Laboratory Studies and Calculations on the Influence of Crack Width on Chloride-Induced Corrosion of Steel in Concrete

by Peter Schießl and Michael Raupach

To clarify the corrosion mechanism and the dominant influencing variables, especially the influence of crack width, laboratory tests were performed on cracked reinforced concrete beams. Test results and a mathematical model[1] were then used to calculate the effect of crack distance and the effect of a crack width limitation by reducing the rod diameters on the steel removal rates due to chloride-induced corrosion.

The results show that after local depassivation of the steel surface by chlorides penetrating through cracks in concrete the steel in the cracked zone acts as an anode (iron removal) and the steel between the cracks acts as a cathode (oxygen reduction). Therefore the corrosion rate in the crack zone is influenced considerably by the conditions between the crack. It has been found that thickness and quality of concrete cover influence the corrosion rate much more than the crack width. By simplified calculations it was shown that a crack width limitation by reducing rod diameters from about 0.4 mm (0.016 in) to lower crack widths results in increasing losses of steel diameter. As a consequence, corrosion protection must be assured primarily through adequate concrete quality and cover.

Keywords: chlorides; concretes; corrosi on; cover; cracking (fracturing); crack width and spacing; humidity; reinforced concrete; reinforcing steels; water-cement ratio.

RESEARCH SIGNIFICANCE

Corrosion of steel in concrete has caused serious damage to concrete structures all over the world. To prevent such corrosion problems by adequate construc tion of new structures and to repair corroding structures durably, corrosion mechanisms and influencing factors have to be clearly understood. Although there are numerous recent status reports and studies describing reinforcement corrosion in crack zones,[2-4] only a few results are available from practice-oriented tests describing the dominant mechanism for the case of chloride-induced reinforcement corrosion in the crack zone.[5] This study has been carried out to improve the incomplete knowledge of the corrosion process in the area of a crack in concrete , i.e., the corrosion mechanism, the influencing factors, and espe cially the influence of a crack width limitation.

ELECTROCHEMICAL PRINCIPLES

The electrochemical principles of steel corrosion in concrete have been described in detail.[6] The high alkalinity of the pore solution in concrete fundamentally provides durable protection for steel in concrete. A passive layer of ferrous oxides is formed on the steel surface, which protects the steel against iron dissolution.
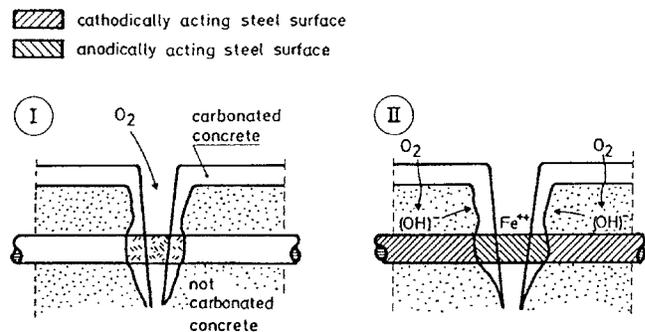

Fig. 1—Schematic representation of corrosion process.

This protection can be destro yed only by carbonation of the concrete or if a critical chloride content at the steel surface is exceeded (depassivation), allowing the reinforcement to corrode provided that sufficient moisture and oxygen are available at the steel surface.

The resulting corrosion is an electrochemical process, taking place in two substeps, as in a battery (Fig. 1):

- The anodic subreaction occu rs at the actual corroding pits, where ions of iron en ter into solution, releasing two electrons per ion.
- The cathodic subreaction is not harmful to the steel. It usually occurs alongside the anodically acting re gions of the steel surface, where the free electrons react with water and oxygen, forming hydroxil ions.

These hydroxil ions in turn re act with the ions of iron in solution, forming the corrosion products, which are generall y deposited near the anode.

In reinforced steel structur es, anode and cathode may be microscopically adjacent or, especially in cases of chloride-induced corrosion, up to a fewmeters apart (Fig. 2). Whereas the anodic reaction is confined to steel surface zones where

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the critical chloride content causing depassivation is exceeded, the cathodic reaction may occur in all areas that are not permanently water-saturated.

Two different corrosion mechanisms are theoretically possible for steel corrosion in the region of cracks:

- In Mechanism 1, the anodic and cathodic subprocesses take place in the crack zone. Anodes and cathodes are extremely small and located closely side by side (microcell corrosion). Oxygen is mainly supplied to the cathodically acting surface zones through the crack.

- In Mechanism 2 the reinforcement in the crack zone acts mainly as an anode, the passive steel surface between the cracks forming the cathode. In this case, oxygen transport to the cathode is dominantly via the uncracked area of the concrete (macrocell corrosion). Much higher corrosion rates are to be expected than in Mechanism 1, since the steel surface involved in the cathodic subprocess is much larger.

The laboratory tests described below were carried out particularly to determine whether and to what extent the steel between the cracks acts cathodically during reinforcement corrosion in the crack zone.

LABORATORY TESTS
Test setup

To be able to generate cracks with defined cracks widths, reinforced concrete beams ($70 \times 15 \times 9.7$ cm^3 [$27.6 \times 5.9 \times 3.8$ in^3]) were clamped against steel girders (Fig. 3). As a further measure to insure unambiguous test conditions, only a single crack was generated in each beam on the top side in the center field.

For the electrical cell's current measurements, it is necessary to interrupt the steel reinforcement at the points where the electric current is to be measured. A low-resistance ammeter can then be used to determine the current between the reinforcing steel zones via cables leading to an external measuring point.[1] In the present tests, the reinforcement in the immediate vicinity of the crack, which can be depassivated by the action of the chlorides, therefore had been separated from the neighboring zones, which can act only as a cathode, to observe any macrocell formation. The depassivated steel surface is, however, very small and reaches about 1 cm inside both crack flanks.

It is not possible to induce a crack through the surface area of a roughly 2-cm-(0.79-in.)-long section of reinforcing steel, since the required anchoring length is substantially greater. This problem was solved by using a reinforcing steel that has been coated with an epoxy resin by a coating company except for a 2 cm central section. This allows the crack to be induced through the surface area of the uncoated section, while the coated sections are used to apply the force into the steel bar.

The potentially cathodic reinforcement between the cracks was simulated by adding three reinforcing steel sections on
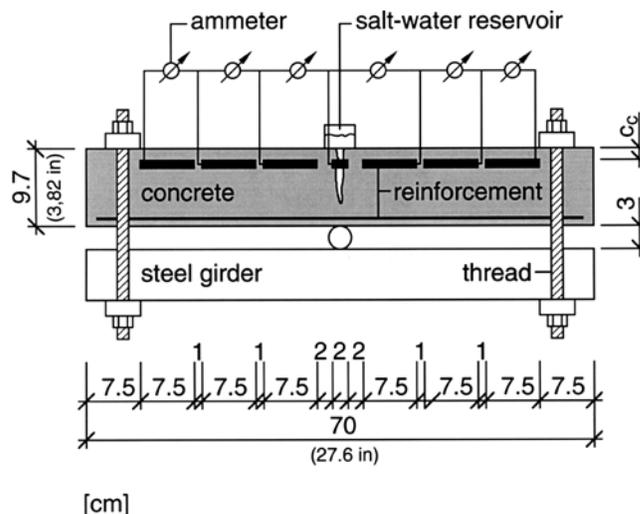

Fig. 2—Schematic representation of micro and macrocell corrosion.

each side of the crack, allowing cathodic action to be determined as a function of crack distance (Fig. 3). The rod diameter was 14 mm (0.55 in.) in each case.

The side faces and undersides of the beams were coated with an epoxy resin-based coating to prevent most of the effects of rapid drying of the beams via these areas; this simulates the situation in a large component more closely.

Concrete composition

The concrete in the tests is composed of 300 kg/m^3 (500 lb/yd^3) OPC 35 F with a *w/c* ratio of 0.6 and an AB aggregate grading curve according to DIN 1045. This concrete has Strength Class B 35 and standard consistency according to DIN 1045.

PERFORMANCE OF TESTS
Fabrication and storage of specimens

The beams were cast in steel shuttering, removed after 1 day, and stored for a total of 2 days in a humid room. Three days after concreting, each of the beams was clamped against the steel girders and cracks of the desired width were induced. The required central position of the crack was obtained by inserting a 0.5 mm (0.02 in.) thick, 7 mm (0.28 in.) deep plastic strip in the surface of the green concrete as a crack initiator.

The specimens were stored in a climatic chamber at a temperature of 20 ± 1 C and 80 ± 5 percent relative atmospheric humidity.

Wetting

The tests described here relate to chloride-induced corrosion and not to corrosion caused by carbonation of the concrete.

To establish clear conditions and prevent depassivation of the reinforcing bars designed as cathodes, chloride wetting was confined to 2-cm-(0.79-in.)-wide wetting frames placed on the top of the beam in the crack zone (Fig. 3). In the tests described here, wetting was started 28 days after casting of the specimens.

On the basis of results from studies[7] in the splash zone of a motorway near Düsseldorf during the harsh winter of 1986-87, a 1 percent chloride solution was poured into the wetting frame above the crack once weekly for a period of 24 hr. Twelve wetting periods were followed by two periods in which tap water without added chlorides was introduced.
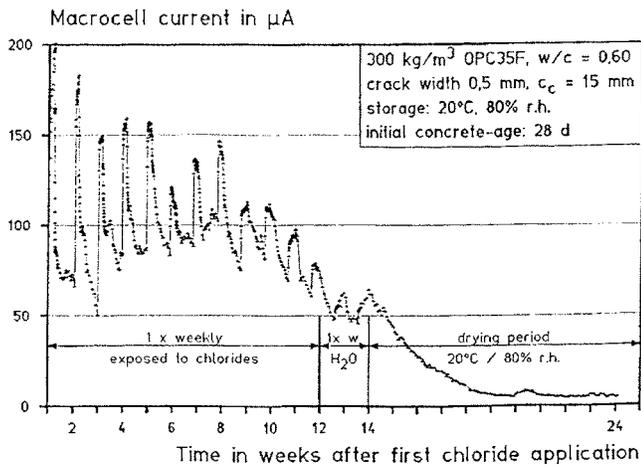
Fig. 3—Test setup and measuring points for laboratory tests on cracked reinforced concrete beams.
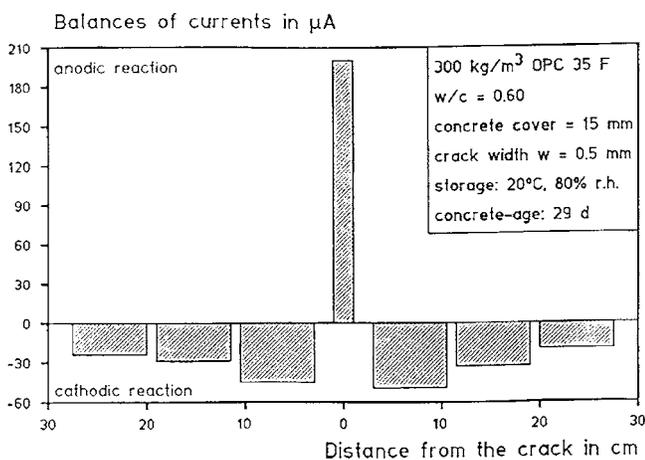

Fig. 4—Measured time curves for cell currents between steel in crack zone and steel sections outside crack.

The specimens were then stor ed without further wetting at 80 percent relative atmospheric humidity in a climatic chamber; after 1 year the cycle of 12 wetting periods was repeated to simulate real conditions as closely as possible.

Cell current measurement

The cell currents were measur ed under computer control using a data logging system developed at the Institute for Building Materials Research (ibac) at the Technical University of Aachen, Germany.[1]

The electric current values can be converted directly into corrosion rates: a constant current of $100\,\mu A$ is equivalent to an annual iron mass of 911 mg (0.002 lb) transformed into rust. Assuming that the entire steel surface of 8.80 cm ^{2} (1.36 in.^{2}) in the crack zone is corroded, the steel removal rate will be 132 $\mu m/a$. Since it is unlikely that the entire steel surface in the crack zone will corrode and since removal of the depassivated steel surface is not uniform, the actual local corrosion rates are, however, considerably higher.

TEST RESULTS

General

Figure 4 shows the time curves of the measured cell currents between the steel in the crack zone and the six steel sections outside the crack for the basic mix and a crack

width of 0.5 mm (0.002 in.). Anextremely high cell current of some 200 μA is observed immediately after the first chloride wetting, corresponding to a local steel removal rate in the crack zone of more than $250\,\mu m$ per year. When the salt solution is removed from the wetting frame on the next day, however, the corrosion   rate declines significantl y. The 11 successive chloride wettings that follow are also characterized by current maxima during the wetting phase and a subsequent decrease in cell current. There are probab ly several reasons for the fact that the current peaks decrease over time, although the chloride concentration in the crack zone increases with each wetting:

- The permeability of the concrete diminishes with greater concrete age, increa sing the electrolytic resis-tance and inhibiting all ion transport processes.
- Chloride binding is progressive over time.
- The formation of corrosion products in the crack zone obstructs material transport to and from the corrosion pits.

It is not possible at present to judge which of these causes is finally decisive. It may be suspected that all three play a significant role.[1]

Wetting with tap water containing no added chloride in the 13th and 14th weeks after initial chloride wetting produces a cell current curve similar to that in the preceding periods. After wetting ends, the concrete in the crack zone dries and cell currents fall to values below 10 μA during the succeeding 10 weeks. This behavior docu-ments the known decisive influence of water content on the corrosion rate of the reinforcement.

In some specimens, macrocorrosion initiated only after a number of wetting cycles or did not occur at all.

Corrosion mechanism

By measuring the electric cu rrent between the reinforcing steel electrodes (Fig. 3) it is possible to determine for each region of the reinforcing steel whether more electrons have been received than sacrificed (cathode) or vice versa (anode).

Figure 5 shows the current ba lance of each of the rein-forcing steel sections for the basic mix on the first day after chloride wetting with a crack width of 0.5 mm (0.02 in.). To show the effect of the distance between the reinforcement and the crack, the widths of the bars correspond to the lengths of the reinforcing steel sections.

It is apparent that regions of the reinforcement outside the crack behave cathodically up to a crack distance of more than 20 cm (8 in.). The curr ent density of the cathodes falls with increasing distance from the crack because electrolytic resistance rises with crack distance.

The evaluation of the current balances for the specimens aged up to 4 months and for   the other concrete mixes and covers investigated in the testsshowed that the current distri-bution deviates only slightly fromthat depicted in Fig. 5. The conductivity of the concrete is accordingly high enough to allow formation of quite large cathodes at the relative atmo-spheric humidity of 80 percent used in these investigat ions. In general, it may be concluded from the results of cell current measurements   that Corrosion Mechanism 2 (macrocell formation) is usually to be ex pected the dominant corrosion mechanism in crack zones.

Influence of concrete cover on corrosion rate

Figure 6 plots the total mass of steel removed during the 24-week test period, as calculated from the measured cell

current curves[1] against the tested concrete covers, *w/c* ratios, and crack widths. Fig. 7 shows the total mass losses after a period of two years, i.e., after a total of 2 * 14 chloride or water-wetting periods. As expected, it is evident that mass losses increase substantially, whereas mass losses from specimens depassivated during the first wetting period are roughly twice as high after 2 years as after 24 weeks; particularly high rates for the mass losses are observed in specimens not depassivated until the second year. Only one specimen with a concrete cover of 35 mm (1.4 in.), *w/c* = 0.5, and a crack width of 0.1 mm (0.004 in.) showed no corrosion throughout the whole test period.

Fig. 6 and 7 show that increasing concrete cover from 15 to 35 mm (0.6 to 1.4 in.) leads to much-reduced removal rates. This relationship is already known from other research work.[1-3]

Influence of water-cement ratio

A reduction in the *w/c* ratio from 0.6 to 0.5 yielded a further reduction in steel mass loss in the crack zone (Fig. 5 and 6). This influence is especially pronounced after 24 weeks, while the influence of the *w/c* ratio becomes much smaller after 1 year.

This relationship may be explained by the fact that the period up to depassivation is prolonged by a reduction of the *w/c* ratio, but after the onset of corrosion the *w/c* ratio has no significant influence. This result applies, however, only to the tested *w/c* ratios between 0.5 and 0.6. With larger differences (e.g., between 0.4 and 0.7), a significant influence is to be anticipated even after the onset of corrosion; this factor was not investigated in the present study.

Influence of crack width

The results in Fig. 6 and 7 indicate that corrosion currents usually increase with growing crack width given the selected test conditions and observation periods, but it is also evident that concrete cover and composition have a much greater influence than crack width.

In general, the influence of crack width declined with rising test duration, as illustrated by the specimen with a concrete cover of 35 mm (1.4 in.) and a *w/c* ratio of 0.6. Whereas mass loss was still increasing with crack width after 24 weeks, no further systematic crack width influence was detectable after 2 years. This result is again due to the fact that time up to depassivation increases as crack widths fall; on the other hand, there is no significant relationship between crack width and corrosion rate after depassivation for the conditions of the present study.

CALCULATIONS OF CORROSION RATE OF REINFORCEMENT IN CRACK ZONE THROUGH FORMATION OF MACROCELLS

As clearly demonstrated by the cell current measurements described previously, macrocorrosion cells with local anodes in crack zones and large cathodes between the cracks, effective up to several decimeters from the crack, occur in chloride-induced reinforcement corrosion in the crack zone. The corrosion rate of such macrocorrosion cells can be calculated on the basis of simplifying assumptions.[1]

Fig. 8 shows the simplified corrosion model of a macrocell consisting of one anode and different cathode elements. The electrical currents between anode and the cathode elements cause voltage drops *IR* within the concrete and polarizations ΔU at the surfaces of the cathode elements. The cathodic polarization curve of a passive steel in concrete shown in the
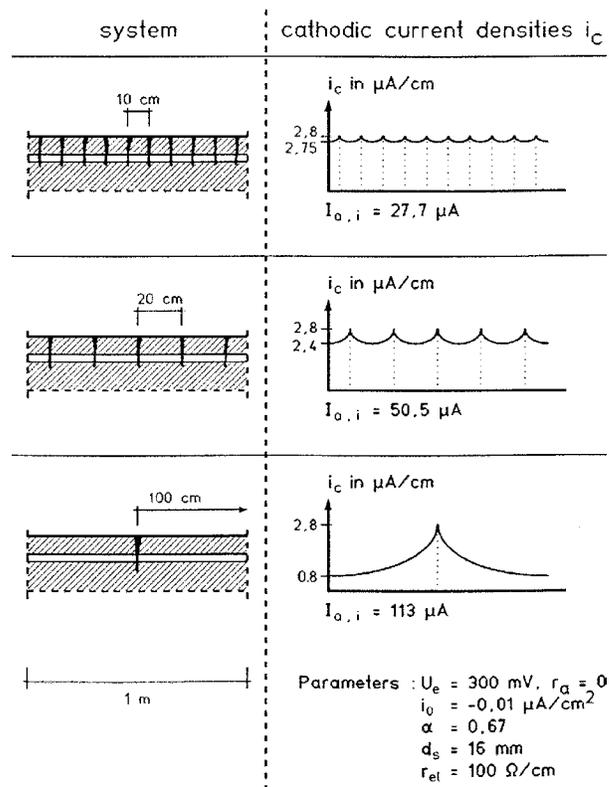

Fig. 5—*Current balances of individual reinforcing steel sections during initial chloride wetting.*
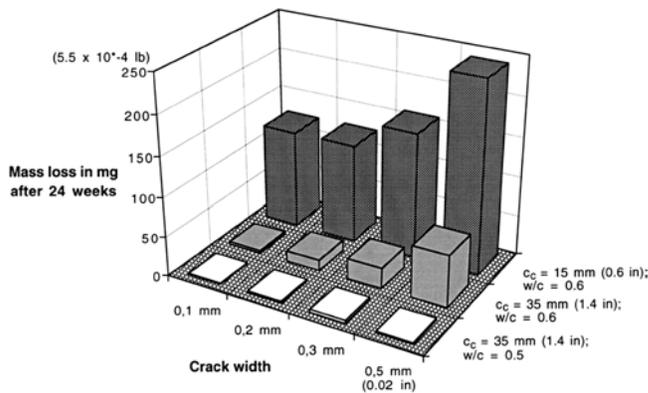

Fig. 6—*Calculated losses in mass of steel in crack zone due to macrocell corrosion after test period of 24 weeks (cf. Fig. 3).*
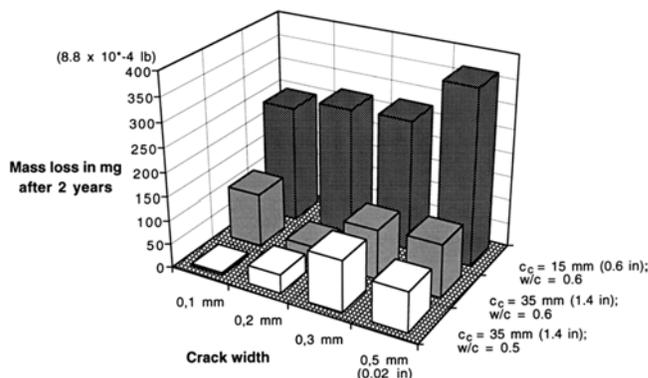

Fig. 7—*Calculated losses in mass of steel in crack zone due to macrocell corrosion after test period of 2 years (cf. Fig. 6).*

Table 1—Results of comparative calculations on effect of crack width limitation by restricting rod diameters on loss of cross section of steel in crack zone

Parameters	Dimensions	Case 1	Case 2	Case 2/Case 1
Reinforcement	mm (in.)	2 Ø 20 (0.79)	12 Ø 8 (0.31)	—
Cross section of steel	cm^2 ($in.^2$)	6.28 (0.97)	6.03 (0.93)	0.96
Depassivated steel surface area	cm^2 ($in.^2$)	25.1 (3.9)	24.1 (3.7)	0.96
Total perimeter of steel bars	cm (in.)	12.6 (5.0)	30.1 (11.9)	2.4
Distance from crack to crack	cm (in.)	16.2 (6.4)	9.5 (3.7)	0.6
Total macrocell current	µA	78	135	1.7
Mean anodic current density	$µA/cm^2$ ($µA/in.^2$)	3.11 (20.1)	5.60 (36.1)	1.8
Mean rate of steel removal	µm/a	36.1	65.0	1.8
Mean annual loss in cross section of steel bars	percent/a	0.36	1.63	4.5

Evans diagram (Fig. 8) has been determined at 60 specimens with different concrete mixes. [1] As the anodic polarization depends on various factors, it has been assumed that the anodes are not polarizable ($U_{R,a} = U_{C,a}$) or that the corrosion potential at the anode $U_{C,a}$ is known. The concrete resistance r_{el} is related to the distance between the steel elements.

The following section will first discuss the influence of crack distance on the reinforcement corrosion rate in the crack zone, [1] going on to calculate and explain the effects of a crack width limitation by reducing the steel diameter.

Influence of crack distance

Fig. 5 presents calculated results indicating the effect of crack distance on current distribution and steel corrosion rate in the crack zone. [1] The following assumptions are made:

- Unidimensional electrical field.
- The crack distance is assumed to be the maximum cathode size for each anode; i.e., overlapping electrical fields for several anodes are not taken into account.
- Steel diameter: 16 mm (0.63 in.).
- Voltage between active anode and cathode: 300 mV.
- Polarization of the anode is negligible, i.e., for example, high chloride and moisture contents.
- Electrical resistance of the concrete per unit length: 100 Ω/cm.
- The polarization behavior of the cathode was described by α = 0.67 and i_o = 0.01 $µA/cm^2$; i.e., there is no oxygen deficiency at the cathode, entailing concrete that is not permanently water-saturated. [1]

Figure 5 shows that the cathodic currents decrease significantly with growing crack distance, agreeing with the result of laboratory tests presented in Fig. 5.

Since the cathodically acting regions of the steel surface may extend to a distance of several decimeters from the crack, the potential corrosion rate is reduced where crack distances are small. It is, for example, apparent from Fig. 5 that raising the crack distance from 10 to 20 cm (3.9 to 7.9 in.) results in an approximate doubling of the steel corrosion rate in the crack zone, whereas an additional fivefold rise in crack distance from 20 cm to 1 m (7.9 to 39 in.) leads only to a further doubling of the corrosion rate. These calculations indicate that a significant reduction in potential corrosion rates occurs when a number of anodes are close to one another.

Influence of crack width limitation through reduction of rod diameter

It is known that crack widths can be reduced by limiting the rod diameter. The effects of such a diameter limitation on the steel corrosion rate in the crack zone are calculated below.

It is assumed that the number of reinforcement rods is selected to insure that the overall steel cross section remains constant if it is likewise assumed that the length of depassivated steel in the crack zone increases in direct proportion to the steel diameter in accordance to the length showing reduced bond. For these assumptions, the depassivated surface of a steel in the crack zone will be four times the cross section of the steel. Given a constant overall steel cross section, it follows that the total depassivated area will remain the same irrespective of rod diameter.

In terms of the cathodically acting regions of the steel surface, a reduction in rod diameter has two main effects:

- The total surface area of a single steel bar between the cracks increases in inverse proportion to the ratio of the diameter.
- The crack distance decreases less than proportionately to the ratio of the diameters. [8]

Whereas increasing the total surface area of the steel fundamentally tends to accelerate the reinforcement corrosion rate in the crack zone, reduction of the crack distance has the opposite effect, as already indicated. However, since the steel diameter has a greater influence on the total surface of the steel than on the crack distance, a reduction in rod diameter will generally lead to higher corrosion rates in the crack zone, as shown below on the basis of a comparative calculation. This applies only to a case in which the influence of the cathodic reaction is not negligibly small. However, if the chloride content is above the critical limit value in the crack zone, the cathodic reaction always plays a significant role. [1]

The following calculation was made to quantify the effect of a diameter reduction:

In a flexurally stressed beam with a width of 40 cm (15.7 in.) and an effective height related to crack width limitation of 8.75 cm (3.4 in.), a reinforcement cross section of 6 cm^2 (0.93 $in.^2$) is required. [8] In the calculation, the expected corrosion rates for a reinforcement configuration with 2 Ø 20 mm (0.79 in.) (Case 1) and 12 Ø 8 mm (0.31 in.) rods (Case 2) are compared. All other assumptions correspond to those described previously.

Table 1 summarizes the various parameters and the calculated results for the two cases. As already noted, given the previous assumptions the depassivated area in the crack zone amounts to four times the steel cross section.

In Case 2, the total steel surface area increases in inverse proportion to the ratio of the diameters, i.e., by a factor of 20/8 = 2.5. According to Eq. (5), [8] the mean crack distances are 16.2 cm (6.4 in.) (Case 1) or 9.5 cm (3.7 in.) (Case 2), respectively. It may be inferred from the influences of the steel surface area and crack distance that the total corrosion rate in Case 2 will be 1.7 times higher than in Case 1. The mean removal rates can be

determined from the mean current densities,[5] which rise by a factor of 1.8 due to the reduction in steel diameter.

However, it is not the removal rate but the loss in cross section that is decisive for assessment of the bearing capacity of reinforced concrete structures. The loss in cross section is obtained by dividing the mean removal rates by the radius of the rod. In the present example, a reduction in rod diameter from 20 to 8 mm (0.79 to 0.31 in.) resulted in an increase in annual losses in cross section by a factor of $1.8 \times 20/8 = 4.5$.

The same calculation was performed under the assumption that only every second or fourth crack is depassivated. Results showed that the losses in cross section rise by a factor of 5.75 if each second crack is depassivated or by a factor of 7.25 if each fourth crack is depassivated. This result can be explained by the fact that the larger total surface of the steel has a stronger effect with small diameters if the distance between the depassivated cracks increases.

In general, it may be stated that the losses in cross section of the steel are greatly increased by reducing the rod diameter. The factor for increased loss of cross section is roughly 1 to 3 times the reciprocal of the ratio of rod diameters.

CONCLUSIONS

Cell current measurements on cracked reinforced concrete specimens showed that chloride-induced steel corrosion in the crack zone involves the formation of macrocorrosion cells. The steel in the crack zone acts as the anode and the steel between the cracks, up to a distance of several decimeters from the cracks, as the cathode. The steel corrosion rate in the crack zone is therefore influenced considerably by the conditions between the cracks.

It was also established that the steel removal rates in the crack zone fall significantly with increasing concrete cover and are slightly lower for a concrete with a w/c of 0.5 than for one with a w/c of 0.6 under the testing conditions described previously. The influence of crack width declined significantly with test duration until after a period of 2 years no clear relationship between crack width and steel removal rates in the crack zone was observable.

Besides these parameters the water content of the concrete influences the corrosion rate. During water applications in the cracked area the highest corrosion rates have been detected. Without direct water application the corrosion rates declined significantly. To quantify the influence of the ambient humidity and the resulting water content of the concrete additional research studies are being carried out.[1]

Using the method of calculation already described in the literature,[1] it was shown that the corrosion rate in the crack zone also decreases with falling crack distance because the cathodically acting steel surface area for each crack is reduced. Finally, it was shown that limiting rod diameters to restrict the crack widths results in higher losses in cross section despite the reduced crack distance and crack width. Calculations indicated that the increase is usually about 1 to 3 times the reciprocal of the ratio between the rod diameters. This calculation and the results of the laboratory tests clearly indicate that the problem of reinforcement corrosion in crack zones cannot solely be solved by crack width limitation in the range from roughly 0.3 to 0.5 mm; corrosion protection must be assured primarily through adequate concrete quality and cover.

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