MULTIMODEL SOFTWARE TOOL FOR DURABILITY DESIGN AND ASSESSMENT OF CONCRETE STRUCTURES WITH RESPECT TO NEW MODEL CODE




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Abstract

The paper focuses on durability limit states of concrete structures. Both initiation and propagation periods of reinforcement corrosion are discussed. The information for selection of appropriate degradation models forming a particular limit state function is provided, including the models encompassed in the *fib* Model Code 2010. As many uncertainties are involved the approach is the probabilistic one, enabling the reliability-based prognosis of service life. The developed software tool may serve to facilitate the effective decision making of designers and clients. A numerical example is presented considering the initiation period: a comparison of modelled carbonation depth with in-situ measurements on a cooling tower, analysed by several model variants and showing also the utilization of the Model Code model.

Keywords: Concrete, Carbonation, Initiation period, Probability approach, Service life

1 Time aspect in *fib* Model Code 2010

Draft Model Code – *fib* Bulletins No. 55 and 56 (2010) (marked as MC in the following text) fosters some advanced trends which might be well expressed repeating the words by J. Walvraven (convener of MC):

”… *An important new aspect in Model Code 2010 is the introduction of the aspect time. This means that in future structures should be designed for structural safety and serviceability for a defined number of years. This includes as well that high significance is attributed to durability,*

 ISBN 978-80-87158-29-6

maintenance, inspection and repair, already in the state of design. This life cycle thinking is reflected by the set-up of the document. It is recognized that durability is not only a matter of durability of materials, but also of good conceptual design, in combination with good quality management in the construction stage, a good maintenance and inspection plan and the application of appropriate materials. MC2010 is as well written for dealing with existing structures, especially with regard to their remaining service life. In order to determine the remaining service life of a structure the reliability question is an important issue. Therefore the probabilistic safety format is introduced as an alternative for partial factor design. "

Moreover, from the Section **3.3.2** of MC may e.g. be cited: "*Performance requirements are established by means of the performance criteria and the associated constraints related to **service life** and **reliability**. The performance requirements are satisfied if all relevant performance criteria are met during the service life at the required reliability level. … Constraints related to service life are given by means of a specified (design) service life (relevant for the design of new structures) or a residual service life (relevant for the re-design of existing structures). The specified service life and the residual service life refer to the period in which the required performance shall be achieved … Constraints related to reliability are specified by means of a target reliability level. A target reliability level refers to an acceptable failure probability corresponding to a specified reference period, which is required to assure the performance of a structure or structural component for which it has been designed. The target reliability level … may adequately be expressed in terms of the target reliability index β_t or target probability of failure P_{ft}.*"

What might be a not common view for engineers till present is (3.2.2) „…*Specifying performance requirements and associated constraints of service life and reliability create an initial bridge between the needs of the stakeholders and the design or the assessment*". … *The specified (design) service life and the residual service life are related to the **required service life** as **given by the stakeholders** and to other implications of service criteria agreement e.g. with regard to structural analysis, maintenance and quality management. The required service life should be given by the owner in consideration of the interests of other stakeholders (i.e. users, contractors, society). … The nominal/formal end of the service life is reached when the performance criteria are not anymore met at the required reliability level".*

These approaches deal with *durability* and *reliability* issues, which both rank amongst the most decisive structural performance characteristics. This is reflected by other recent standardization activities, e.g. the ISO 13823 (2008) and it is also in the focus of ***fib* Commission 2** activities (the present paper can be viewed as a part of it).

Section **7.8** of MC specializes in the durability design of concrete structures. The objective is to identify agreed durability related models and to prepare the framework for the standardization of performance-based durability design approaches. This model code deals with designing that takes into account environmental actions leading to the degradation of concrete and embedded steel. Verification of limit states associated to durability may be done according to one of the safety formats given in chapter 4 of MC:

(i) The full probabilistic safety format;
(ii) The partial safety factor format;
(iii) The deemed-to-satisfy approach;
(iv) The avoidance-of-deterioration approach.

The following deterioration mechanisms are treated in MC: carbonation-induced corrosion, chloride-induced corrosion, and, freeze-thaw attack with or without de-icing agents. For carbonation effect and for chloride ion penetration some complex and well-verified analytical models are introduced – one model for each effect; more details are presented in *fib* Bulletin 34.

Traditionally, national and international concrete standards give requirements to achieve the desired design service life based on the (iii) and/or (iv) approach. Such operative requirements have to be calibrated by the responsible standardization body. The clause 7.8 gives guidance for such calibration and the (i) format creates a crucial tool. Note, the approach (i) is the only one which provides the quantitative information about the safety level; also, the values of partial safety coefficients for durability of concrete structures in format (ii) are not codified yet (according to the authors best knowledge) and moreover, they should be verified by format (i). This shows clearly the dominancy of the full probabilistic safety format.

It appears that predictive models are needed to estimate how resistance (and/or loads) will change over time. The utilization of such models is decisive by checking a probability limit condition: considering the Ultimate Limit State (ULS) and/or the Serviceability Limit State (SLS), the general condition for probability of failure P_f may be formulated as:

$$P_f = P(A \geq B) < P_d \tag{1}$$

where A is the action effect, B is the barrier and P_d is the design (acceptable, target) probability value. Reliability index β is alternatively utilized instead of the probability of failure in practice. Generally, both A and B (and hence P_f or β) are time dependent; this has not been considered for common cases of ULS or SLS in design practice very frequently up to now. Broad utilization of such an approach is, unfortunately, still prevented by the insufficient dissemination of basic ideas, relevant knowledge or by the lack of experimental evidence, suitable models and of efficient and user-friendly design instruments (software and other). The durability and reliability issues should be addressed during the design process and discussed with the stakeholders that create the basis for the application of a ***performance-based approach*** (see e.g. Teplý et al., 2010).

2 Durability limit states

In case of durability, the limit state can be expressed by means of service life format as:

$$P_f = P(t_S \leq t_D) \leq P_d \tag{2}$$

where t_D is design life and the service life t_S can be determined as the sum of two service-life predictors:

$$t_S = t_i + t_p \tag{3}$$

where t_i is the time of the initiation of reinforcement corrosion and t_p is the part of service life after corrosion initiation (the propagation period).

When considering the degradation of reinforced concrete structures, the corrosion of reinforcement is the dominating effect. In the context of the initiation period only one limit state can be recognized/defined: depassivation of reinforcement, i.e.

$$t_S = t_i \tag{4}$$

The direct consequence of passing this limit state is only that possible future protective measures for repair become more expensive. This rather conservative limit state is therefore normally linked to a corresponding relaxed target reliability level for failing. Often, such limit states are recognized as Durability Limit States (DLS). Generally, the principal factors causing depassivation of reinforcement in concrete are carbonation and/or chloride ingress. The variables representing A and B in Eq. (1) are in such cases:

(i) ***concrete carbonation***: B is concrete cover and A is the depth of carbonation at time t_D;

(ii) ***chloride ingress***: B is the critical concentration of Cl^- which leads to steel depassivation and A is the concentration of Cl^- at the reinforcement at time t_D.

Considering a progress of **reinforcement corrosion**, i.e. the propagation period t_p, then following situations may be assessed according to Eq. (1): B is the critical tensile stress that initiates a crack in concrete (on an interface with a reinforcing bar) due to corrosion, A is the tensile stress in concrete at time t_D; or B is the critical crack width on the concrete surface and A is the crack width on the concrete surface generated by reinforcement corrosion at time t_D; alternatively B is the reinforcement cross-sectional area at time t_D and A is the minimum acceptable reinforcement cross-sectional area with regard to either the SLS or the ULS.

The relevant values of variables A and B used in Eq. (1), which are random quantities, have to be assessed by utilization of a suitable degradation model or by field or laboratory investigations. For the purposes of the former case effective probabilistic software tools are needed. Note, the

 ISBN 978-80-87158-29-6

broader choice of models is useful in many design situations, e.g. due to problems with the availability of statistical data or the testing method for the input variables of each model, and/or with respect to the authenticity of the model when using the available data in relation to limit state in question and/or to the required design accuracy level.

3 Software tool

There are many predictive computational models for degradation modelling mainly carbonation of concrete and corrosion of reinforcement. They are mainly heuristic using more or less simplified approaches and data. Common feature of all these models is that input data are very uncertain. Authors developed a software implementation where all relatively well-known models are summarized within the framework of unified software environment. It is called *FReET-D* where a combination of analytical models and simulation techniques has been amalgamated to form specialized software for assessing the potential degradation of newly designed as well as existing concrete structures (Teplý et al., 2010; Veselý et al. 2010). Implemented models for carbonation, chloride ingress, corrosion of reinforcement and frost attack may serve directly in the durability assessment of concrete structures in the form of a DLS, i.e. the assessment of service life and the level of the relevant reliability measure. Several features are offered including parametric studies and Bayesian updating. Altogether, 32 models are implemented as pre-defined dynamic-link library functions.

FreET-D actually represents a specialized module of *FReET* software (Novák et al., 2010). The probabilistic software FReET allows simulations of uncertainties of the analyzed problem basically at random variables level (typically in civil/mechanical engineering – material properties, loading, geometrical imperfections). The attention is given to those techniques that are developed for analyses of computationally intensive problems; nonlinear FEM analysis being a typical example. Stratified simulation technique Latin hypercube sampling (LHS) is used in order to keep the number of required simulations at an acceptable level. This technique can be used for both random variables' and random fields' levels. Statistical correlation is efficiently imposed by the stochastic optimization technique – the simulated annealing (Vořechovský and Novák, 2009). Sensitivity analysis is based on nonparametric rank-order correlation coefficients. State-of-the-art probabilistic algorithms are implemented to compute the probabilistic response and reliability generally, including durability limit states.

4 Application example – cooling tower

Utilizing three models encompassed in *FReET-D* the carbonation depth on an RC cooling tower was analyzed: (a) a complex model for carbonation process developed within the European joint research, incorporated into the MC (together 12 input variables; for more details see fib Bulletin (2006)), (b) the model developed by Papadakis et al. (1992), based on mass conservation (13 input variables), and, (c) a more simple model developed by Morinaga (6 input variables; see also Papadakis et al. (1992)). The models (b) and (c) were updated as far as the RH-function is concerned utilizing experimental results and literature sources (details see in User manual of *FReET-D* (Veselý, et al., 2010)).

For the present example the input values listed in **Tab. 1** were used. The tower with a height of 206 m was investigated at the age of 19.1 years and the depth of carbonation was measured (using phenolphthalein tests) at 75 locations on both the internal and external surfaces. At the same time, the range of reinforcement corrosion was visually observed and sorted into three classes (Keršner et al. (1996)). In this way, relatively reliable in situ statistical data were obtained. Also, the strength of concrete in compression gained in situ was established: a mean value of 34 MPa with a coefficient of variation (COV) equal to 28.3 % (external surface) and 14.3 % (internal).

Tab. 1 Input parameters for models

Input parameter	Unit	Mean value	COV	PDF	Model
Time of exposure	years	19.1	-	Deterministic	a, b, c
CO_2 content in the atmosphere	mg/m^3	800	0.12	Normal	a, b, c
Relative humidity: external internal	%	70 93	0.07 0.03	Beta (bounds a = 0, b = 100)	a, b, c
Unit content of cement in concrete	kg/m^3	342	0.03	Normal	b, c
Unit content of water in concrete	kg/m^3	188	0.03	Normal	b, c
Unit content of aggregate (0-4 mm)	kg/m^3	834	0.03	Normal	b
Unit content of aggregate (4-8 mm)	kg/m^3	373	0.03	Normal	b
Unit content of aggregate (8-16 mm)	kg/m^3	614	0.03	Normal	b
Specific gravity of cement in concrete	kg/m^3	3100	0.02	Normal	b
Specific gravity of aggregate (0-4 mm)	kg/m^3	2590	0.02	Normal	b
Specific gravity of aggregate (4-8 mm)	kg/m^3	2540	0.05	Normal	b
Specific gravity of aggregate (8-16 mm)	kg/m^3	2660	0.05	Normal	b
Exponent of regression of execution transfer parameter function k_c	-	-0.567	0.04	Normal	a
Curing period	days	1	-	Deterministic	a
Inverse effective carbonation resistance of dry concrete from ACC-test	$\frac{m^2/s}{kg/m^3}$	9.8×10^{-11}	0.48	Normal	a
Regression parameter of function $R^{-1}_{NAC;0}$	-	1.25	0.28	Normal	a
Error term of function $R^{-1}_{NAC;0}$	$\frac{m^2/s}{kg/m^3}$	1×10^{-11}	0.15	Normal	a
Days with rainfall h_{ND} > 2.5 mm per year	days	27.3	-	Deterministic	a
Probability of driving rain – internal external	-	0.002 0.200	-	Deterministic	a
Exponent of regression of weather function w	-	0.446	0.37	Normal	a
Uncertainty factor of model	-	1	0.15	Lognormal (2 par)	a, b, c
Concrete cover – internal external (in-situ measurements)	mm	23.6 28.4	0.30	Lognormal (2 par)	*

*) employed for reliability index assessment only (model (b), **Fig. 2**)

Tab. 2 presents a comparison of the analytical results with experimental data, namely the mean and COV of carbonation depth for both surfaces. The agreement is rather satisfactory for models (b) and (c). Also note, the modelled mean value for the internal surface is more "exact" then for the external one – most probably due to the almost steady humidity inside the tower in reality; also the uniformity of concrete quality is better.

It appears the model (a) underestimates the carbonation depth. The reason might be the "problematic" assessment of some input data, e.g. the number of rainy days (defined by a minimum amount of precipitation water of 2.5 mm per day) and the probability value of driving rain – both as

 ISBN 978-80-87158-29-6

certain representation for the age of the structure. Also the estimation of curing period is disputable in case of such cooling tower. The availability of the more models seems to be rewarding in some situations, having in mind that more sophisticated computational model requires larger amount of input data, not always available.

Tab. 2 Carbonation depths in a cooling tower: comparison of analytical models with measurements on a real structure at the age of 19.1 years

	External surface (RH = 70%)		**Internal surface (RH = 93%)**	
	Mean [mm]	COV [%]	Mean [mm]	COV [%]
Model a	10.8	48	4.4	60
Model b	12.7	18	8.3	51
Model c	11.9	21	7.7	53
in situ measurement (Keršner et al. 1996)	14.9	56	8	29

The analysis by FReET-D also provides a prognosis for future decades and its correction by Bayes updating while utilizing the real (measured) data for the age of 19.1 years with the consequence of more reliable residual service life prediction. A comparison of the results calculated using the (b) model and Bayes updating is plotted in **Fig. 1** for the internal surface in the range of 0 to 50 years. A considerable decrease in carbonation depth scatter may be observed.

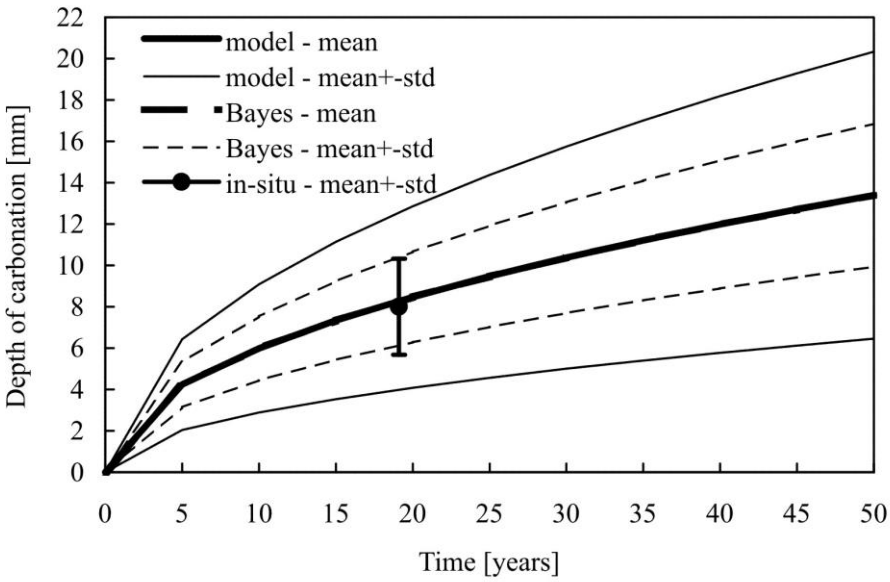

Fig. 1 Comparison of carbonation model results and Bayes updating for the internal surface, model (b)

Additionally, the reliability measure for the initiation period t_i according to Eq. (1) has been assessed, where A was depth of carbonation and B concrete cover (external or internal). Model (b) was used to compute the reliability index values (FORM) at different time steps applying the in-situ measured concrete cover value statistics (**Tab. 1**). **Fig. 2** presents the resulting reliability profile for both surfaces together with the target reliability level of β = 1.5. Defining the service life

t_S conservatively (*fib*-Model Code 2010) according to Eq. (4) it may be observed that t_S would be about 30 years. It should be noted that the used model provides the carbonation level at "one point" of structure only, thus representing the carbonation state of the whole structural element rather generally, i.e. as an average. A more appropriate and economical definition of t_i is based on the reinforcement depassivation reached in a certain proportion of the structure, e.g. 20 % of it. Such a kind of assessment would be much more laborious albeit possible.

In any case, the results gained while utilizing the simple approach shown above and presented in **Fig. 2** correlate relatively well with the on-site findings (Keršner et al. (1996)) where for approximately 90 % of samples "no corrosion" state was observed at 19.1 years of structure age.

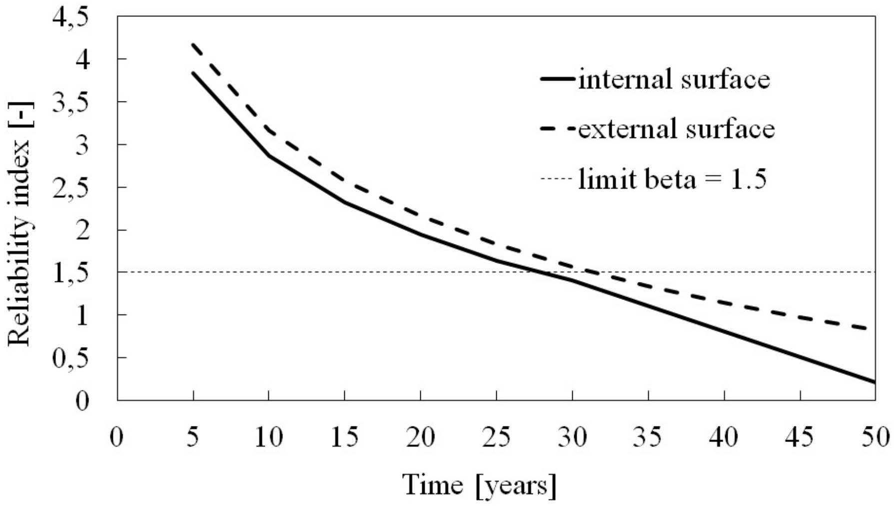

Fig. 2 Reliability profile for both surfaces gained by model (b)

5 Conclusions

Time aspect in fib Model Code 2010 emphasizes the urgent need for durability limit states consideration. Nowadays, there many models exist for degradation modeling, which can be in certain sense confusing. Software implementation can help to choose appropriate models and to assess durability limit states.

On the presented example it is shown that the broader choice of models is useful e.g. due to problems with the availability of statistical data for the input variables of some models. It appears the model recommended in MC underestimates the carbonation depth in the present example. Moreover, the readiness of effective models for both the initiation as well as the propagation period may enable to verify the SLS and/or ULS in different time steps. In this way the assessment of all limit states at different structural situations is "unified" considering the change of performance in time due to degradation (mentioned in 3.3.2.2 of MC).

The usefulness of effective degradation modeling and hence a reliable design/assessment for durability may bring positive economical and sustainability impacts. It might be noted that the level of reliability in the context of durability should be left to the *client's decision* as well as the choice of appropriate serviceability criteria; unfortunately there is a lack of recognition from investors in this respect, in spite of the fact that the mutual impact of these values on the cost of the project may be crucial.

This outcome has been achieved with the financial support of the Ministry of Education of Czech republic, project No. 1M0579, within activities of the CIDEAS research centre. In this undertaking, theoretical results gained in the project the Czech science foundation grant No. P105/10/1156 "Comocos - Complex modelling of concrete structures" were partially exploited.

ISBN 978-80-87158-29-6

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