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SERVICE LIFE AND MAINTENANCE MODELLING OF REINFORCED CONCRETE BRIDGE DECKS [1]

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ABSTRACT

Recent research in the area of assessment and maintenance of reinforced concrete bridge decks is presented in this paper. Three definitions of service life are introduced and the difficult problem of assessing the service life is discussed. A stochastic modelling of corrosion and corrosion cracking is introduced and the site dependency of corrosion is stressed. Finally, a recently developed optimal repair strategy for bridges is briefly explained.

1. INTRODUCTION

Reinforced concrete bridge slabs play an important role in any country's infrastructure. Unfortunately, concrete bridge decks deteriorate rather quickly due to environment, traffic, and deicing salts with great economic consequences. It is therefore essential to assess the service life of concrete bridge decks and to develop optimal strategies for inspection and repair (replacement). In many countries, the main reason for the deterioration is corrosion of the reinforcement, but several other deterioration mechanisms exist.

In this paper service life assessment is discussed based on stochastic models, and recent work on deterioration due to reinforcement corrosion is presented.

[1] Journal of Reliability of Structures and Materials, Vol. 1, 2004, pp. 1-10.

2. SERVICE LIFE

The service life $T_{service}$ for a reinforced concrete structure has been the subject of discussion between engineers for several decades and an agreement on how to define service life for reinforced concrete structures has not been reached. Definitions related to the reliability of the structure have been proposed in recent years. Several authors; see e.g. Thoft-Christensen [1]; have defined the service life as the initiation time for corrosion T_{corr} of the reinforcement. Estimating T_{corr} is a very complicated matter, since it depends on a large number of uncertain variables. An approach based on diffusion theory seems to have reached general acceptance among researches in this field.

The service life $T_{service}$ has later been modified so that the time Δt_{crack} from corrosion initiation to corrosion crack initiation in the concrete is included; see Thoft-Christensen [2]. The service life is then defined by $T_{service} = T_{crack} = T_{corr} + \Delta t_{crack}$. A stochastic model for Δt_{crack} may be developed on the basis of existing deterministic theories for crack initiation; see Liu & Weyers [3]. The corrosion-cracking model is restricted to the stresses resulting from the expansion of the corrosion products. Three stages are considered in the model: Free expansion, stress initiation, and crack initiation.

The service life $T_{service}$ may further be modified so that the time $\Delta t_{crack\ width}$ from corrosion crack initiation to formation of a certain (critical) crack width is included; see Thoft-Christensen [4]. The reliability profile (reliability as a function of time) is estimated by first relating the amount of corrosion products to time and then the reliability and the crack width to the amount of corrosion products. By this modelling it is possible to estimate the reliability of a given structure on the basis of measurements of the crack widths on the surface of the concrete structure. It is believed that such a methodology may be useful in rating corroded concrete bridges and therefore become an important tool in optimal repair strategies for reinforced concrete bridges.

The three definitions defined above may be written

$$
\begin{aligned}
T_{service}^{(1)} &= T_{corr} \\
T_{service}^{(2)} &= T_{crack} = T_{corr} + \Delta t_{crack} \\
T_{service}^{(3)} &= T_{crack\ width} = T_{crack} + \Delta t_{crack\ width} = T_{corr} + \Delta t_{crack} + \Delta t_{crack\ width}
\end{aligned}
\tag{1}
$$

and are illustrated in figure 1.

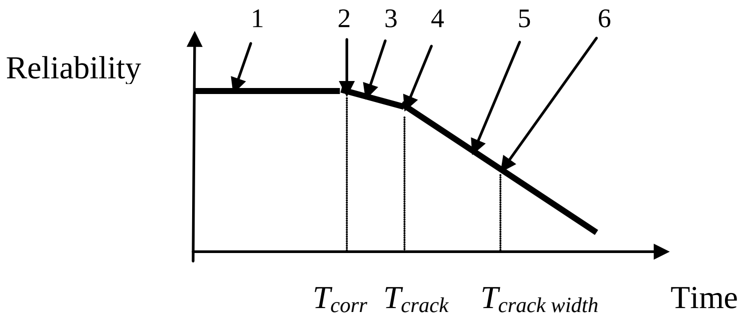

Figure 1. Reliability profile with service life definitions.

The following deterioration steps in the reliability profile are shown in figure 1:

1. Chloride penetration of the concrete.
2. Initiation of the corrosion of the reinforcement.
3. Evolution of corrosion of the reinforcement.
4. Initial cracking of the concrete.
5. Evolution of cracks in the concrete.
6. Critical crack width.

3. ESTIMATION OF THE SERVICE LIFE

Corrosion initiation period refers to the time during which the passivation of steel is destroyed and the reinforcement starts corroding actively. If Fick’s law of diffusion can represent the rate of chloride penetration into concrete, then it can be shown that the time T_{corr} to initiation of reinforcement corrosion is

$$T_{service}^{(1)} = T_{corr} = \frac{d^2}{4D}\left(erf^{-1}(\frac{C_{cr}-C_0}{C_i-C_0}) \right)^{-2} \qquad (2)$$

where d is the concrete cover, D is the diffusion coefficient, C_{cr} is the critical chloride concentration at the site of the corrosion, C_0 is the equilibrium chloride concentration on the concrete surface, C_i is the initial chloride concentration in the concrete, *erf* is the error function.

After corrosion initiation the rust products will initially fill the porous zone around the steel/concrete surface caused by the transition from paste to steel and entrapped/entrained air voids and then result in an expansion or the concrete near the reinforcement. As a result of this, tensile stresses are initiated in the concrete. With increasing corrosion the tensile stresses will reach a critical value and cracks will be developed. During this process the volume of the corrosion products at initial cracking of the concrete W_{crit} will occupy three volumes, namely the porous zone W_{porous}, the expansion of the concrete due to rust pressure W_{expan}, and the space of the corroded steel W_{steel}. With this modelling and some minor simplifications it can then be shown that the time from corrosion imitation to crack initiation is; see Liu & Weyers [3].

$$\Delta t_{crack} = \frac{1}{2\times 0.383\times 10^{-3} D_{bar} i_{corr}}\left(\frac{\rho_{steel}}{\rho_{steel}-0.57\rho_{ust}}\left(W_{porous}+W_{\exp an}\right)\right)^2 \qquad (3)$$

where D_{bar} is the diameter of the reinforcement bar, i_{corr} is the annual mean corrosion rate, ρ_{steel} is the density of the steel, and ρ_{rust} is the density of the rust products. It is in the derivation of (3) assumed that the diameter $D_{bar}(t)$ of the reinforcement bar at the time t is modelled by

$$D_{bar}\left(t\right) = D_{bar}(T_{corr}) - c_{corr} i_{corr}(t-T_{corr}) \qquad (4)$$

where c_{corr} is a corrosion coefficient.

After formation of the initial crack the rebar cross-section is further reduced due to the continued corrosion, and the width of the crack is increased. Experiments (see e.g. Andrade, Alonso & Molina [5]) show that the function between the reduction of the rebar diameter ΔD_{bar} and the corresponding increase in crack width Δw_{crack} in a given time interval Δt measured on the surface of the concrete specimen can be approximated by a linear function

$$\Delta w_{crack} = \gamma \, \Delta D_{bar} \tag{5}$$

where the factor γ is of the order 1.5 to 5. This linearization has been confirmed by FEM analyses; see Thoft-Christensen [6]. It follows from (4) and (5) that

$$w_{crack}(t) = w_{crack}(T_{crack}) + \gamma \left(D_{bar}(T_{crack}) - D_{bar}(t) \right) = w_{crack}(T_{crack}) + \gamma c_{corr} i_{corr} \left(t - T_{rack} \right) \tag{6}$$

Let the critical crack width be $w_{critical}$ corresponding to the service life $T^{(3)}_{service}$. By setting $w(T^{(3)}_{service}) = w_{critical}$ the following expression is obtained for $T^{(3)}_{service}$

$$T^{(3)}_{service} = \frac{w_{critical} - w_{crack}(T_{crack})}{\gamma c_{corr} i_{corr}} + T_{crack} \tag{7}$$

$w_{crack}(T_{crack}) \approx 0$ is the initial crack width at time T_{crack}.

It follows from (2), (3), and (7) that the three life cycle definitions defined earlier may be estimated by

$$\begin{aligned}
T^{(1)}_{service} &= \frac{d^2}{4D} (erf^{-1}(\frac{C_{cr} - C_0}{C_i - C_0}))^{-2} \\
T^{(2)}_{service} &= T^{(1)}_{service} + \frac{1}{2 \times 0.383 \times 10^{-3} D_{bar} i_{corr}} \left(\frac{\rho_{steel}}{\rho_{steel} - 0.57 \rho_{ust}} \left(W_{porous} + W_{\exp an} \right) \right)^2 \\
T^{(3)}_{service} &= T^{(2)}_{service} + \frac{w_{critical} - w_{crack}(T_{crack})}{\gamma c_{corr} i_{corr}}
\end{aligned} \tag{8}$$

Using Monte Carlo simulation, the distribution functions of $T^{(1)}_{service}$, $T^{(2)}_{service}$, and $T^{(3)}_{service}$ can then for given structure be estimated for any value of the critical crack width when stochastic distributions are known for all parameters.

4. THE DIFFUSION COEFFICIENT

The diffusion coefficient D is an important factor in the assessment of reinforced concrete decks. It follows from (2) that the time to corrosion imitation is inversely proportional in D. The diffusion coefficient D for concrete depends on a number of factors. Jensen [7] and Jensen et al. [8] have concluded based on extensive experimental investigations that the most important factors are the water/cement ratio *w/c*, the temperature Φ, and the amount of e.g. silica fume. The data presented in this section are all based on Jensen [7] and Jensen et al. [8]. The diffusion coefficient D increases significantly with *w/c* as well as the temperature Φ, see figure 2. The influence of *w/c* and the temperature Φ may be explained by the chloride binding. Only the free chloride is important for the diffusion coefficient D. With increased *w/c* ratio less chloride is bound and D is therefore increased. The strong influence of the temperature is mainly caused by thermal activation of the diffusion process, but may also be due to a reduced chloride binding when the temperature is increased.

The data presented in figure 2 are detailed discussed in Thoft-Christensen [10]. The curves in figure 2 are obtained by regression.

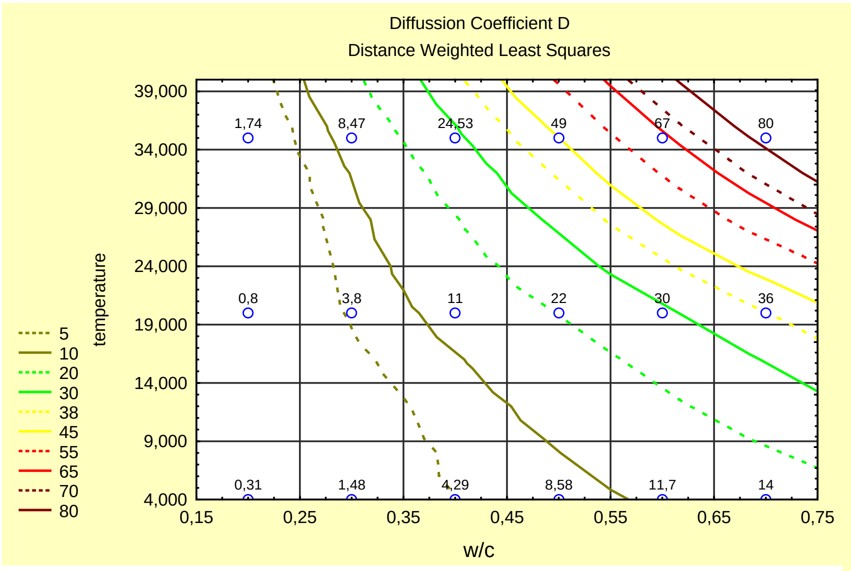

Figure 2. The diffusion coefficient D (10^{-12}m^2/s) as a function of the water-cement ratio *w/c* and the temperature Φ °C (centigrade); see Thoft-Christensen [9], [10].

5. RELIABILITY-BASED MAINTENANCE OF BRIDGE DECKS

A major problem in connection with reliability-based maintenance is the need during the inspection to estimate the reliability of the reinforced concrete deck. A direct estimation will require information of the degree of corrosion in the reinforcement, but this information is difficult or at least in most cases expensive to obtain. By relating the reliability to the corrosion crack width it would be easier to get an estimate of the reliability simply by measurements of the crack width.

For a reinforced bridge slab the bending strength is approximately proportional to the area of the reinforcement (or to the rebar diameter D_{bar}). After the initial cracking of the concrete the reduction of the diameter D_{bar} is proportional to the corrosion crack width w_{crack}. Therefore, the reduction in bending strength is approximately proportional to the increase in w_{crack}. As a consequence of this, the reliability with regard to the considered bending failure mode can be directly related to the crack width w_{crack}. In principle, the reliability can be estimated simply by measuring the crack width w_{crack} if only deterioration due to corrosion of the reinforcement is considered.

6. OPTIMAL REPAIR STRATEGIES FOR BRIDGE DECKS

After a structural assessment of the reliability of a reinforced concrete bridge deck at the time T_0 the problem is to decide if the bridge deck should be repaired and, if so, how and when should it be repaired? Solution of this optimisation problem requires that all future inspections and repairs are taken into account. In a decision model proposed in the European research project BREU 3091 [11] some approximations are introduced. After each structural assessment the total expected benefits minus expected repair and failure costs in the residual lifetime of the bridge are maximized considering only the repair events in the residual service life of the bridge.

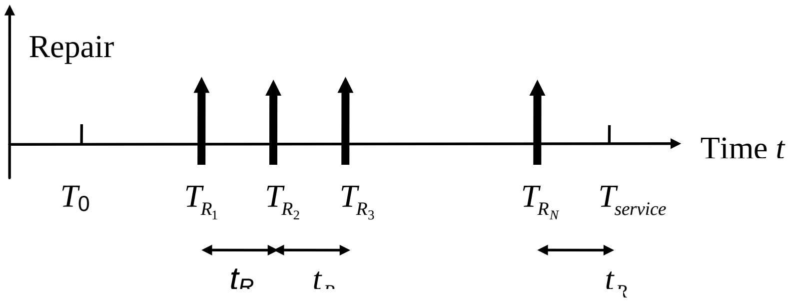

Figure 3: Repair strategy.

In order to simplify the decision modelling it is assumed that N_R repairs of the same type are performed in the residual service life $T_{service}$ of the bridge. The first repair is performed at time T_{R_1}, and the remaining repairs are performed at equidistant times with the time interval $t_R = (T_{service} - T_{R_1}) / N_R$, see figure 3. This decision model can be used in an adaptive way if the model is updated after an assessment (or repair) and a new optimal repair decision is made with regard to t_R. Therefore, it is mainly the time T_{R_1} of the first repair after an assessment, which is of importance. In order to decide which repair type is optimal after a structural assessment; the following optimisation problem is considered for each repair technique; Thoft-Christensen [12]:

$$\begin{aligned} &\max_{T_R, N_R} W(T_R, N_R) = B(T_R, N_R) - C_R(T_R, N_R) - C_F(T_R, N_R) \\ &s.t. \quad \beta^U(T_{service}, T_R, N_R) \geq \beta^{\min} \quad \text{or/and} \quad T_{service}(T_R, N_R) \geq T_{service}^{\min} \end{aligned} \tag{9}$$

where the optimisation variables are the expected number of repairs N_R in the residual service life and the time T_R of the first repair. W are the total expected benefits minus costs in the residual lifetime of the bridge. B is the benefit. C_R is the repair cost capitalized to the time t = 0 in the residual service life of the bridge. C_F are the expected failure costs capitalized to the time t = 0 in the residual service life of the bridge. $T_{service}$ is the expected service life of the bridge. β^U is the updated reliability index. $\beta^{\min}$ is the minimum reliability index for the bridge (related to a critical element or to the total system). $T_{service}^{\min}$ is the minimum acceptable service life. The inspection costs are not included in the optimisation problem, since they do not influence the choice of repair action in the present modelling.

The benefits B play a significant role and are modelled by

$$B(T_R, N_R) = \sum_{i=[T_0]+1}^{[T_{service}]} B_i (1+r)^{T_0 - T_{ref}} \frac{1}{(1+r)^{T_i - T_0}} \tag{10}$$

where $[T]$ signifies the integer part of T measured in years and B_i are the benefits in year i (time interval $\left[T_{i-1}, T_i\right]$. T_i is the time from the construction of the bridge. The i^{th} term in (10) represents the benefits from T_{i-1} to T_i. The benefits in year i is modelled by $B_i = k_0 V(T_i)$ where k_0 is a factor modelling the average benefits for one vehicle passing the bridge. k_0 is estimated by the cost of rental of an average vehicle/km times the average detour length. The reference year for k_0 is T_{ref}. It is assumed that bridges are considered in isolation. Therefore, the benefits are the considered marginal benefits by having a bridge at all (with the alternative that there is no bridge, but other nearby routes for traffic). V is the traffic volume per year which is estimated by $V(T) = V_0 + V_1(T - T_{ref})$, where V_0 is the traffic volume per year at the time of construction, V_1 is the increase in traffic volume per year and T is the actual time (in years).

The expected repair costs C_R capitalized to the time t = 0 are modelled by

$$C_R(T_R, N_R) = \sum_{i=1}^{N_R} (1 - P_F^U(T_{R_i}))C_{R_=}(T_{R_i}) \frac{1}{(1+r)^{T_{R_i} - T_0}} \tag{11}$$

$P_F^U(T_R)$ is the updated probability of failure in the time interval $]T_0, T_R]$. The updating is based on a no failure event and the available inspection data at the time T_0. The factor $(1 - P_F^U(T_{R_i}))$ models the probability that the bridge has not failed at the time of repair. r is the discount rate. $C_{R_0}(T_{R_i})$ is the cost of repair modelled by $C_{R_0}(T_{R_i}) = C_{R,func} + C_{R,fixed} + C_{R,unit}$, where the three terms are the functional repair costs, the fixed repair costs, and the unit dependent repair costs, respectively. The first term in (11) represents the functional costs and the last two terms represent the direct repair costs. The functional repair costs are modelled by $C_{R,func} = \frac{n}{365} \frac{n_L}{t_{n_L}} k_1 V(T_{R_i})(1+r)^{T_0 - T_{ref}}$, where n is the duration of the repair in days, n_L are the number of lanes closed down for the repair, t_{n_L} is the total number of lanes, and k_1 is a factor used to model the marginal functional repair costs for one vehicle. If the bridge is totally closed, then $k_1 = k_0$. The fixed costs are modelled by $C_{R,fixed} = k_{F_i} L_B + R_i r_{R_i}$, where $k_{F\,i}$ is a coefficient modelling the costs due to the distance from the headquarter [ECU/km], L_B is the distance from the headquarter [km], R_i are the roadblock costs for a period of 8 hours per lane, and r_{R_i} is the number of 8 hour periods needed to perform the repair of the bridge. The unit costs are modelled by $C_{R,unit} = f_D f_T Q_i (k_{L_i} c_h + k_{M\,i})$, where f_D is a factor which depends on how easy the defect is to repair (1.0, 1.3 or 1.5), f_T is a factor which depends on he time needed to perform the repair (1.0, 1.3 or 1.5), Q_i is a quantity describing the extent of the repair using the relevant repair technique, k_{L_i} are the man hours needed per unit of parameter Q_i for the repair technique considered [hours/unit], c_h is man hour cost [ECU/h], $k_{M\,i}$ is the material/equipment cost per unit of parameter Q_i.

The capitalized expected costs C_F due to failure are determined by

$$C_F(T_R, N_R) = \sum_{i=1}^{N_R+1} C_F(T_{R_i})(P_F^U(T_{R_i}) - P_F^U(T_{i-1_i}))\frac{1}{(1+r)^{T_{R_i}}} \tag{12}$$

where $T_{R_0} = T_0$ is the time of the structural assessment and $T_{R_{N_R+1}} = T_{service}$ is the expected service life. The i^{th} term in (12) represents the expected failure costs in the time interval $]T_{R_{i-1}}, T_{R_i}]$. $C_F(T)$ is the cost of failure at the time T and is modelled by $C_F(T) = (C_{F_0} + \frac{n_r}{365} k_0 V(T))(1+r)^{T_0 - T_{ref}}$, where C_{F_0} are the direct failure costs, and n_r is the number of days needed for replacement of the failed bridge. The first part of (12) represents the direct failure costs and the second part represents the functional failure costs modelled by loss of benefit.

7. IMPLEMENTATION OF THE OPTIMISATION PROBLEM

The optimisation problem formulated above has been implemented in the project BREU P3091 [11]. An expert system called BRIDGE2 developed within BREU P3091 contains a number of submodules, which can be used to find solutions to the optimisation problem formulated in this paper. Some aspects of BRIDGE2 are briefly presented below. A more detailed presentation is given by Thoft-Christensen, P. [12].

The reliability of the bridge is estimated using the reliability index β for a single failure mode or the system reliability index β^S for the structural system (the bridge), see Thoft-Christensen & Murotsu [13]. When new information from e.g. an inspection becomes available, the estimates of the reliability of the bridge are updated using Bayesian statistical theory. The reliability indices before and after the inspection are used together with expert knowledge to decide whether a structural assessment is needed or not. The solution of the optimisation problem above is performed for a large number of relevant repair techniques to determine the optimal repair technique, the optimal time for the repair, the benefits minus repair and the repair costs.

8. CONCLUSIONS

It is shown in the paper how the service life for a reinforced concrete deck may be based on a critical corrosion crack width. Further, the corrosion crack width is closely related to the reliability with regard to bending failure. The service life may therefore be evaluated simply by measuring the corrosion crack width.

A recent stochastic model for an optimal repair strategy is also discussed in the paper. The expected benefits of having the bridge operational minus the expected repair and failure costs are maximized with reliability or service life constraints.

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