Modelling the Loss of Steel-Concrete Bonds in Corroded Reinforced Concrete Beams

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Modelling the loss of steel-concrete bonds in corroded reinforced concrete beams

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ABSTRACT: The existing stochastic models for deterioration of reinforced concrete structures is extended by adding modelling of "loss of bond" due to corrosion between the reinforcement bars and the surrounding concrete.

1 INTRODUCTION

Life-cycle assessment of the structural reliability of a reinforced concrete structure is based on modelling of the deterioration of the concrete. The most serious deterioration is often corrosion of the reinforcement due to chloride penetration of the concrete. 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 (1998). 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 profile that is the capacity or reliability as functions of time; see e.g. Thoft-Christensen (1998).

The reliability profile consists of six parts:

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

Deterioration steps 1-3 are well understood, and are presented in numerous papers, but steps 4, 5 and 6 have only recently been investigated in this connection; see e.g. Thoft-Christensen et al. (2005), (2007).

In this paper the effect of deterioration on steel-concrete bond in reinforced concrete structures is studied. A lot of experimental research in this area has been published; see e.g. FIB (2000). The drawback of this research is that the results are highly dependent on the specific structures considered. In a paper by Lundgren (2002) a more general approach based on a modelling of the corrosion layer between the steel and the concrete is taken. The results of this approach, and similar approaches used by other researchers, are in this paper used to formulate the bases for a stochastic model for loss of bond due to corrosion of the reinforcement. The above mentioned six parts of the reliability profile outlined above will hereby be increased with one more important subject of significance for several of the six parts of the reliability profile.

2 THE PHYSICAL EFFECTS OF CORROSION

Only chloride induced corrosion of the reinforcement is considered is paper. If the rate of chloride penetration into the concrete is modelled by Fick's law of diffusion, then the time to initiation of reinforcement corrosion may be evaluated. The result will depend on several factors namely the concrete cover, the diffusion coefficient, the critical chloride concentration at the site of the corrosion, the equilibrium chloride concentration on the concrete surface, and the initial chloride concentration in the concrete. All these parameters are modelled by stochastic variables or stochastic processes. The distribution of the corrosion initiation time may then be estimated using Monte-Carlo simulation.

With increasing corrosion the tensile stresses in the concrete 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 will occupy three volumes, namely the porous zone near the reinforcement, the expansion of the concrete due to rust pressure, and the space of the corroded steel. With this modelling and some minor simplifications the time from corrosion imitation to crack initiation may be estimated. The controlling parameters are the diameter of the reinforcement bar, the annual mean corrosion rate, the density of the steel, and the density of the rust products. All these parameters are modelled by stochastic variables or stochastic processes. The distribution of the crack initiation time may then be estimated using Monte-Carlo simulation.

After formation of the initial crack the rebar cross-section is further reduced due to the continued corrosion, and the crack width w_{crack} is increased. Several researchers have investigated the evolution of corrosion cracks in reinforced concrete beams experimentally. In the experiments an impressed current are normally used to artificially corrode the beams.

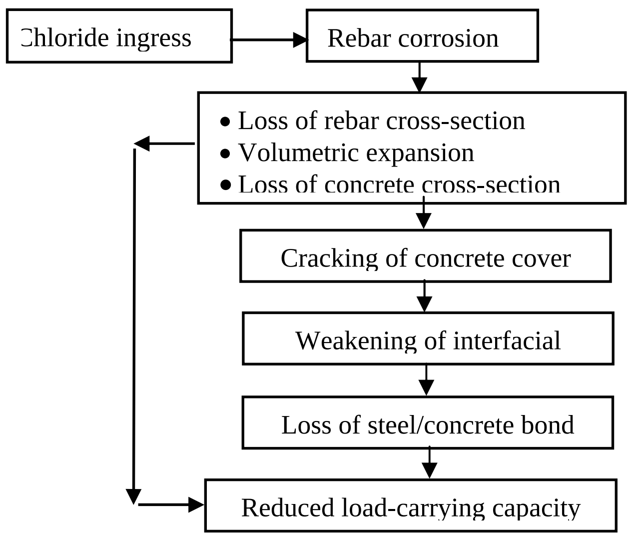

Figure 1. The physical effect of corrosion.

The corrosion process and its effect on a reinforced concrete structure are schematic illustrated in figure 1. The final result will be a reduced load-carrying capacity of the structure primarily due to the reduced steel bar cross-section and the loss of bond between the steel bar and the surrounding concrete.

3 THE BOND STRESS

In figure 2 is shown the interfacing zone between the steel bar and the surrounding concrete. The spitting stress component perpendicular to the rebar is due to the volume increase of the corrosion product. The bond stress parallel to the rebar is due to the effect of the corrosion on the friction between the reinforcement and the concrete.

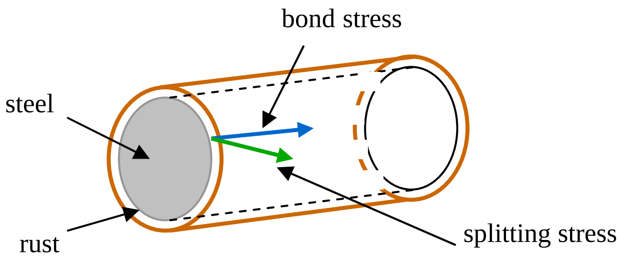

Figure 2. Definition of bond and splitting stresses.

Initially the chemical bond between the steel reinforcement and the surrounding concrete is weak. Therefore the weak bond is broken at a very low stress and only friction between the steel and the concrete contribute to the bond stress. For a plain round rebar friction is the main component to the bond stress. For ribbed bars the mechanical interlock between the ribs and the concrete plays a major role although the concrete between the ribs easily breaks.

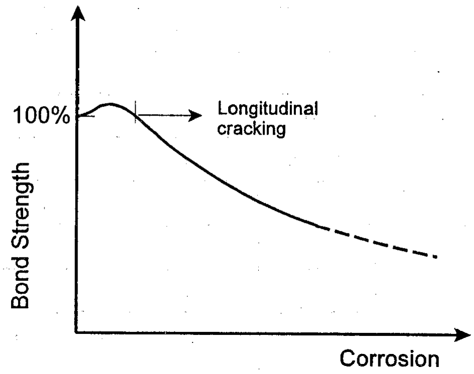

Figure 3. Schematic illustration of the variation in bond strength with corrosion (taken from FIB (2000)).

In figure 3 is shown the variation in the bond strength with corrosion. The bond stress is increased a little in the time period from corrosion initiation to longitudinal cracking is established due to the increased pressure between the reinforcement and the surrounding concrete. Rust is enclosed between the steel and the concrete and only a minor part of the rust may disappear into small cracks and voids in the concrete. When cracks are opened from the rebars to the surface of he concrete then the rust will to some extend be able to disappear into these open cracks and the pressure between the rebars and the surrounding concrete will decrease. The friction between the rebars will therefore also decrease and so will the bond stress.

4 CORROSION INDICES

In this section three important corrosion indices will be introduced, namely

- The crack /corrosion index γ equal to the increase of the surface crack width divided with the corresponding decrease of the rebar diameter
- The bond /corrosion index λ equal to the reduction of bond strength divided with the corresponding decrease of the rebar diameter.
- The bond /crack index η equal to the reduction in bond strength divided with the corresponding increase of surface crack

These 3 indices and will be discussed in more detail in the next chapters.

5 THE CRACK/CORROSION INDEX γ

Several researchers have investigated the evolution of corrosion cracks in reinforced concrete beams experimentally, e.g. Andrade et al. (1993). To reduce the duration of testing, artificial corrosion by electrical current is a common experimental approach. During the test, the loss of rebar cross-section is monitored and the corresponding crack evolution is measured by strain gauges attached to the surface of the specimen. In the study by Andrade et al. (1993), four simple test specimens were investigated. The specimens are small reinforced concrete beams with only a single rebar with a 20 or 30 mm concrete cover. In figure 4, the results from this experimental study are shown.

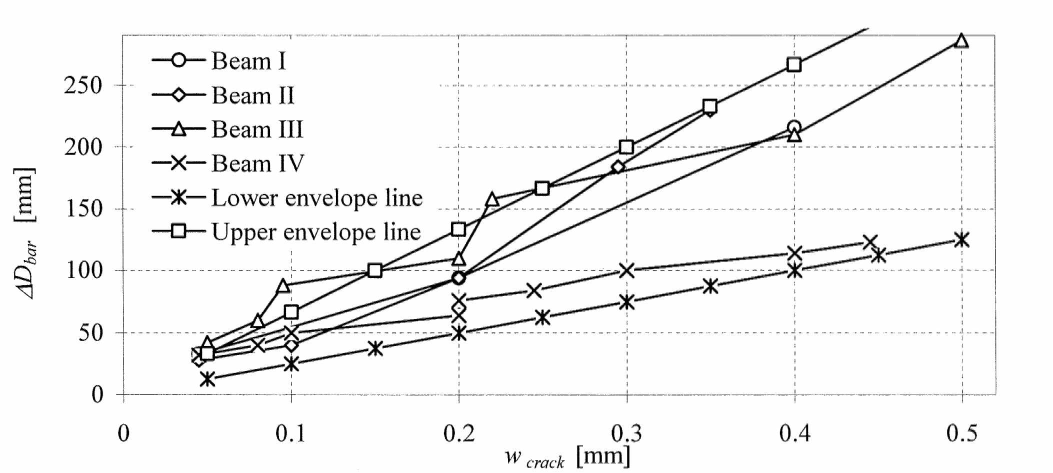

Figure 4. Reduction of rebar diameter ΔD_{bar} versus the crack width w_{crack}, based on data in Andrade et al. (1993).

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 as shown in figure 2. In all four experiments by Andrade et al. (1993), proportionality between the reduction of the rebar diameter ΔD_{bar} and the maximum crack Δw_{crack} width measured at the surface of the concrete specimen is found

$$\Delta w_{crack} = \gamma \, \Delta D_{bar} \tag{1}$$

where the crack-corrosion index γ is the proportionality constant. From the experiments shown in figure 4, the upper and lower bound for γ is determined as being of the order 1.5 to 5.

6 THE BOND/CORROSION INDEX η

In the literature there are a vast number of experimental tests on the relationship between the residual bond strength f_{bond} and the corrosion penetration. Empirical expressions for this relationship based on laboratory testing as been publisher by e.g. Rodriguez et al. (1994a), (1994b), (1996), Coronell & Gambarova (2004), Fang et al. (2004), and Söylev & Francois (2005).

The relationship has also been numerically studied by several researchers (using FEM analysis) such as Lundgren (2000), (2002) (2005a,b), Lundgren & Gylltoft (2000), and Berra et al. (2003).

The behaviour of reinforced concrete beams with loss of bond has also been studied by several researchers: see e.g. Jeppsson & Theladersson (2003), Harajli (2004), Toongoenthong & Maekawa (2000) & Darwin (2005).

Figure 5 (taken from FIB (2000)) shows a linear relationship between the residual bond strength f_{bond} and the corrosion penetration based on Rodriguez et al. (1994a), (1994b), and

(1996). The linear relation shown in figure 5 between the reduction of the bond strength $\Delta\tau_{bond}$ (%) and the decrease the rebar diameter ΔD_{bar} may be written

$$\Delta\tau_{bond} = \eta\,\Delta D_{bar} \tag{2}$$

where the bond/corrosion index η is equal to 50 %/mm.

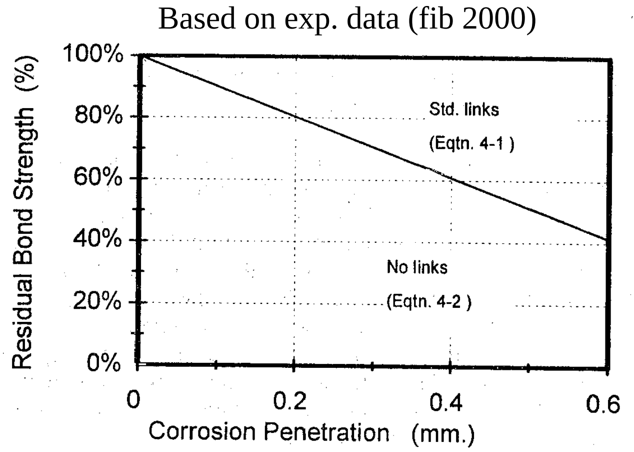

Figure 5. Relationship between residual bond strength f_{bond} and the corrosion penetration (taken from FIB (2000)).

7 THE BOND/CRACK INDEX λ

By combining (1) and (2) a linear relation between the reduction of the bond strength $\Delta\tau_{bond}$ (%) and the increase in crack width Δw_{crack} (mm) is obtained

$$\Delta\tau_{bond} = \frac{\eta}{\gamma}\Delta w_{crack} = \lambda\,\Delta w_{crack} \tag{3}$$

where the bond/crack index $\lambda = \gamma/\eta$ is of the order 10 – 60 %/mm depending of the beam cross-section.

In the paper by Cairns, Du & Law (2006) it is suggested that corroded plain bars the surface crack width Δw_{crack} may provide a better parameter than the corrosion penetration (0.5 ΔD_{bar}) in assessing the residual bond strength.

In figure 6 is shown some test data obtained by Cairns, Du & Law (2006). In the figure is the relationship between the bond strength and the surface crack width for bottom and top castings. Using non-linear curve fitting the obtained the following relation between the residual bond strength f_{bond} and the crack width w_{crack}

$$f_{bond} = \frac{1}{(1+0.8w_{crack})} f_0 \tag{4}$$

where f_0 is the design bond strength. If instead a linear curve fitting is used (the straight dotted line) one obtain approximately

$$f_{bond} = (1 - 0.30 w_{crack}) f_0 \tag{5}$$

and the bond/crack index λ = 30 %/mm. This is in good agreement with the estimations indicated above.

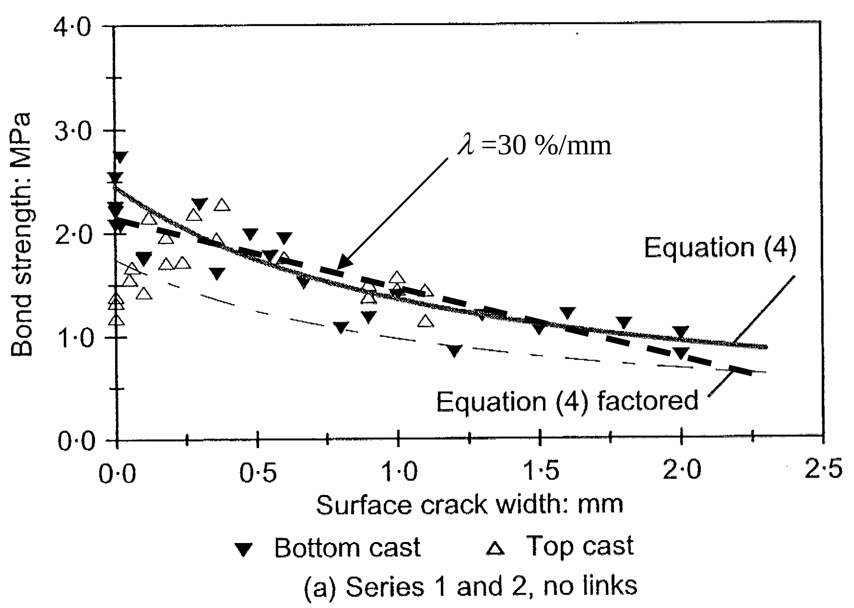

Figure 6. Relationship between bond strength f_{bond} and longitudinal crack width w_{crack} (from the top part of Fig. 8 in Cairns, Du & Law (2006)).

8. DISCUSSION

The final outcome of the deterioration due to corrosion of the reinforcement of a reinforced concrete beam is a reduced load-carrying capacity of the structure. This is primarily due to

- the reduced steel bar cross-section, and
- the loss of bond between the steel bar and the surrounding concrete.

In estimation of the load-carrying capacity, it is therefore of great value to be able to estimate the reduced steel bar cross-section (the corrosion of the rebar) and the loss of bond. In the paper is presented a new linear model by which the corrosion degree as well as the loss of bond may be evaluated by simple measurement of the concrete crack width at the surface of the structure.

The corrosion is estimated by the crack-corrosion index γ . γ may be estimated experimentally or numerical using two or three dimensional FEM techniques. The value of γ is typical in the interval 1.5 – 5. Due to the uncertainties involved in the estimation of the crack-corrosion index γ will normally be modelled as a stochastic variable.

The loss of bond is estimated by the bond/crack index $\lambda = \gamma / \eta$, where bond/corrosion index η may be estimated using the methods described in section 6. It is also possible to estimate λ directly by testing. η and λ are modelled as stochastic variables.

Using these linear models, it is possible estimate the load-capacity reduction. Like the above mentioned indices the load capacity will be modelled as a stochastic variable.

9. CONCLUSIONS

In the paper are derived two linear equations by which the amount of corrosion of the reinforcement as well as the loss of bond may be estimated by observing the width of cracks at the surface of the reinforced concrete element.

REFERENCES

Andrade, C., C. Alonso & F.J. Molina. 1993. Cover Cracking as a Function of Bar Corrosion: Part 1 - Experimental Test. *Materials and Structures*, Vol. 26, pp. 453-464.

Berra M., A. Castellani, D. Coronelli, S. Zanni & G. Zhang. (2003). Steel-Concrete Bond Deterioration due to Corrosion: Finite-Element Analysis for different Confinement Levels. *Magazine of Concrete Research*. Vol. 55, pp. 237-247.

Cairns, J., Y. Du & D. Law (2006). Residual Bond Strength of Corroded Plain Round Bars. *Magazine of Concrete Research.* Vol. 58, pp. 221-231.

Coronell, D. & P. Gambarova. 2004. Structural Assessment of Corroded Concrete Beams: Modelling Guidelines. *Journal of Structural Engineering,* Vol. 130, pp. 1214-1224.

Darwin, D. 2005. Tension Development Length and Lap Splice Design for Reinforced Concrete Members. *Prog. Structural Engineering Materials*. Vol. 7, pp. 210-225.

Fang, C., K. Lundgren, L. Chen & C. Zhu. 2004. Corrosion Influence on Bond in Reinforced Concrete. *Cement and Concrete Research*, Vol. 34, pp. 2159-2167.

FIB. 2000. Bond of reinforcement in concrete. State-of-the art report. Task Group Bond Models. FIB bulletin 10, Lausanne 2000.

Harajli, M.H. 2005. Comparison of Bond Strength of Steel Bars in Normal- and High-Strength Concrete. *Journal of Materials in Civil Engineering*. Vol. 16, pp. 365-374.

Jeppsson, J. & S. Thelandersson. 2003. Behavior of Reinforced Concrete Beams with Loss of Bond at Longitudinal Reinforcement. *Journal of Structural Engineering*, Vol. 129, pp. 1376-1383.

Lundgren, K. & K. Gylltoft 2000. A Model for the Bond between Concrete and Reinforcement. *Magazine of Concrete Research*, Vol. 52, pp. 53-63.

Lundgren, K. 2001. Bond between Corroded Reinforcement and Concrete. Department of Structural Engineering, Chalmers University of Technology, Sweden. Report No. 00:3, 2001

Lundgren, K. 2002. Modelling the Effect of Corrosion on Bond in Reinforced Concrete. *Magazine of Concrete Research*, Vol. 54, pp. 165-173.

Lundgren, K. 2005a. Bond between Ribbed Bars and Concrete. Part 1: Modified Model. *Magazine of Concrete Research*, Vol. 57, pp. 371-382 .

Lundgren, K. 2005b. Bond between Ribbed Bars and Concrete. Part 2: The Effect of Corrsion. *Magazine of Concrete Research*, Vol. 57, pp. 383-395.

Rodriguez, J., L.M. Ortega & A. M. Garcia. 1994a. Assessment of Structural Elements with Corroded Reinforcement. *Corrosion and Corrosion Protection of Steel in Concrete,* Academic Press, 1994, pp. 171-185.

Rodriguez, J., L.M. Ortega & J. Casal. 1994b. Corrosion of Reinforcing Bars and Service Life of Reinforced Concrete Structures. International Conference on "Concrete across Borders", Odense, Denmark Vol. II, 1994, pp. 315-326.

Rodriguez, J., L.M. Ortega, J. Casal & J.M. Diez. 1996. Assessing Structural Conditions of Concrete Structures with Corroded Reinforcement. International Congress on "Concrete in Service Mankind", Dundee, UK. 1996.

Söylev, T.A, & R. Francois. 2005. Corrosion of Reinforcement in Relation to Presence of Defects at the Interface between Steel and Concrete. *Journal of Materials in Civil Engineering*. Vol. 17, pp. 447-455.

Thoft-Christensen, P. 1998. Assessment of the Reliability Profiles for Concrete Bridges. *Engineering Structures*, Vol. 20, pp. 1004-1009.

Thoft-Christensen, P. 2000. Modelling of the Deterioration of Reinforced Concrete Structures. *Proc. IFIP WG7.5 Conference on "Optimization and Reliability of Structural Systems", Ann Arbor, MI, USA*, September 25-27, pp. 15-26.

Thoft-Christensen, P., S. Svensson & H.L. Frandsen. 2005. 3D-modelling of Corrosion Crack Opening. Proc. IFIP WG 7.5 Conference, Aalborg, Denmark, May 22-25.

Thoft-Christensen, P., H.L. Frandsen & S. Svensson. 2007. Numerical Study of Corrosion Crack Opening. Accepted for publication in *Structures and Infrastructure*, 2007.

Toongoenthong, K. & K. Maekawa. 2005. Multi-Mechanical Approach to Structural Performance Assessment of Corroded RC Members in Shear. *Journal of Advanced Concrete Technology*. Vol. 3, February 2005, pp. 107-122.