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LIMIT STATES OF CONCRETE STRUCTURES SUBJECTED TO ENVIRONMENTAL ACTIONS

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Abstract: *In the context of performance requirements, whole life costing and sustainability, the concrete structure durability issue has recently gained considerable attention. The present paper deals with service life assessment utilizing durability limit states specially defined for concrete structures, i.e. focusing on degradation due to environmental effects. Both the initiation and propagation periods of reinforcement corrosion are considered and a comprehensive choice of limit states is provided. The approach is based on degradation modelling together with probabilistic assessment, enabling the evaluation of service life and the relevant reliability level. A suitable software tool is introduced which makes use of a variety of degradation models and is thus able to facilitate the effective decision making of designers and clients.*

Keywords: ***Concrete structures, Limit states, Durability, Reliability, Environmental effects, Modelling.***

1. Introduction

Structures must be designed to have adequate structural resistance, serviceability and durability. This is achieved by quality management measures including (among other things) reliability requirements, which are usually assessed by the use of limit states as stipulated by ISO 2394 (1998) and EN 1990 (2001). In general, a distinction between ultimate limit states (ULS) and serviceability limit states (SLS) should be made together with the associated levels of reliability described by the probability of failure or by the index of reliability. The relevant design situation should be taken into account with consideration also given to time dependent effects related to the specified period, i.e. the specified design working life (or the service life). Inevitably, the mathematical modelling of degradation phenomena and the utilization of probabilistic simulation methods are needed for both newly designed structures (in the design phase) and existing structures (during assessment, redesign and refurbishment). Thus, not only the mechanical load/actions (permanent, variable and accidental actions) but also the environmental influences/actions that could affect durability should be taken into account. Due to such environmental actions structural materials can be degraded and so both the ULS and SLS can be affected.

The formulation and assessment of relevant limit states for environmental actions differs from those commonly considered by engineers while solving mechanical action effects; therefore, “durability” limit states are discussed in the following section in more detail. Moreover, the present paper concentrates on concrete structures where a variety of degradation processes due to environmental actions may be encountered.

2. Concrete structures

2.1. Limit states in the context of durability - DLS

The *level of reliability* in the context of durability should be left to the client’s decision as well as the appropriate serviceability criteria - as indicated e.g. in *fib* Draft Model Code (2010). It should be noted that reliability level, limit state definition, target service life and financial savings are mutually related

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- some elements of whole life costs are service-life dependent. The joint impact of these values, as well as their effect on the cost of the project, can be considerable.

Durability and its reliability implications need to be addressed during the design process; the agreement or decision of the client should be a basic part of that process - which is not yet common. While assessing the reliability level, an appropriate LS has to be defined and assessed. Within this context, limit states associated with durability (for the purpose of this paper they are termed Durability Limit States - DLS) are recognized – also see ISO 13823 (2008) and the *fib* Bulletin (2006). They can fall into either the ULS or the SLS category, according to the type of degradation effect, its extent and location.

Basically, the reliability level is described by the probability of failure P_f, which may be converted into the reliability index β. For SLS (and hence DLS) the values $0.8 \leq \beta \leq 1.8$ are currently being discussed/considered. The "older" codes (still currently utilized) generally do not provide sufficient reliability support in this respect – they do not allow for a design which is subject to a specific (target) service life and/or to a specific reliability level. Note that the serviceability requirements are agreed for each individual project by a relevant authority.

The service life of a building, structure and/or structural member is determined by its design, construction, ageing and maintenance during use. The combined effect of structural performance and ageing should be considered. When assessing the degradation of reinforced concrete structures, the corrosion of reinforcement is the dominant effect. Usually the initiation and propagation periods are considered – Tuutti (1982). The first period is the time from concrete casting to the moment when the reinforcement is no longer passivated, while the latter also includes the period after corrosion initiation. Generally, the DLS condition may be written as:

$$P_f(t_D) = P\left\{B\left(t_D\right) - A\left(t_D\right) \leq 0\right\} \leq P_d \quad , \tag{1}$$

where P_f is the probability of failure, A is the action effect, B is the barrier at time t_D = design service life and P_d is the design (acceptable, target) probability value. P_d refers to an acceptable failure probability required to assure the performance of the structure and corresponding to a specified reference period.

In the following section let us summarize the possible basic actions and barriers usable for the DLS of concrete structures noting that the corrosion of reinforcement is the dominating effect.

2.2 Initiation period - depassivation of reinforcement

Generally, the dominant factors causing depassivation of reinforcement in concrete are carbonation and chloride ingress:

(i) *Carbonation*: the carbonation process is driven by the diffusivity of ambient CO_2 in concrete and the reactivity of CO_2 with concrete. The CO_2 penetrating from the surface decreases pH to a value of 8.3. When the carbonation depth equals the concrete cover, the steel is depassivated and corrosion may start (when oxygen and moisture are present). The rate of carbonation progress from the concrete surface to the reinforcement depends on many parameters, e.g. concrete cover thickness and permeability, the ambient temperature, relative humidity and carbon dioxide content, while the concrete cover permeability itself depends on the concrete mix type and composition, the aggregate gradation and the processing and curing of the concrete mix. All these variables are uncertain in reality and should be represented by random variables (or random fields). In the case of carbonation the B in condition (1) is represented by concrete cover a while A is represented by x_c – the depth of carbonation at time t_D. So, Eq. (1) specializes into

$$P_f(t_D) = P\left\{a - x_c\left(t_D\right) \leq 0\right\} \leq P_d \quad . \tag{2}$$

Analytical models of the effect of this action are based on the diffusion of CO_2 in the concrete pore system and have been discussed elsewhere, e.g. in *fib* Bulletin (2006) and Teplý et al. (2010). Note that the carbonation processes in concretes fabricated from Portland Cement (OPC) and from blended cements should be distinguished from each other; more details about the latter case can be found e.g. in (Chromá et al. 2007), including a discussion of the *k*-value concept.

(ii) *Chloride ingress*: In order to consider chloride ingress (e.g. due to de-icing salts) Eq. (1) may be transformed into the formula:

$$P_f(t_D) = P\left\{C_{cr} - C_a\left(t_D\right) \le 0\right\} \le P_d \ . \qquad (3)$$

So, in this case $B = C_{cr}$ represents the critical concentration of dissolved Cl^- (a prerequisite of steel depassivation) and $A = C_a$, which is the concentration of Cl^- at the reinforcement at time t_D.
Note that the depassivation of reinforcement in concrete can also be caused by other chemical influences, e.g. sulphate attack; a suitable model for the prognosis of such an effect is not yet known (to the present authors´ knowledge).

2.3 Propagation period - corrosion of reinforcement

When reinforcing steel is depassivated then corrosion is initiated in the presence of sufficient oxygen and moisture. Damage to the structure can proceed via one or more of the following processes:

(iii) *Concrete cracking due to corrosion*: $B = \sigma_{cr}$ is a critical tensile stress that initiates a crack in the concrete at the interface with the reinforcing bars; $A = \sigma_t$ is the tensile strength of the concrete. Eq. (1) specializes into

$$P_f(t_D) = P\left\{\sigma_{cr} - \sigma_t\left(t_D\right) \le 0\right\} \le P_d \ . \qquad (4)$$

Also, a consecutive stage may be assessed where $B = w_{cr}$ is the critical crack width on the concrete surface and $A = w_a$ is the current crack width on the concrete surface at time t_D. The condition reads

$$P_f(t_D) = P\left\{w_{cr} - w_a\left(t_D\right) \le 0\right\} \le P_d \ . \qquad (5)$$

(iv) *A decrease in the effective reinforcement cross-section* due to corrosion: $B = A_t$ is the effective reinforcement cross-sectional area at time t_D and $A = A_{min}$ is the minimum acceptable reinforcement cross-sectional area with regard to the relevant SLS or ULS:

$$P_f(t_D) = P\left\{A\left(t_D\right) - A_{\min} \le 0\right\} \le P_d \ . \qquad (6)$$

Note that an alternative formulation to the general Eq. (1) may be used, this being Eq. (7):

$$P_f(t_D) = P\left\{t_{PS}\left(X_i, t\right) \le t_D\right\} \le P_d \quad , \qquad (7)$$

where t_{PS} is a predicted time value modelled as a function of basic variables $\boldsymbol{X}_i$ ($i = 1, 2,..., n$), n is the number of input parameters involved in the model in question, and time is t.

The values and characteristics of the variables presented in Eqs. (2) to (7) have to be gained via utilization of the stochastic approach together with a suitable model. Evidently, some efficient tools are needed – two such codes developed under the supervision of the authors are briefly described in the next section and have been referred to previously, e.g. Teplý et al. (2007), (2010) and Vořechovská et al. (2009).

3. Software

- The ***RC LifeTime*** programme has been specially developed for carbonation process modelling and uses three models – two for concretes made from Portland cement and one for concretes made from blended cements; it is freely accessible at http://rc-lifetime.stm.fce.vutbr.cz and offers two options:

(i) "Service Life Assessment", which provides the evaluation of service life and its statistical characteristics based on Eq. (2). The input data are model variables (optionally deterministic or random). The statistical characteristics of the relevant service life are output data automatically generated for a series of cover values or for a specific concrete cover. Optionally, the reliability index may be an additional input value associated with the given cover value; the corresponding service life then becomes an output value.

(ii) "Concrete Cover Assessment", providing the statistical evaluation of concrete cover. Apart from model variables the specified/target service life can also be inputted. The statistical characteristics of the carbonation depth vs. time are output data automatically calculated for a series of time values. Optionally, the value of the required concrete cover may be an input value and the relevant reliability index β then becomes an output value, again along the time axis.

Thus, structural members may be designed for a given service life or given reliability level during the initiation period due to the carbonation effect while considering the statistical characteristics of concrete mix, concrete cover and environmental conditions.

- The software package **FReET-D** represents a specialized professional code for the assessment of the potential degradation of newly designed as well as existing concrete structures. For more details see www.freet.cz, Teplý et. al. 2012, Novák et al. 2011. Altogether, 35 models for different reinforced concrete degradation types are implemented as pre-defined dynamic-link library functions. Models were selected from the literature; some of them were originally developed as deterministic models and have been converted into probabilistic form for the purposes of the presented software:

(i) models for concrete carbonation – 13 models or variants for the assessment of Eqs. (2) or (7);

(ii) chloride ingress effect – 7 models or variants, Eqs. (3) or (7);

(iii) reinforcement corrosion – 9 models or variants, Eqs. (4) to (7);

(iv) frost attack – 4 models. A condition in the form of Eq. (1) can be constructed;

(v) acid attack – 2 models. A condition in the form of Eq. (1) can be constructed.

The uncertainties associated with parameters involved in deterioration processes are modeled by random variables, and several simulation techniques may be optionally used (Crude Monte Carlo, Latin Hypercube Sampling or FORM). Statistical, sensitivity as well as reliability analyses are provided. The implemented models 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 safety level of the relevant reliability measure. Several features are offered, including automatic parametric studies and Bayesian updating. Statistical correlation of input variables is efficiently imposed by a stochastic optimization technique termed simulated annealing (Vořechovský & Novák, 2009). Sensitivity analysis is based on nonparametric rank-order correlation coefficients. The models included in the new international document *fib* Model Code (2010) are also inserted within model groups (i) and (ii).

The software has already been used in several practical tasks, e.g. the analysis of a cooling tower, bridges, a TV tower, and other structures. Some illustrative examples created while using the above-mentioned software will be presented at the conference.

5. Conclusions

The usefulness of effective degradation modelling and hence reliable design or assessment for durability may bring positive financial and sustainability impacts - the probabilistic approach often results in a safer and more realistic design than the frequently used deemed-to-satisfy approach encompassed in current codes. Moreover, the availability of a variety of models seems to be beneficial in some situations, bearing in mind that a more sophisticated computational model requires a larger amount of input data, which are not always available.

The probabilistic approach provides quantitative information about the safety level of reliability measures and, together with degradation modelling, enables service life prognosis. It is a useful methodology with regard to the fact that the required level of reliability and the target service life should be left to the client's decision.

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