sufficient interval between the observations) are needed for the calculation of the ageing factor.

Other models may also be used, provided that the basic principles formulated in 5.4.1 are fulfilled.

6.2.1.3 Limit states: corrosion-induced cracking and spalling

See 6.1.1.3.

6.2.2 Partial factor method

See 6.1.2.

6.2.3 Deemed-to-satisfy method

See 6.1.3.

6.2.4 Avoidance-of-deterioration method

See 6.1.4, but substitute "chloride penetration" for "carbonation".

6.3 Influence of cracks upon reinforcement corrosion

The minimum structural reliability of a cracked reinforced concrete structure shall be of comparable magnitude to the minimum reliability of a comparable exposed uncracked structure.

A simplified approach is used by most operational standards. This implies that corrosion of the reinforcement is not influenced by crack widths under a certain characteristic value. Depending on the severity of the environment and sensitivity of the structure, this limiting crack width is normally given as a characteristic value (5 % upper fractile) in the range of 0,2 mm to 0,4 mm

Similar to the procedure given in 6.1 and 6.2, unwanted events with regard to serviceability/functionality shall be identified (SLS). In addition, it shall be checked whether ultimate limits are affected by continuously corroding reinforcement within the cracked zone or not.

In harsh exposure conditions (for example exposure classes XD3/XS3 as defined in ISO 22965-1), if functionality or structural integrity is affected, and if inspection and possible intervention cannot be performed, an avoidance-of-deterioration is recommended/needed.

6.4 Risk of depassivation with respect to pre-stressed steel

Relevant application rules given in 6.1, 6.2 and 6.3 to avoid depassivation of pre-stressed steel on a ULS reliability level shall be applied.

Since corrosion of pre-stressed steel can result in a sudden collapse of the structure, and without previous warning by cracking and spalling associated with ordinary reinforcement, a higher level of reliability for not passing the depassivation limit state is normally needed. See 5.1.

6.5 Freeze/thaw attack

6.5.1 Full probabilistic method

6.5.1.1 Limit state: freeze/thaw damage causing local loss of mechanical properties, cracking, scaling and loss in cross-section – without de-icing agents or sea-water

The following limit state function shall be fulfilled:

$$p\{\} = p_{\text{freeze/thaw damage}} = p\{\ T(t < t_{\text{SL}}),\ S_{\text{CR}} - S_{\text{ACT}}(t < t_{\text{SL}}) < 0\} < p_0 \tag{14}$$

where

$p\{\}$	is the probability that freeze/thaw damage occurs;
S_{CR}	is the critical degree of saturation [-];
$S_{\text{ACT}}(t)$	is the actual degree of saturation at the time t [-];
$T(t)$	is the concrete temperature at the time t [°C];
t_{SL}	is the design service life [years];
p_0	is the target failure probability.

The variables S_{CR} and $S_{\text{ACT}}(t)$ shall be quantified in a full probabilistic approach.

Other methods may be used, provided that the basic principles formulated in 5.4.1 are fulfilled.

NOTE To exemplify the design procedure and the quantification of the given quantities, an applicable design method is given in Annex B of fib MC SLD[1]

6.5.1.2 Limit state equation for the salt-freeze/thaw induced surface scaling – with de-icing agents or sea-water

The following limit state function shall be fulfilled:

$$p\{\} = p_{\mathrm{scaling}} = p\{T(t \le t_{\mathrm{SL}}) - T_{\mathrm{R}}(RH, n_{\mathrm{cycles}}, t, Cl^{-}, ...) < 0\} < p_0 \quad (15)$$

where

$p\{\}$	is the probability that scaling occurs;
$T(t)$	is the concrete temperature at the time t [°C];
$T_{\mathrm{R}}(t,\ldots)$	is the critical freezing temperature for scaling to occur at the time t [°C];
t_{SL}	is the design service life [years];
Cl^{-}	is the chloride content [% by mass of binder];
RH	is the relative humidity [%];
n_{cycles}	is the number of freeze/thaw cycles [-];
p_0	is the target failure probability.

The variables T and T_{R} shall be quantified in a full probabilistic approach.

Formula (15) 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.

Other methods may be used, provided that the basic principles formulated in 5.4.1 are fulfilled

NOTE To exemplify the design procedure and the quantification of the above-given quantities, an applicable design method is given in Annex B of fib MC SLD[1].

6.5.1.3 Limit states: freeze/thaw-induced deformation

With regard to load-carrying capacity and deformations, the design shall include the effect of localized changes in mechanical properties due to freeze/thaw damage.

6.5.2 Partial factor method

The following limit state function needs to be fulfilled:

$$S_{\mathrm{CR,d}} - S_{\mathrm{ACT,d}}\ (t < t_{\mathrm{SL}}) \ge 0 \quad (16)$$

where

$S_{\mathrm{CR,d}}$	is the design value of the critical degree of saturation [-];
$S_{\mathrm{ACT,d}}(t < t_{\mathrm{SL}})$	is the design value of the actual degree of saturation at time t [-];
t_{SL}	is the design service life [years].

The design value of the critical degree of saturation shall be calculated as follows:

$$S_{\mathrm{CR,d}} = S_{\mathrm{CR,min}} - \Delta S_{\mathrm{CR}} \quad (17)$$

where

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

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

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

$$S_{ACT,d}(t) = S_{ACT,k}(t) + \Delta S_{ACT} \tag{18}$$

where

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

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

Other methods may be used, provided that the basic principles formulated in 5.4.1 are fulfilled.

NOTE To exemplify the design procedure an applicable design method is given in Annex C of fib MC SLD[1].

6.5.3 Deemed-to-satisfy method

Within this approach one or more of the following remedies are normally required, based on calibration to long-term experience:

— limitations to the porosity of the concrete (traditionally expressed by the water/cement ratio);

— available space for expansion of freezing water (air entrainment);

— selection of type of cement and aggregates.

6.5.4 Avoidance-of-deterioration method

Generally, avoidance is achieved if freeze/thaw deterioration cannot take place due to infinite material resistance or zero environmental load.

NOTE Due to absence of freezing, and/or moisture levels below $S_{CR,}$ any detrimental reaction due to freezing will be avoided for most structures.

6.6 Chemical attack

6.6.1 Acid attack

6.6.1.1 Full probabilistic method

At the time of publishing this International Standard, no time-dependent model with general international consensus is available for this deterioration process. A full probabilistic approach for SLD is therefore not feasible.

6.6.1.2 Partial factor method

See 6.6.1.1

6.6.1.3 Deemed-to-satisfy method

Within this approach one or more of the following remedies are normally required, based on calibration to long-term experience:

— limitations to the porosity of the concrete (traditionally expressed by the water/cement ratio);

— composite cements or the use of additions like silica fume, fly-ash and slag;

— type of aggregates.

6.6.1.4 Avoidance-of-deterioration method

Solutions with pH greater than about 6,5 are considered as not detrimental to the cement paste based on Portland clinker.

In case of severe acid attack, linings or durable coatings should be provided to avoid direct contact of the acid with the concrete surface.

6.6.2 Sulfate attack

When hardened cement paste is attacked by sulfates, two deterioration mechanisms can occur: ettringite/gypsum formation and thaumasite formation.

Ettringite (sulfoaluminate from C_3A) and gypsum formation of calcium causes expansion and can result in disruption of the hardened paste.

In the presence of calcium carbonate and low temperatures (5 °C to 8 °C) thaumasite can be formed, causing disruption of the hardened cement paste without expansion.

6.6.2.1 Full probabilistic method

At the time of publishing this International Standard, no time-dependent model with general international consensus is available for this deterioration process. A full probabilistic approach for SLD is therefore not feasible.

6.6.2.2 Partial factor method

See 6.6.2.1.

6.6.2.3 Deemed-to-satisfy method

Within this approach one or more of the following remedies are normally required based on calibration to long-term experience:

— maximum amount of C_3A in the cement. Cements with C_3A less than 3 % to 5 % and blast furnace slag cements with more than 60 % slag are often regarded as “sulfate resistant”;

— composite cements or the use of additions like silica fume, fly-ash and slag;

— limitations to the porosity of the concrete (traditionally expressed by the water/cement ratio);

— avoidance of limestone aggregates or fillers in case of the risk of thaumasite formation.

6.6.2.4 Avoidance-of-deterioration method

To support a sulfate attack, sulfate ions and C_3A need to be present in the concrete member. If one or both of these conditions are below certain concentrations, avoidance of deleterious reactions will be achieved.

In case of severe sulfate attack, linings or durable coatings should be provided to avoid direct contact of the sulfate with the concrete surface.

6.7 Alkali-aggregate reactions

Alkali-aggregate reactions involve deleterious chemical reactions between the alkalis in the paste and silica or carbonate constituents in the aggregate.

6.7.1 Full probabilistic method

At the time of publishing this International Standard, no time-dependent model with general international consensus is available for this deterioration process. A full probabilistic approach for SLD is therefore not feasible.

6.7.2 Partial factor method

See 6.7.1.

6.7.3 Deemed-to-satisfy method

Within this approach one, or a combination of more, of the following remedies are normally required based on calibrations to long-term experience:

— maximum amount of alkalis originating from cement and other sources. The total alkali is often expressed as Na_2O equivalent = $Na_2O + 0{,}658 \times K_2O$;

— composite cements or the use of additions like silica fume, fly-ash and slag;

— use of lithium nitrite/hydroxide and other chemical additions;

— limitations to the alkali-reactivity of the aggregate.

Because of possible differences in local geology, care should be taken to transfer experience from one region to another.

6.7.4 Avoidance-of-deterioration method

To support an alkali-aggregate reaction, reactive aggregate, sufficient moisture and sufficient alkalis need to be present in the concrete member. If one or more of these conditions are missing, avoidance of reaction will be achieved.

NOTE The use of linings and protective coatings may avoid ingress of alkalis from the external environment.

7 Execution

7.1 General

The SLD according to this International Standard assumes the requirements for execution and its quality management given in ISO 22966 will be met.

7.2 Execution specification

The execution specification shall cover technical data and requirements for a particular project prepared to supplement and qualify the requirements of ISO 22966.

The execution specification shall include all necessary information and technical requirements for execution of the works and agreements made during the execution.

The execution specification shall therefore transpose all the assumptions to materials, execution and condition control made in the specific SLD into specific requirements.

7.3 Formwork

Possible other requirements than those listed in ISO 22966 shall be stated in the execution specification

7.4 Materials

7.4.1 Reinforcement

Requirements for types of reinforcement other than ordinary steel according to ISO 6935 (galvanized, stainless, coated, non-metallic, etc.) shall be stated in the execution specification.

7.4.2 Pre-stressing

Requirements for post-tensioning systems other than those referred to in ISO 22966 (plastic sheaths, non-metallic strands, etc.) shall be stated in the execution specification.

7.4.3 Concrete

The concrete shall be specified according to, and conform to ISO 22965-1 and ISO 22965-2.

The execution specification shall include possible additional requirements related to the specific SLD models applied.

If test methods not referenced in ISO 22965 are to be applied, the sampling, these test methods and the statistical interpretation of their results shall be stated in the execution specification.

7.4.4 Geometry

The term “permitted deviation” in ISO 22966 on geometrical tolerances may be interpreted as the 5 % percentile.

Possible other assumptions on geometrical tolerances applied in the SLD than those given in ISO 22966 shall be stated in the execution specification.

NOTE The requirements on geometrical tolerances given in class 1 in ISO 22966:2009, Clause 10[4] are assumed to have direct relevance to the design assumptions, while those given in ISO 22966:2009, Annex G do not.

7.5 Inspection

A conformity evaluation of the completed work shall be carried out and the results documented.

The execution specification may give requirements for “as-built-documentation” depending on the specifics of the actual SLD.

Such specifics may be the documentation of the achieved direct input parameters applied in the SLD models, such as diffusion coefficients and cover thickness to the reinforcement.

7.6 Action in the event of non-conformity

If the inspection reveals that the original SLD assumptions are not met during the construction, actions as given in Clause 9 shall be taken.

8 Maintenance and condition assessment

8.1 General

The SLD according to this International Standard assumes that the requirements for maintenance and condition assessments given in ISO 16311[3)] will be met.

Annex D gives advice for the extent of inspection/monitoring of the structure during its service life. Here four “Condition Assessment Levels” are described as guidance to the reliability differentiation to be used in design.

8.2 Maintenance

In this International Standard, the term “maintenance” is used for 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.

A maintenance plan shall state type and frequency of the foreseen activities. The maintenance plan shall be based on the design assumptions and be updated according to the result of the inspection of the completed structure.

NOTE 1 ISO 15686-1:2011[8], B.7 gives further guidance on the content of a maintenance plan.

3) To be published. ISO 16311-1, -2, -3 and -4 are under preparation.

NOTE 2 The maintenance plan can comprise activities like general cleaning, drainage, addition of sealants, replacement of components, etc.

8.3 Condition assessment

8.3.1 Condition assessment plan

The condition assessment plan shall state:

— what types of inspection/monitoring are required;

— what components of the structure are to be inspected/monitored;

— frequency of the inspections;

— performance criteria to be met;

— documentation of the results;

— action in the event of non-conformity with the performance criteria (see Clause 9).

8.3.2 Inspection and monitoring during service life

In this International Standard, “inspection” means activities to evaluate conformity to 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 continual basis.

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

9 Action in the event of non-conformity

If the inspection after the completion of the construction, or during its service life, reveals that the original SLD assumptions are not met, one or more of the following actions shall be taken.

— The scope of the performance survey shall be widened to improve the quality and representativeness of the data.

— A partial or full recalculation of the original SLD shall be performed to assess the remaining 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 Clause 5.

— The structure shall be repaired or strengthened to bring its performance back to the design criteria. The repair shall be based on a partial or full recalculation of the original SLD as stated under Clause 5.

— The structure shall be protected to reduce the action. The protection shall be based on a recalculation of the original SLD as stated under Clause 5.

— The structure is permitted to become obsolete earlier than intended.

According to A.2, the serviceability criteria to be applied during the assessment shall be specified for the project after agreement with the owner.

NOTE 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 is usually lower.

Annex A
(informative)

Basis of design

A.1 Guidance for 5.1.1 – Basic requirements

Traditionally national, regional 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 International Standard gives guidance for such calibration

ISO 22965-1 gives an example of how to differentiate the environmental loads with respect to deterioration on the structure by 17 “exposure classes”. The same classification is adopted by the European CEN standards on the design of concrete structures.

This classification is qualitative in nature and is, by the local standardization body, often linked directly to “deemed-to-satisfy” and “avoidance of deterioration” requirements in operational standards.

If more refined service life designs are to be undertaken by the use of deterioration modelling, this classification of the environmental load must be related to quantified parameters, for instance chloride surface concentrations for marine structures.

When publishing this International Standard, such quantified parameters are not available in any operational standard. Information must therefore be found by measurements on existing structures and in the literature, for instance in fib Bulletin no. 34 - MC SLD[1] and Concrete Society Technical Report no. 61[9].

Another concept related to the verification of limit states associated to durability is described in the guidance paper F to the Construction Products Directive[10] of the European Community. This is the use of so-called “torture tests”.

This method implies that the material or component is subject to test conditions without doubt harsher than will be the case in the actual exposure during the design service life. If the material stands the test, it is also accepted that the verification is fulfilled, however, with an unknown margin.

Many freeze/thaw tests for concrete are in this category.

It is not possible to conclude from a failure to withstand such an onerous “torture test” that the material would underperform during real in-field exposure.

At the time of publishing this International Standard, the European Committee for Standardization (CEN) is working on a concept named “Equivalent Durability Procedure – EDP”[11]. This implies that a material composition not dealt with in the operational standard might be compared to one reference with a proven long-term performance.

The comparison is made on the basis of testing. Based on these test results the performance of the candidate material has to be assessed at the end of its design service life and then compared with that of the reference. Such an extrapolation of test results involves the use of modelling.

The EDP assumes that this modelling is done according to the provisions given by the responsible national standardization body.

A.2 Guidance for 5.1.3 – Design Service Life

The design service life of a structure should be agreed with the owner within possible boundaries given in local legislation.

ISO 2394:1998[3], Table 1 gives examples of design service lives.

Class	Notional design working life	Examples
1	1 to 5	Temporary structures
2	25	Replacement structural parts, e.g. gantry girders, bearings
3	50	Buildings and other structures, other than those listed below
4	100 or more	Monumental buildings and other special or important structures. Large bridges

The table should be used with care.

Some buildings, for instance factories, will often have an economical service life corresponding to the installed machinery. On the other hand, structural parts of residential buildings will, by the general society normally have an expected service life of much more than 50 years as indicated in the table.

A.3 Guidance for 5.1.3 – Quality management

Those quality management and control measures in design, detailing, execution and during the structure's service life required in this International Standard aim to eliminate failures due to gross errors, and ensure the resistance assumed in design

Annex B
(informative)

Verification of service life design

B.1 Guidance for 6 – Verification of service life design

This International Standard gives only models for the verification of service life in the cases when only one mechanism is dominating the deterioration of the structure. If physical interaction effects are envisaged (e.g. for cracking and scaling due to corrosion of the reinforcement combined with freeze/thaw actions), more elaborated models must be applied.

B.2 Guidance for 6.1.1.1 – Limit state: Depassivation due to carbonation

When publishing this International Standard, there are no models with broad international consensus available for predicting the length of the corrosion period till cracking, spalling or collapse of the structure occurs. When considering the effect of corrosion of the reinforcement after its depassivation, splitting stresses in the cover zone from the reinforcement due to the effects of other mechanical actions/loads should also be considered. Everywhere that there are bond stresses in the reinforcement there are also “bursting stresses” in the concrete of the same nature as those from the expanding corrosion product, leading up ultimately to formation of the same type of cracking and spalling of the cover.

For these reasons, service life designs are normally done with the limit state of depassivation (reaching a reduced pH of 8–9 at the rebar surface).

The first direct consequence of passing this limit state is normally that possible future protective measures or repair become more expensive. This conservative limit state is therefore normally linked to a corresponding relaxed target reliability level for failing, often in the order of 10^{-1} to 10^{-2}. See also fib MC SLD[1].

If the length of the service life is defined with more relaxed levels of reliability for the limit state of depassivation, attention should be given to the fact that a significant part of the population in this case often will also have passed the limit state of cracking and spalling at the same term.

To support corrosion of the reinforcement a certain level of humidity 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.

B.3 Guidance for 6.1.1.2 – Design model - carbonation

According to Formula (5), the ingress of the carbonation front is proportional to the square root of time ($t^{0,5}$).

For structures exposed to wet conditions, best curve-fitting of the model towards observed ingress of the carbonation front may be obtained by a somewhat lower exponent than 0,5.

fib MC SLD[1], Annex B, gives further information on how the meso-climatic conditions influence the factors governing W, k and the exponent.

Both the uncertainty of the data and the model shall be taken into account in the design.

When deriving the product of W and k from existing structures, the influence of these uncertainties will decrease considerably the older the structure is.

An overview of other models in use, and a database with supporting parameters, is given in Concrete Society Technical Publication no. 61[9].

B.4 Guidance for 6.1.1.3 – Limit state: Corrosion-induced cracking and spalling

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 (for instance chlorides originating from sea-water or de-icing salts). Macro-cell corrosion effects may trigger high corrosion rates in areas with less severe micro-environmental conditions. 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 $\Delta r_{(\mathrm{S}),}$ (t_{SL}) and $\Delta r_{(\mathrm{R})}$ in Formula (6). 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, fib TG 5.6, when preparing fib MC SLD, organized a Delphic oracle. One result of the exposure dependent output of this Delphic oracle is given in fib MC SLD[1], 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 may be performed, see Formula (6).

B.5 Guidance for 6.1.2.1 – Partial factor method – Limit state: Depassivation

The nominal value for the concrete cover is the dimension given to the constructor in the execution specification (i.e. on drawings) and is assumed to represent the mean value of the cover depth.

The safety margin, Δa, is to ensure that the great majority (in operational standards often assumed as the 95 % fractile) of the cover thickness for the reinforcement bars is larger than the minimum cover used as basis for the service life design.

ISO 22966:2009[4] assumes $\Delta a = 10$ mm if no other values are given in the execution specification.

To exemplify the design procedure and the quantification of the given quantities, an applicable design method is given in Annex C of fib MC SLD[1].

B.6 Guidance for 6.1.3 – Deemed-to-satisfy method – Carbonation

For given design service lives, basic requirements with regard to minimum cover to the reinforcement, concrete limiting values, e.g. maximum water/cement ratio, crack width limitation and minimum level of workmanship are given in most operational concrete codes. These sets of requirements should be verified according to Clause 5.

An example of such a verification is given in Maage and Smeplass[12].

B.7 Guidance for 6.2.1.1 – Limit state: Depassivation – Chloride induced corrosion

As with carbonation, there were no available models with broad international consensus available for predicting the length of the corrosion period till cracking, spalling or collapse of the structure occurs when this International Standard was published. For this reason service life designs are normally based on the limit state of depassivation (reaching a critical chloride concentration at the rebar surface). As with carbonation, this rather conservative limit state is then normally linked with a corresponding relaxed target reliability level for failing, often in the order of 10^{-1} to 10^{-2}. See also fib MC SLD[1] and B.2.

B.8 Guidance for 6.2.1.2 – Design model – Chloride ingress

fib MC SLD[1] gives further information on the use of the model given as Formula (12) and the influence of the various conditions discussed above.

Both the uncertainty of the data and the model shall be taken into account in the design.

The “apparent” diffusion coefficient after a period t of chloride exposure, $D_{app}(t)$, represents a constant equivalent diffusion coefficient giving a similar chloride profile as the measured one for a structure exposed to the chloride environment over a period t.

Formula (12) is then solved by first solving Formula (13) for the time of exposure in question and then inserting this apparent diffusion coefficient in Formula (12).

A more detailed background on the procedure is given by Maage et al.[13].

When deriving the factor $D_{app}(t_0)$ from existing structures, the influence of these uncertainties will increase considerably the younger the structure is.

This is also the case when the reference diffusion coefficient is derived from a short-term laboratory test on young specimens. It will then not reflect execution or ageing effects and if used for design will normally result in an uneconomic over-design.

The uncertainty in determining the ageing factor α increases the shorter the span in time represented by the observations.

The uncertainty in using an ageing factor α for design for service life periods far beyond that represented by the observations used to derive α should also be considered.

In the given model, the surface chloride concentration, C_S, for practical reasons is assumed constant over the time of exposure. This is not correct as a build-up period is obviously needed. It is however often assumed that this build-up period is relatively short compared with the normal length of a design service life and that the influence of this simplification is limited on the design result. More sophisticated models where the build-up period of C_S is modelled are also sometimes used.

A realistic C_S for new designs should be based on in-field observations on existing structures with similar material composition, execution and exposure. For assessing remaining service life, C_S may be determined directly on the structure in question.

The critical chloride content, C_{crit}, when the depassivation of the reinforcement occurs, is also a variable depending on factors like steel properties, type of cement and additions, moisture content, imperfections in the contact zone between steel and paste, etc. For practical design, a realistic distribution has to be applied. The fundamental knowledge on this property is still limited, and most numbers reported in the literature are based on in-field observations. Often these observations are grouped in tables with qualitatively described ranges as “negligible risk of corrosion”, “possible risk of corrosion”, etc. When performing a probabilistic service life design, it is imperative to make clear if the actual limit state is linked to occurrence of corrosion (based on the statistical spread of C_{crit}), or to a conservative deterministic value of for example “negligible risk” or “possible risk”.

For structures subject to cyclic exposure of chlorides, in particular where deicing salts are used only during the winter season, the chloride ions in the surface layer are not only transported by diffusion. Other effects like capillary suction, wash-out etc will also play a role. The depth of this influenced surface layer will be reduced the more impermeable the concrete is and the shorter the cycles are. Suggestions for how to handle such situations are given in the fib MC SLD[1] and in the general literature.

An overview including other models, and a database for supporting parameters, are given in Concrete Society Technical Publication no. 61[9].

B.9 Guidance for 6.3 – Influence of cracks upon reinforcement corrosion

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 ingress from the top. For usual design service lives of 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 the case of vertical surfaces and soffits with chloride spray or chloride containing water not leaking through cracks, a high quality of concrete cover and ordinary crack width limitation ensures sufficiently long service life (≥ 50 years) without extra protection.

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

B.10 Guidance for 6.4 – Risk of depassivation with respect to pre-stressed steel

fib Bulletin 33[14] describes multi-barrier systems for the protection of pre-stressing systems.

The Bulletin describes three different "protection levels". To ensure the integrity of the component, more barriers are recommended the harsher the exposure conditions for the structure.

These systems are supposed to satisfy the design criteria with ample margin and may be classified according to Clause 5 as "deemed-to-satisfy" requirements based on the "avoidance-of-deterioration" approach

B.11 Guidance for 6.5.3 – Deemed-to-satisfy method – Freeze/thaw attack

For structures exposed to freezing and with the potential for a higher level of moisture than the critical degree of saturation, a design according to the deemed-to-satisfy method is the normal procedure.

As an example, ISO 22965-1 suggests the following set of requirements on the concrete composition for such structures:

— cement type and maximum water/cement ratio;

— minimum strength and/or minimum cement content;

— freeze/thaw-resisting aggregates;

— air entrainment or, if allowed by national provisions, adequate performance demonstrated by freeze/thaw testing (scaling).

Sets of requirements based on long-term field experience are found in most national, regional and international standards.

Due to the complexity in performing design according to the "full probabilistic method", the calibration of these "deemed-to-satisfy" requirements is normally based on an "avoidance-of-deterioration" approach, often based on so-called "torture tests". See also A.1.

Acceptance criteria associated with such test results are normally based on mass loss by scaling (surface damage) or reduction in dynamic modulus (internal damage).

When the requirements are based on the use of torture tests on young specimens (e.g. based on scaling or internal damage) it is assumed that the freeze/thaw resistance of the concrete is not reduced over time. This assumption is questioned for binders with a high content of blast furnace slag and where the structure also will be subject to carbonation.

Annex C
(informative)

Execution

C.1 Guidance for 7.2 – Execution specification

ISO 22966 refers to product and component standards for concrete, reinforcement, pre-stressing systems, prefabricated elements etc.

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

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

C.2 Guidance for 7.3 – Inspection – Birth certificate

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

C.3 Guidance for 7.6.3 – Concrete

To define a set of minimum requirements for the performance of concrete, a product standard prepared according to the principles given in this International Standard is needed as a reference. ISO 22965 fulfils this role.

If the SLD is based on performance characteristics of the concrete, these may be replaced by requirements limiting the concrete composition either in the design phase based on previous experience, or after initial testing during the construction phase. It should be stated in the concrete specification if the requirements for the concrete are target values or characteristic values.

If the SLD is based on other material characteristics than those dealt with in traditional concrete standards such as ISO 22965 (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 execution specification shall refer to the relevant test methods and the statistical interpretation of the results (for instance characteristic values or target values). The specification should also define whether the performance is required from test specimens or specimens taken from the structure.

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

C.4 Guidance for 7.6.4 – Geometry

The geometrical tolerances given in tolerance class 1 of ISO 22966:2009[4], Clause 10 are considered needed to achieve the normally required level of safety in the design assumptions. These are related to both SLD and the given partial factors for materials used in load bearing design.

The tolerances given in ISO 22966:2009[4], Annex G are considered to have minor structural influence.

Annex D
(informative)

Maintenance and condition assessment

D.1 Guidance for 8.1 and 8.2 – General – maintenance and condition assessment

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. One of the condition assessment levels (CAL) during the service life given in Table D.1 may be applied.

Table D.1 — Condition Assessment Levels (CAL)

Level	Characteristics	Requirements
CAL3	Extended inspection	Systematic inspection and monitoring of relevant parameters for the deterioration process(es) that is (are) critical in the SLD
CAL2	Normal inspection	Regular visual inspection by qualified personnel
CAL1	Normal inspection	No systematic monitoring nor inspection
CAL0	No inspection	No possible inspection, for instance due to lack of access

Guidance is provided in ISO 15686-7:2006[15].

In every-day construction, CAL1 is often the most appropriate level, and its consequences should be taken into account for the reliability management for the SLD.

If extended inspection, CAL3, is applied on the structure during its service life, the owner will get the possibility to apply proactive measures in case the expectations for the service life design are not met. The consequences of possible unacceptable performance are thus reduced. This opens for applying a more liberal consequence class and associated reliability class.

On the other hand, when no inspection is possible during the service life, CAL0, the owner will get no warning before the relevant limit state is passed. For such structures a greater reliability (stricter reliability class) should be applied during the service life design.

D.2 Guidance for 8.3.1 – Condition assessment plan

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 should therefore focus on the evaluation of the design data applied in these deterioration models.

Annex E
(informative)

Guidance on a national annex

It is anticipated that the great majority of new concrete structures also in the future will be designed for service life according to the “deemed-to-satisfy” and the “avoidance-of-deterioration“ method.

This implies that the actual standardization body must make such prequalified methods available, either in a national annex to this International Standard or in other relevant national standards.

These prequalified methods (i.e. limiting values for concrete composition, cover depth for the reinforcement, crack width limitations, etc.) should be verified according to the provisions given in this International Standard.

It is recommended that this verification is based on assessments of remaining service life of concrete structures with comparable exposure and design.

To make this International Standard operational for design of new structures according to the “full probabilistic” and the “partial factor” method, the national annex must provide quantified parameters on level of reliability and other design parameters to supplement the provisions given in this International Standard.

Topics to be covered by a national annex might be one or more from the following list:

— deemed-to-satisfy requirements calibrated according to the requirements of this International Standard;

— avoidance-of-deterioration requirements calibrated according to the requirements of this International Standard;

— how to select input data for the deterioration models when doing service life design on new structures according to the “full probabilistic” and the “partial factor” method;

— partial factors to be used when doing service life design on new structures according to the “partial factor” method;

— specific provisions for reliability management;

— required level of reliability for various limit states;

— specific provisions for quality management.

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