Papers

Volume 6: 2001-2003

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CHAPTER 114

CORROSION CRACK BASED ASSESSMENT OF THE LIFE-CYCLE RELIABILITY OF CONCRETE STRUCTURES[1]

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ABSTRACT

In this paper it is proposed that the width of corrosion cracks in structural elements is used as an indicator of the corrosion degree. The so-called reliability profile of a structure is by this new procedure estimated on the basis of the corrosion cracks and not as usual based on the initiation of corrosion of the reinforcement only. By this procedure simple measurement of the width of the corrosion cracks may be used to estimate the decrease in reliability with time for corroded structural elements. A related new definition of service life is also discussed.

1. INTRODUCTION

Life-cycle assessment of the structural reliability of a reinforced concrete structure is based on modelling of the deterioration of the concrete. For optimization of e.g. maintenance strategies a stochastic modelling is sufficient since the uncertainties can only be treated satisfactorily in that way. However, if a single structure like a reinforced concrete bridge is considered then a more precise modelling of the deterioration of that special structure is required.

Deterioration of reinforced concrete is mainly due to some kind of penetration of the concrete. In many countries the most serious deterioration is corrosion of the reinforcement due to chloride penetration of the concrete. However, modelling of the initiation of corrosion of the reinforcement and the corrosion development with time are functions of several parameters such as the chloride content on the surface of the

[1] Proceedings of ICOSSAR'01 (cd-rom), Newport Beach, California, USA, June 2001.

concrete as a function of time and the density of the concrete as a function of space. The situation is so complicated that a number of severe assumptions must be made to obtain a workable modelling. A fully satisfactory modelling of the corrosion of the reinforcement has not yet been established.

Modelling of the corrosion initiation is often based on Fick's law of diffusion; see e.g. Thoft-Christensen [1]. After initiation of corrosion in the reinforcement it is often assumed that the cross-section of the reinforcement decreases with time. By this modelling it is simple to perform a deterministic or stochastic estimate of the so-called reliability that is the capacity or reliability as functions of time; see e.g. Thoft-Christensen [1]. On this basis optimal bridge management systems may be established: see Thoft-Christensen [2].

The above-mentioned procedure based on stochastic modelling seems to be useful for groups of bridges, where averaging of parameters can be used. The situation is more complex if a single bridge is considered. The degree of corrosion at a given point of time can generally not be directly observed on the surface of e.g. a reinforced concrete beam. The only indication of corrosion is a change in the colour of the surface and/or a complicated pattern of corrosion cracks; see figure 1. This paper addresses a methodology by which the reliability of corroded structural elements can be evaluated based on measurements of the crack width.

Figure 1. Corrosion cracking of a reinforced concrete beam.

2. RELIABILITY PROFILES

The reliability profile for a reinforced concrete structure is shown in Figure 2. The reliability profile consists of 6 parts:

1. Chloride penetration
2. Corrosion initiation
3. Corrosion evolution
4. Initial cracking
5. Crack evolution
6. Spalling.

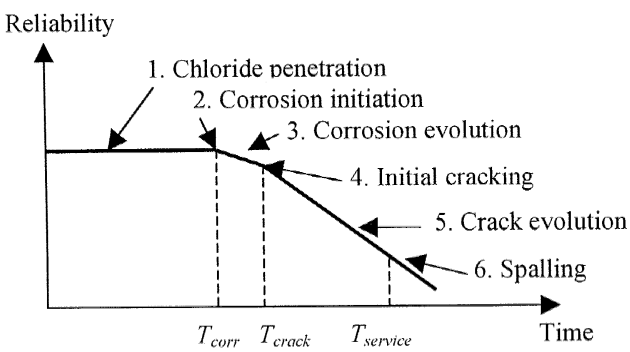

Figure 2. Reliability profile.

Deterioration steps 1-3 are well understood, and are presented in numerous papers, e.g. Thoft-Christensen [3]. Step 4 has been addressed in Thoft-Christensen [4], but steps 5 and 6 have only recently been investigated in this connection, Thoft-Christensen [5].

Thoft-Christensen [3] defined the service life $T_{service}$ for a reinforced concrete structure as the initiation time for corrosion T_{corr} of the reinforcement

$$T_{service} = T_{corr} \tag{1}$$

Using diffusion modelling of the chloride penetration of the concrete it is shown based on simulation data that the corrosion initiation time for a considered example may be modelled by a Weibull distribution. This approach based on diffusion theory seems to have reached general acceptance among researchers in this field.

Thoft-Christensen [4] has modified the service life definition so that the time ΔT_{crack} from corrosion initiation to corrosion crack initiation in the concrete is included. The service life is then defined as

$$T_{service} = T_{crack} = T_{corr} + \Delta T_{crack} \tag{2}$$

The stochastic modelling of ΔT_{crack} is based on existing deterministic theories for crack initiation; see Liu & Weyers [6]. The corrosion-cracking model is restricted to the stresses resulting from the expansion of the corrosion products. In Thoft-Christensen [5], a new service life definition is introduced

$$T_{service} = T_{crack} + \Delta T_{service} = T_{corr} + \Delta T_{crack} + \Delta T_{service} \tag{3}$$

where $\Delta T_{critical}$ is the time from initial cracking to a critical crack width is developed. By (3) the definition is extended to include the situation after initiation of corrosion cracking.

3. CRACK INITIATION

3.1 Free expansion

The time to crack initiation is estimated by calculating the amount of corrosion products and estimate the space needed for these products. It is assumed that there is a porous zone around the steel/concrete surface caused by the transition from paste to steel, entrapped/entrained air voids, and corrosion products diffusing into the capillary voids in the cement paste. At a certain time the total amount of corrosion products exceeds the amount of corrosion products needed to fill the porous zone around the steel. The corrosion products will then create expansive pressure on the surrounding concrete.

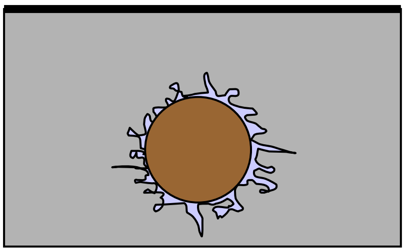

Figure 3. The porous zone.

Some porosity will always exist in the concrete, especially close to reinforcement bars. Very close to the bars the porosity is close to one, but usually the porosity decreases with the distances from the bars. The porosity is typically of the order of 0.5 about 10-20 μm from the bars so the porous zones are very narrow; see figure 4. Define t_{por} as the thickness of an equivalent zone with porosity 1 around a steel bar. Then the amount of corrosion products necessary to fill the porous zone is

$$W_{porous} = \pi \rho_{rust} t_{por} D \quad (4)$$

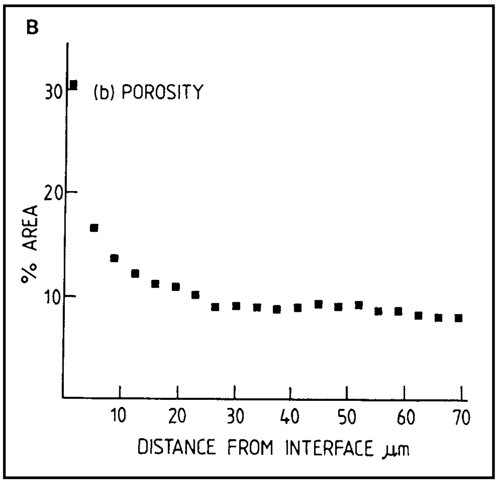

Figure 4. Estimation of the porosity of the concrete near the reinforcement; Scrivener. Crumbie & Pratt [7].

where D is the diameter of the reinforcement bar and ρ_{rust} is the density of the corrosion products.

For illustration, let t_{por} be modelled by a lognormal distribution with the mean 12.5 μm and a standard deviation of 2.54 μm. Further, let ρ_{rust} and D be modelled by normal distributions N(3600, 360) kg/m^3 and N(16, 1.6) mm, respectively.

Then by Monte Carlo simulation it can be shown that W_{porous} with a good approximation can be modelled by a shifted lognormal distribution with a mean $2.14e-03$ kg/m, a standard deviation $0.60e-03$ kg/m and a shift of $0.82e-03$ kg/m, see figure 5.

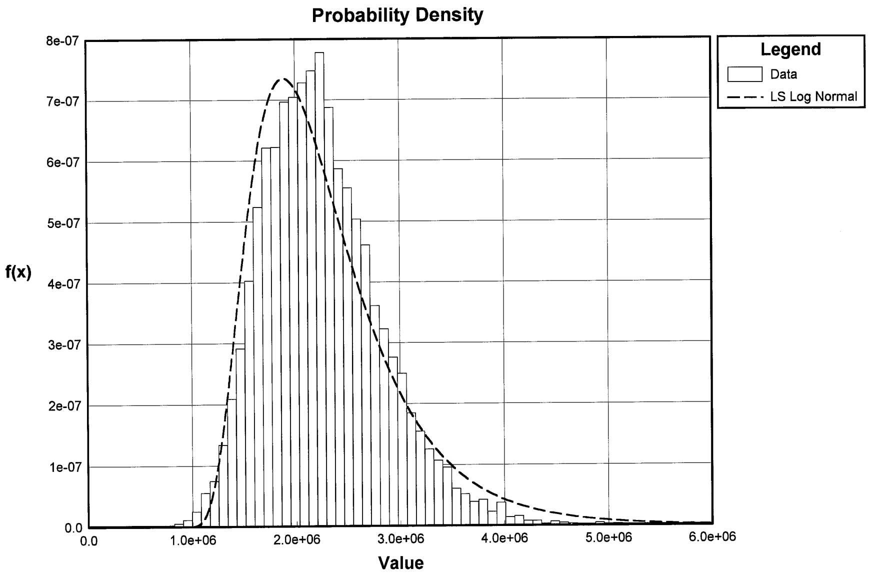

Figure 5. Stochastic modelling of W_{porous}. P. Thoft-Christensen [4].

3.2 Crack initiation

Initially the rust products will fill the interconnected porous zone completely and then result in an expansion of the concrete near the reinforcement. A result of this is that tensile stresses are initiated in the concrete. After some time with increasing corrosion the tensile stresses will reach a critical value, when the reinforcement is relatively close to the surface of the concrete, and corrosion cracks will be developed.

During this process the corrosion products at initial cracking of the concrete will occupy three volumes

- the porous zone,
- the expansion of the concrete due to rust pressure, and

- the space of the corroded steel.

The corresponding total amount of critical rust products W_{crit} to fill these volumes is

$$W_{crit} = W_{porous} + W_{expan} + W_{steel} \tag{5}$$

W_{expan} is the amount of corrosion products needed to fill in the space due to the expansion of the concrete around the reinforcement, and W_{steel} is the amount of corrosion products that generate the cracking.

Let the expansion of the concrete around the reinforcement be t_{crit} (the critical thickness) at the time of crack initiation. W_{expan} can then be written

$$W_{expan} = \rho_{rust}\pi(D + 2t_{por})t_{crit} \tag{6}$$

Liu & Weyers (1998) have estimated t_{crit} assuming that the concrete is a homogeneous elastic material and can be approximated by a thick-walled concrete cylinder with inner radius $a = (D + 2t_{por})/2$ and outer radius $b = c + (D + 2t_{por})/2$, where c is the cover depth. Then an approximate value of the critical expansion t_{cr} is

$$t_{crit} = \frac{cf_t'}{E_{ef}}(\frac{a^2 + b^2}{b^2 - a^2} + \nu_c) \tag{7}$$

E_{ef} is the effective elastic modulus of the concrete and f_t' is the tensile strength of the concrete. ν_c is Poisson's ratio of the concrete.

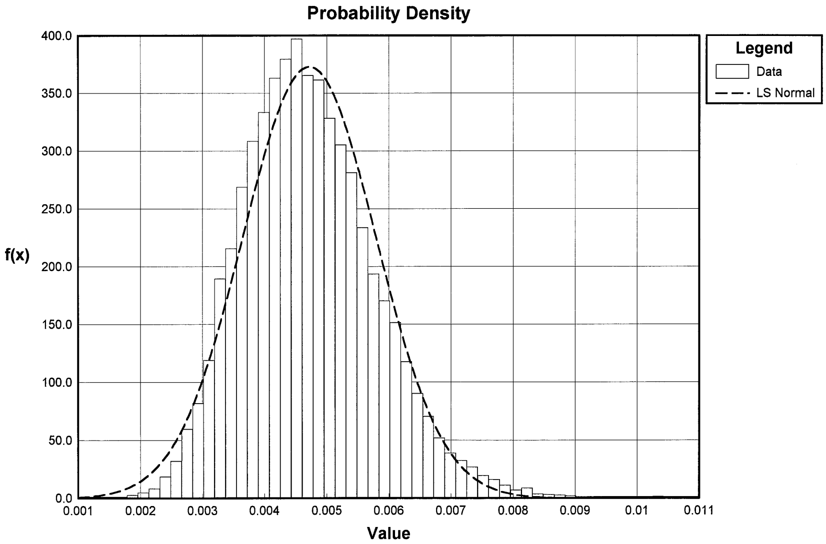

Figure 6. Stochastic modelling of W_{expan}. P. Thoft-Christensen [4].

In this paper E_{ef}, ν_c, and c are considered deterministic with values 10 GPa, 0.25, and 60 mm, respectively. The tensile strength f_t' is modelled as a normally distributed variable with the mean value 4 MPa and the standard deviation 0.6 MPa. Let ρ_{steel} be modelled by a normal distribution N(8000 ; 800) kg/m^3. Then by Monte Carlo simulation it can be shown that W_{crit} with a good approximation can be modelled by a normal distribution N(0.010 ; 0.0027) kg/m, see figure 6.

The rate of rust production as a function of the time t (years) from corrosion initiation can be written; Liu & Weyers [6]

$$\frac{dW_{rust}(t)}{dt} = k_{rust}(t)\frac{1}{W_{rust}(t)} \tag{8}$$

i.e. the rate of corrosion dW_{rust}/dt is assumed to be inversely proportional to the amount of rust products W_{rust} (kg/m). The factor $k_{rust}(t)$ (kg^2/m^{2}t) is assumed to be proportional to the annual mean corrosion rate $i_{cor}(t)$ (μ A/cm^2) and the diameter D (m) of the reinforcement. The proportionality factor $k_{rust}(t)$ depends on the types of rust products, but is here taken as 0.383e$-$3.

$$k_{rust}(t) = 0{,}383 \times 10^{-3} D i_{corr}(t) \tag{9}$$

By integration

$$W_{rust}^2(t) = 2\int_0^t k_{rust}(t)dt \tag{10}$$

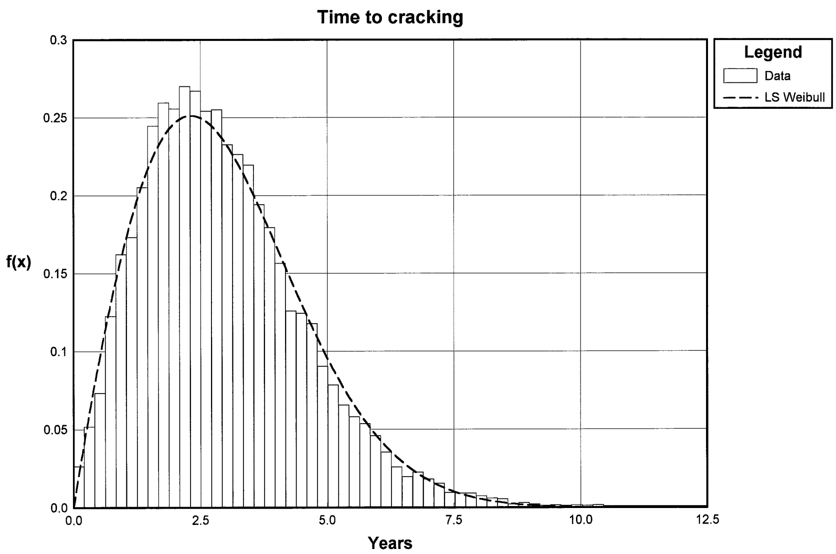

Figure 7. Stochastic modelling of ΔT_{crack}. P. Thoft-Christensen [4].

Let $i_{cor}(t)$ be modelled by a time-independent normally distributed stochastic variable N(3 ; 0.3) (μA/cm^2) then the time from corrosion initiation to cracking ΔT_{crack} can be estimated by (10) by setting $W_{rust}(\Delta T_{crack}) = W_{crit}$.

$$\Delta T_{crack} = \frac{W_{crit}^2}{2k_{rust}} = \frac{W_{crit}^2}{2 \times 0.383 \times 10^{-3} D i_{corr}} \tag{11}$$

By Monte Carlo simulation it can be shown that with a good approximation ΔT_{crack} can be modelled by a Weibull distribution W(3.350 ; 1.944 ; 0) years, see figure 7. The mean is 2.95 years and the standard deviation 1.58 years.

4. CRACK EVOLUTION

The evolution of corrosion cracks in reinforced concrete beams has been investigated experimentally by Andrade, Alonso & Molina [8]. After the initial crack has been formed the rebar cross-section is further reduced due to the continued corrosion, and the width of the crack is increased. In the paper by Andrade, Alonso & Molina [8] four simple test specimens have been investigated. The specimens are simplified small reinforced concrete beams with only a single rebar and 2 or 3 cm of cover. An impressed current artificially corrodes the concrete beams. The reduction of the rebar sections is monitored and the corresponding crack evolution is measured by the use of strain gauges attached to the surface of the beams. In all four experiments the relationship between the reduction of the rebar diameter ΔD and the maximum crack width Δw measured at the surface of the concrete specimen can be approximated by a linear function, see figure 8.

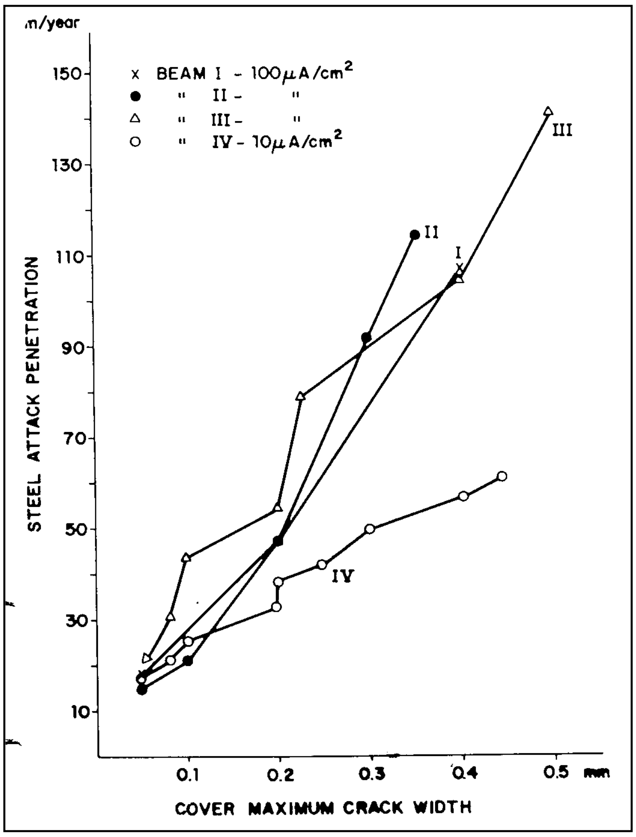

Figure 8. Loss in rebar diameter versus the crack width. Andrade, Alonso & Molina [8].

Let Δw be the increase in crack width in the time interval Δt and let the corresponding loss of rebar diameter be ΔD. Then $\Delta w = \gamma \, \Delta D$, where γ is of the order 1.5 to 5 in the experiments reported by Andrade, Alonso & Molina [8].

A simple approximate estimation of γ for a given beam cross-section may be performed as shown below. For illustration, assume that the diameter of the rebar at the time of crack initiation is 16 mm and that the cover is 30 mm, see figure 9.

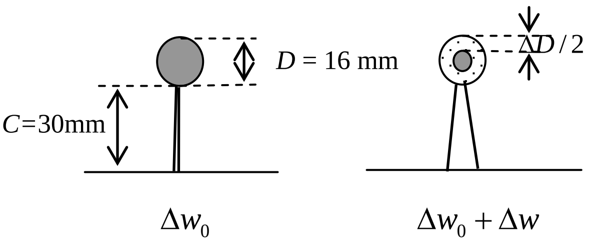

Figure 9. Evolution of cracks.

To the left in figure 9 the crack at the time of the initial crack forming is shown. The initial crack width is Δw_0. To the right the assumed crack configuration at the time when the diameter loss is ΔD is shown. The crack width is then $\Delta w_0 + \Delta w$. By assuming that the increase in the internal volume of the crack is equal to the volume of the corrosion products produced when the diameter is reduced to $D - \Delta D$, the relationship between Δw and D can be obtained approximately by

$$\frac{1}{2}\left(\frac{D/2}{D/2+c}+1\right)c\Delta w=(\alpha-1)\pi D\frac{\Delta D}{2} \tag{12}$$

$\alpha=\rho_{rust}/\rho_{steel}$ (the relation between the densities of the rust product and the steel) depends on the type of corrosion products. Typical values of α are 2 - 4. By inserting the above-mentioned data into (12) one obtains for this case $\gamma=1.4-4.2$ in good agreement with the experimental results described by Andrade, Alonso & Molina (1993.

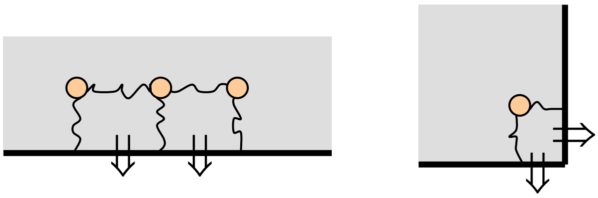

Figure 10. Typical examples of concrete spalling.

Using the Finite Element Methods the procedure presented above can be extended to estimating the time for corrosion based spalling of concrete for e.g. slabs and beams, see figure 10.

5. CRACK WIDTH LIMIT STATES

Limit states are, according to EUROCODE 2, states beyond which the structure no longer satisfies the design performance requirements. *Ultimate limit states* (ULS) are those associated with collapse or with other forms of structural failure, which may endanger the safety of people. Examples are loss of equilibrium of the structure or failure due to excessive deformation, rupture or loss of stability. *Serviceability limit states* (SLS) correspond to states beyond which specified service requirements are no longer met. Examples are deformations or deflections, vibrations, cracking of concrete, or damaging of concrete.

A corrosion and crack based serviceability limit state can be formulated as

$$M(t)=w_{service}-w(t) \tag{13}$$

where the critical serviceability corrosion crack width $w_{service}$ is e.g. 0.3 mm and

$$\begin{aligned} w(t) &= w_0+\gamma(D(T_{crack})-D(t)) \\ &= w_0+\gamma c_{corr} i_{corr}(t-T_{crack}) \end{aligned} \tag{14}$$

where T_{crack} is the time of initial cracking. The service life $T_{service}$ may then be calculated by

$$\begin{aligned} w(T_{service}) = w_{service} \Rightarrow T_{service} &= \frac{w_{service}-w_0}{\gamma \quad c_{corr} i_{corr}}+T_{crack} \\ &= a\times\frac{1}{c_{corr} i_{corr}}+T_{crack}+T_{corr} \quad =\Delta T_{critical}+\Delta t_{crack}+T_{corr} \end{aligned} \tag{15}$$

where $a=(w_{service}-w_0)/\gamma$.

Using Monte Carlo simulation the distribution function of $T_{service}-T_{corr}$ can then for a given structure be estimated for any value of the serviceability crack width. In this way it can be evaluated whether it is a benefit to include crack estimations in the definition of service lifetime. As an example such simulations (10000 samples) have been performed for six values of the factor a: 0.05; 0.10; 0.15; 0.20; 0.25; 0.30 and with c_{corr} modelled as a uniformly distributed variable in the interval [0.5 ; 2.0] and i_{corr} modelled as an uniformly distributed variable in the interval [0.02 ; 0.05]. Δt_{crac} is modelled as a Weibull distribution with the mean value 2.95 years and the standard deviation 1.58 years.

a	μ (years)	σ (years)
0.05	4.38	1.75
0.10	5.77	2.14
0.15	7.18	2.64
0.20	8.65	3.28
0.25	10.07	3.92
0.30	11.47	4.60

Table 1. Mean values and standard deviations of simulated data.

For each of the six simulations table 1 shows the mean values μ and the standard deviations σ of the simulated data. The simulated data may be approximated by Weibull distributions, see figures 11 to 16.

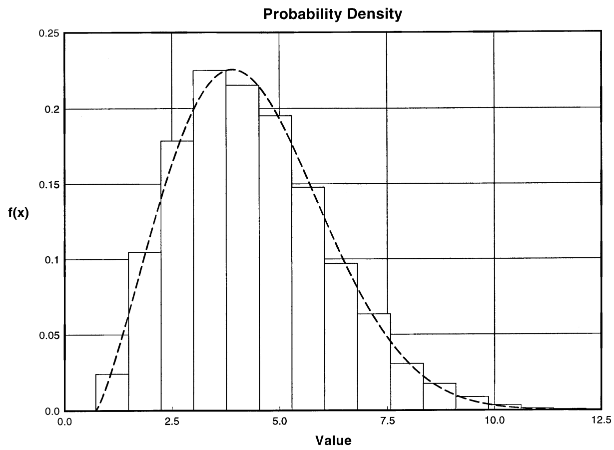

Figure 11. Stochastic modelling of $T_{service} - T_{corr}$ for a=0.05. W(4.8548, 2.25559, 0.718858).

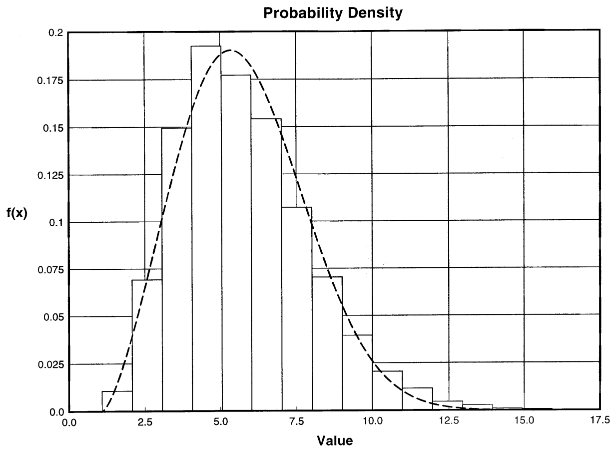

Figure 12. Stochastic modelling of $T_{service} - T_{corr}$ for a=0.10. W(6.3527, 2.47358, 1.08852).

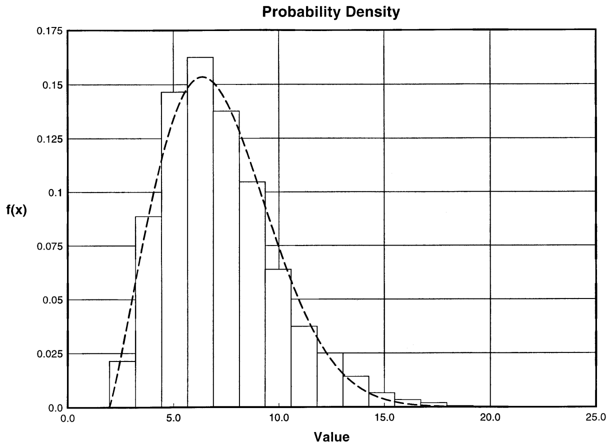

Figure 13. Stochastic modelling of $T_{service} - T_{corr}$ for a =0.15. W(7.85446, 2,15249, 1.97816).

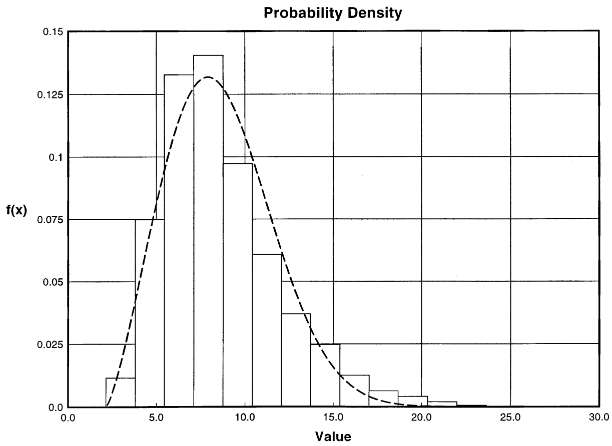

Figure 14. Stochastic modelling of $T_{service} - T_{corr}$ for a =0.20. W(9.44344, 2.34866, 2.14752).

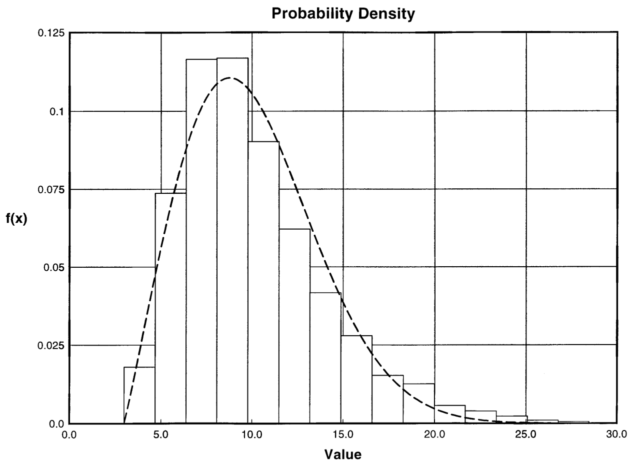

Figure 15. Stochastic modelling of $T_{service} - T_{corr}$ for a =0.25. W(10.935, 2.07848, 2.97745).

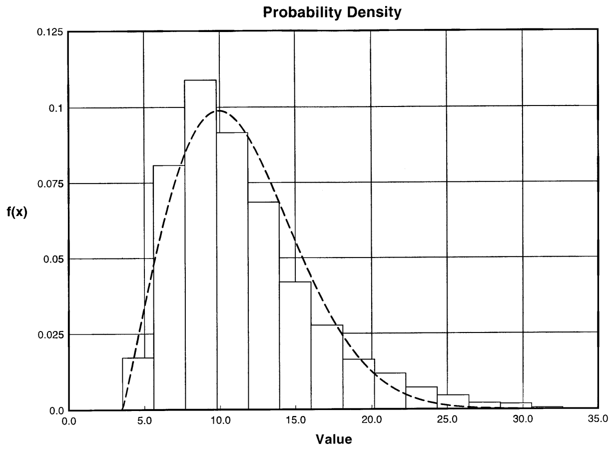

Figure 16. Stochastic modelling of $T_{service} - T_{corr}$ for a =0.30. W(7.85446, 2,15249, 1.97816).

6. CRACK WIDTH AND RELIABILITY

In this section it is shown how the corrosion crack width may be used to estimate the reliability of a given cross-section of a reinforced concrete structural element. As an example consider the simple cross-section of a beam shown in figure 17 with only one reinforcing steel bar. More complicated beams can be treated in a similar way.

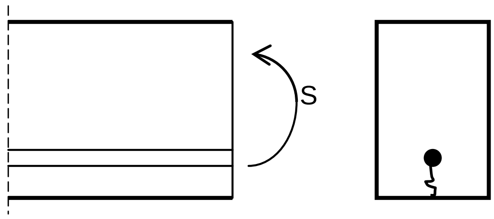

Figure 17. Cross-section of beam with corrosion crack.

Assume that the cross-section is subjected to a constant bending moment *S* and that the bending moment capacity *R(t)* is proportional to the cross-sectional area of the reinforcement *A(t)* with the proportionality factor *b*. The safety margin *M(t)* may then be written

$$M(t) = R(t) - S = b \times A(t) - S \tag{16}$$

To obtain the relationship between the reliability with regard to the considered safety margin and the crack width, a relationship between the cross-sectional area *A*(t) and the crack width *w(t)* must be established for a given value of t.

For $0 < t < T_{corr}$, the area *A(t)* of the rebars is still equal to the original area $A(0) = A_0$. Therefore, in that interval the reliability index $\beta(t)$ is constant and equal to the original reliability index $\beta(0) = \beta_0$. The crack width *w(t)* is equal to 0.

For $T_{corr}<t<T_{crack}$, the area *A(t)* is reduced linearly in the time *t* but the crack width *w(t)* is still equal to 0. Let $A_{crack} = A(T_{crack})$.

For $t > T_{crack}$ the relationship between *A(T)* and *w(t)* can be estimated in the following way. When *D* is much smaller than *c*, equation (12) can be modified to the following differential form

$$c\Delta w = (\alpha - 1)\pi D \Delta D \tag{17}$$

By integration

$$A(t) = A_{crack} - \frac{c}{2(\alpha - 1)} w(t) \tag{18}$$

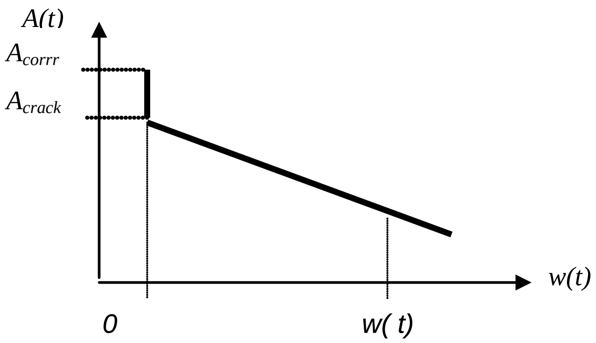

Figure 18. Relationship between cross-sectional area *A(t)* and corrosion crack width w(t).

The relationship between the cross-sectional area *A(t)* of the reinforcement bar and the corrosion crack width *w(t)* is illustrated in figure 18. For a given structure with a measured crack width, the corresponding cross-sectional area can be calculated by equation (18). Then by using (16), the reliability index may be estimated.

7. CONCLUSIONS

In the paper stochastic models for crack initiation and crack evolution are proposed. A new crack based serviceability limit state is presented. It is shown for a number of examples that the time from corrosion to a given crack width may be modelled by Weibull distributions. Finally, it is shown how the reliability of given corroded structural element may be estimated based on measurement of the corrosion crack width.

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