

## **DURABLE CONCRETE STRUCTURES: CRACKS & CORROSION AND CORROSION & CRACKS**

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**Abstract:** Cracking of the concrete cover may adversely affect the durability of reinforced concrete, such as by promoting reinforcing steel corrosion. On the other hand, the process of reinforcing steel corrosion may lead to concrete cracking itself. This contribution summarizes the role of concrete cracks in triggering reinforcing steel corrosion as well as in influencing the rate of corrosion during the so-called corrosion propagation stage. It is concluded that the risk introduced by concrete cracks is often overrated. In a second part, the mechanism of concrete cracking arising from reinforcing steel corrosion is addressed. The processes leading to this are complex and are currently not yet well understood. However, both numerical modeling and practical experience show that accelerated laboratory testing, which often serves as basis for the calibration of crack prediction models, is not representative for practical conditions. Finally, implications for engineering are discussed, in particular concerning the inspection of structures, where concrete surface cracks are considered as the main indicator for internally ongoing corrosion.

### **1 INTRODUCTION**

The relation between concrete cracks and reinforcing steel corrosion is a twofold issue: Concrete cracks may be both the cause and the result of corrosion of the reinforcing steel.

On the one hand, concrete cracks are generally perceived to present an increased risk for corrosion of the reinforcing steel embedded in the concrete. Thus, significant efforts are in standards made to control cracking and limit crack widths, although this has long been subject to controversial discussions [1]. Concrete cracks may arise from a wide number of different mechanisms, including structural loads, restricted volume reductions of the concrete (shrinkage, temperature-induced volume changes, etc.), and expansive stresses such as arising from chemical attack (alkali-silica reaction) or

freezing damage [2]. If these cracks create a direct path between exposure environment and reinforcing steel, they may influence the corrosion process. In a first part of this contribution, lessons learned from various literature sources investigating both corrosion initiation and corrosion propagation in cracked concrete will be presented and discussed.

On the other hand, corrosion of reinforcing steel in concrete may also start in the absence of cracks. This is because both chloride and carbonation may penetrate through the uncracked concrete cover and, once reaching the steel, corrosion may be triggered. In that case, the corrosion process and the related precipitation of rust can lead to expansive stresses, which may lead to concrete cracking. Thus, in certain cases, initially uncracked concrete starts cracking as a result of internally

ongoing corrosion.

Fig. 1 summarizes these two cases schematically. Relevant questions, which are still not completely understood, are the following:

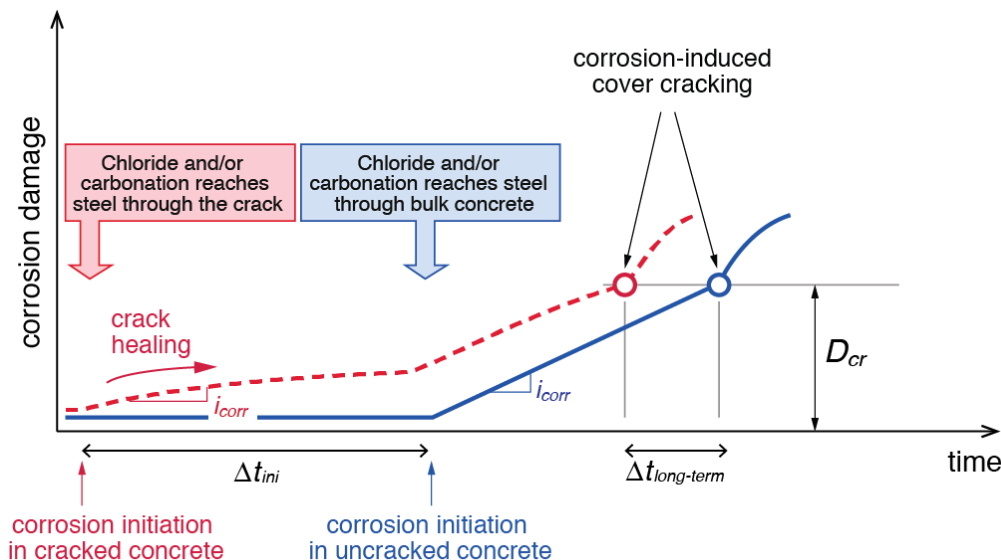
- To what extent ( $\Delta t_{ini}$ ) do cracks in concrete shorten the time until steel corrosion starts?
- To what extent do cracks in concrete influence the rate of corrosion ( $i_{corr}$ ) once corrosion has started?
- To what extent can crack self-healing processes slow down the rate of corrosion in cracked concrete?
- How much corrosion damage at the steel ( $D_{cr}$ ) is tolerable until precipitation of corrosion products in the pore system leads to cracking of the concrete cover (cracking as a result of the corrosion process)?
- To what extent, considering all the above aspects, can cracks shorten the useful life of a structure on the long-term ( $\Delta t_{long-term}$ )?

Ultimately, these points will determine the time at which the acceptable degree of damage (limit state) will be reached. These aspects are discussed in the following sections in detail, both for corrosion due to carbonation and due to chlorides.

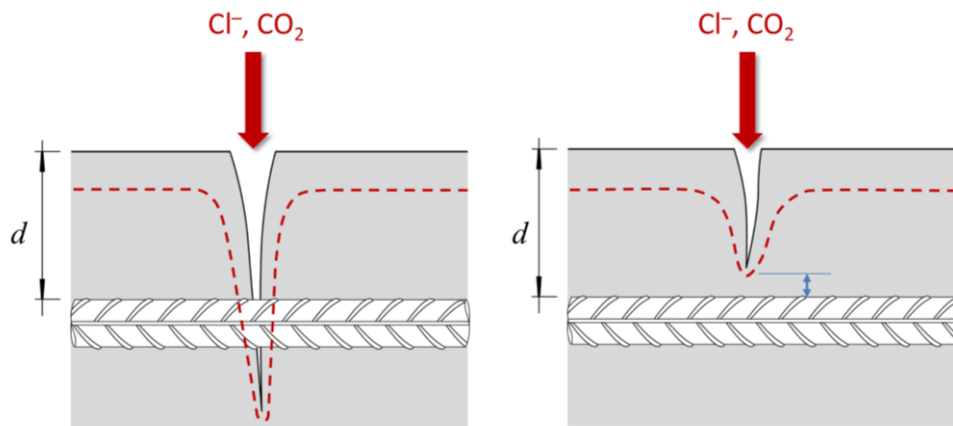
## 2 CORROSION IN CRACKED CONCRETE

### 2.1 Corrosion initiation in cracked concrete

Corrosion of reinforcing steel can be triggered by ingress of chlorides or by carbonation of the concrete. Cracks have been found to locally promote both chloride ingress as well as carbonation [3-5], resulting in much faster ingress of chlorides and the carbonation front in the concrete surrounding the crack compared to the bulk concrete (Fig. 2). As a result, it is not surprising that corrosion tends to start earlier in those zones where cracks meet the reinforcing steel than in zones further away from cracks. Numerous studies have experimentally confirmed a reduction in time to corrosion initiation in cracked concrete compared to uncracked concrete [6-8]. However, there is no agreement on the time by which cracks shorten the time until corrosion initiation ( $\Delta t_{ini}$  in Fig. 1). In some cases, corrosion was in cracked concrete found to start almost immediately upon chloride exposure, while in other studies, it took more time for corrosion to start. Some studies have also indicated that corrosion may



**Figure 1:** Schematic illustration of corrosion in both initially cracked (red, dashed) and uncracked (blue, solid) concrete. In cracked concrete, corrosion generally starts earlier than in uncracked concrete ( $\Delta t_{ini}$ ), but “crack-healing effects” may slow down the corrosion process (rate of corrosion  $i_{corr}$ ) and over the long-term lead to comparable times until the accumulated corrosion damage ( $D_{cr}$ ), in terms of corrosion products precipitated in the pore system of the concrete, lead to cracking of the concrete cover (which would accelerate the deterioration process).



**Figure 2:** Schematic illustration of how cracks locally accelerate the ingress of chlorides or the carbonation front. The red dashed line represents the “corrosive front”, that is, the depth to which a critical chloride concentration or the carbonation has penetrated the concrete. For cracks intersecting the entire cover depth (left), the accelerating effect is more pronounced than for cracks that do not go through the entire cover depth. Nevertheless, in both cases the “corrosive front” reaches the reinforcing steel faster than through the bulk concrete cover.

preferentially start at the end of slip and separation zones typically present in cracked concrete rather than at the primary crack itself [9]. This was explained by a mechanism similar to crevice corrosion. Other researchers found that corrosion only occurred in separation zones of comparatively large thickness [10, 11].

## 2.2 Corrosion propagation in cracked concrete

A recent literature review [12] has revealed that cracks may, in some cases, accelerate the rate of the corrosion process compared to uncracked concrete. It may be noted, however, that this observation was typically made in short-term laboratory studies (a few weeks to a few years at maximum). When cracked concrete samples were exposed for longer times, it was found that effects that are generally summarized with the term “crack healing”, may lead to a decrease in the rate of corrosion over time (Fig. 1). Crack healing could arise from a number of different mechanisms such as deposition of corrosion products (rust) or other precipitates in the crack (e.g. calcites), resulting in crack clogging and thus with time leading to a situation resembling corrosion in uncracked concrete. It may be due to these effects that the few long-term studies available in the

literature [13-15] tend to indicate no pronounced accelerating effects of cracks on the corrosion rate in cracked concrete.

From a mechanistic point of view, Schiessl et al. [16] suggested that rebar corrosion in cracked concrete involves the formation of a galvanic element, where the anodic (iron dissolution) reaction occurs at the location where the crack meets the reinforcing steel, whereas the cathodic reactions (oxygen reduction) occurs on the passive steel surfaces between the cracks. This requires supply of oxygen and moisture at the passive steel, which means transport through the concrete cover between the cracks. Thus, the kinetics of corrosion in cracked concrete are controlled by the oxygen and moisture transport in the uncracked cover concrete, and thus the corrosion rate may be similar to corrosion in uncracked concrete.

In the literature review [12] it was concluded that, based on the current state-of-the-art knowledge, it is difficult to quantify to which extent to which concrete cracks enhance the corrosion rate ( $i_{\text{corr}}$ ) on the long-term. However, the effect seems to be moderate.

## 2.3 Critical crack widths in engineering

For engineering practice, it is desirable to define “critical crack widths”, that is, crack widths below which no corrosion would occur.

While such crack widths may be relevant for deterioration mechanisms such as frost attack or chemical attack (sulphate attack, alkali-aggregate reactions), critical crack widths for corrosion have been subject to controversy for decades [1]. For instance, it has been criticized that the crack width, measured at the concrete surface, may be considerably different from the crack width at the location of the steel, especially in bending cracks with non-parallel crack walls).

In the extensive literature review in [12], it was found that in those studies, in which critical crack widths were proposed, the critical widths scattered considerably (typically between 0.1 and 0.5 mm). On the other hand, there were also a number of studies concluding that critical crack widths cannot be recommended because such a threshold crack width could not be identified, e.g. [17].

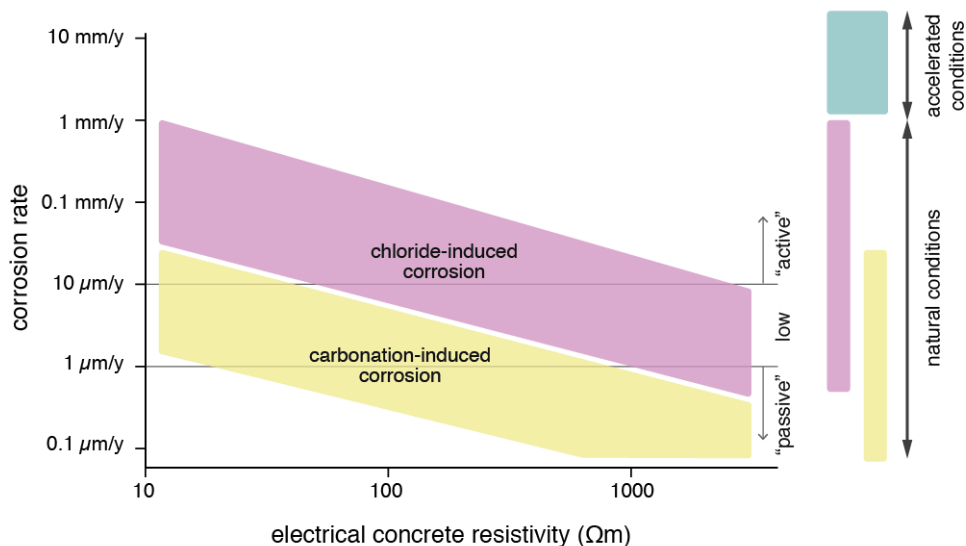
Generally, one may conclude that even very thin cracks are zones where transport of chlorides or the carbonation front is accelerated compared to the bulk concrete, and thus, corrosion even at very thin cracks may start considerably earlier than in uncracked

concrete. However, once corrosion starts, the crack width seems to be a minor influencing factor on the corrosion propagation, since the further kinetics of the corrosion process depend on a number of properties related to the zones between the cracks (cover depth, concrete transport properties, electrical resistivity, etc.) as well as on the exposure conditions (moisture state, possible crack healing effects, etc.). Thus, critical crack widths may more be related to the ability of the crack to self-heal over time. In this regard, critical widths (for self-healing) of the order of 0.4–0.5 mm were suggested for marine exposure in [6, 18]. For road-salt or other exposure environments (carbonation), however, these values may differ. The current literature does not allow to make any recommendations.

### 3 CORROSION IN UNCRACKED CONCRETE

#### 3.1 Corrosion initiation in uncracked concrete

The pH of the liquid phase in sound concrete is typically very high (>13). In this



**Figure 3:** Common representation of corrosion rate as a function of electrical concrete resistivity. The latter is primarily dependent on concrete properties (porosity, pore solution chemistry) and the exposure conditions (moisture state). Both corrosion rate and electrical concrete resistivity are variable over time. The purple (chloride-induced corrosion) and yellow (carbonation-induced corrosion) areas indicate typical values in practical conditions. In chloride-induced corrosion, the corrosion attack is much more local than in carbonation-induced corrosion and the local rates of corrosion attack can be orders of magnitude higher than in the case of carbonation-induced corrosion. The blue area indicates that laboratory tests to simulate corrosion-induced concrete cracking are generally strongly accelerated compared to natural conditions.

condition, the steel is covered with a passive film (a thin film of oxides) that reduces the rate of iron loss to a technically negligible level. However, concrete is a porous and thus permeable material. When exposed to chloride-bearing environments (road salt, seawater) or to carbon dioxide, these species will – in combination with moisture – penetrate the concrete and react with its constituents.

Chlorides may be physically and chemically bound by the cement hydration phases and thus, to some extent, the chlorides are “filtered” by the cover concrete. Nevertheless, at some point, the freely dissolved chlorides will arrive at the steel surface and, if present in sufficient quantity, lead to localized corrosion. This means the highly local loss of passivity and thus the formation of macro-cell corrosion. The amount of chlorides needed to initiate corrosion, the so-called critical chloride content in concrete, scatters widely in the literature [19, 20], as it depends on many influencing parameters.

Carbon dioxide leads to carbonation of the concrete, that is, a reduction in pH of the liquid phase held in the pore system. Due to this loss of alkalinity, the passive film on the reinforcing steel surface is no longer stable, and thus, the steel may corrode typically in a more or less uniform manner.

### 3.2 Corrosion propagation in uncracked concrete

The kinetics of the corrosion process depends on many factors, among them the pore structure of the concrete, the moisture content, the conductivity of the pore liquid, the temperature, and the geometry (cover depth, rebar geometry, etc.). Many of these parameters – with the exception of the geometry – have an influence on the electrical resistivity of the concrete. Thus, it has in the literature been attempted to find relationships between the corrosion rate ( $i_{\text{corr}}$ ) and the electrical resistivity of the matrix [21, 22]. Fig. 3 illustrates common ranges based on numerous studies. While these relationships may hold on a qualitative level, they are,

however, not suitable to quantitatively and reliably predict  $i_{\text{corr}}$ .

It is important to note that in chloride-induced corrosion, the local rate of iron dissolution can be orders of magnitude higher than in carbonation-induced corrosion (Fig. 3). In chloride-induced corrosion local rates of metal loss can be up to 1 mm/y in moist and high chloride (thus low resistive) environments. In carbonated concrete, typical rates of corrosion are of the order of 30  $\mu\text{m}/\text{y}$  at max. Note that these are maximum instantaneous values that may be achieved in situations when the concrete is exposed to liquid water. As this only happens for limited time, the average corrosion rate over time will be considerably lower.

### 3.3 Corrosion causing concrete cracking

In both chloride-induced and carbonation-induced corrosion,  $\text{Fe}^{2+}$  ions are released at the anodic sites on the steel surface. These react with species present in the pore liquid and may form sparingly soluble compounds, typically known as “rust”. Those precipitates are known to exhibit a volume considerably larger than the original iron, such as by a factor of 3–6. Hence, the precipitation of these corrosion products in the pore system of the concrete, gradually filling up the pore space, may lead to expansive stresses, which in turn may lead to cracking of the cover concrete.

In chloride-induced corrosion, the presence of chlorides dramatically increases the solubility of dissolved ferrous ions (such as to  $10^{-1}$  M) [23], which strongly delays the precipitation of corrosion products. Moreover, due to the localized nature of the corrosion attack, corrosion products tend to find larger pore space to fill up, and secondly, the distribution of generated stresses is less critical. Thus, in chloride-induced corrosion, considerable local loss of steel sectional area may occur until the concrete cracks. This is well known from practical experience, as for instance documented in [24]. For chloride-induced corrosion, the relevant limit state in practice is thus often not the concrete cover cracking, but rather the loss of steel sectional

area.

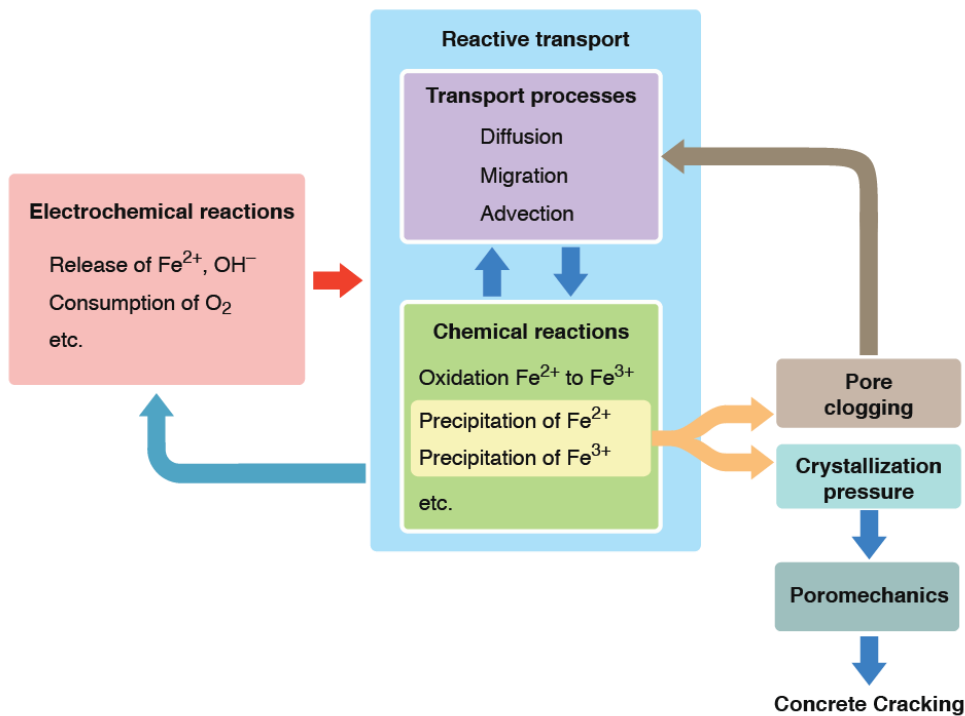
In contrast, in carbonation-induced corrosion, corrosion products are hardly soluble in typical pore solutions of carbonated concrete. As is apparent from a compilation of literature data [25], at a pH of 8.5 the solubility of  $\text{Fe}^{2+}$  is around  $10^{-3}$  M, and the solubility of  $\text{Fe}^{3+}$  is around  $10^{-10}$  M. Thus, virtually all of the iron dissolved in the corrosion process will precipitate in the pore system. Moreover, due to the more uniform nature of corrosion compared to the case of chloride-induced corrosion, the distribution of arising stresses may generate earlier cracking. As a consequence, concrete cracking may occur at comparatively small steel sectional losses (compared with chloride-induced corrosion). The limit state in carbonation-induced corrosion is thus typically not the sectional loss of steel, but rather the cracking of the cover concrete. As a result, the concept in engineering is generally to relate the amount of accumulated sectional loss to the time when the concrete cover cracks ( $D_{\text{cr}}$  in Fig. 1).

Empirical data has revealed that the amount of sectional loss required to lead to concrete

cover cracking ( $D_{\text{cr}}$ ) depends on geometrical factors, including rebar diameter and cover depth, as well as on mechanical properties of the concrete (see literature reviewed in [26]). Typical (empirical) values of sectional loss needed to lead to cracking in carbonated concrete are of the order of 50–100  $\mu\text{m}$ .

### 3.4 Challenges in forecasting concrete cover cracking due to corrosion

The literature proposing numerical models to forecast concrete cover cracking due to corrosion is extensive, see e.g. literature review [27]. The models are based on mathematical expressions of variable complexity, typically considering the corrosion rate ( $i_{\text{corr}}$  in Fig. 1), the cover depth, and the rebar diameter as parameters. Generally, these models were calibrated on the basis of accelerated laboratory tests (rather than natural conditions). In the mentioned review [27], a critical analysis of the different models revealed that generally, the state-of-the-art models are only capable of adequately forecasting the time-to-cracking for those experiments to which they were fitted. None of



**Figure 4:** Interdependent processes involved in corrosion-induced concrete cracking that need to be considered in future predictive models.

the models could, however, predict the time-to-cracking for the data produced with a different experiment.

A major shortcoming of these models is that they are based on accelerated testing (Fig. 3), which typically involves an impressed current to strongly accelerate the anodic reaction, that is, the rate of release of  $\text{Fe}^{2+}$  at the steel surface. A recent study [25] of the different processes that follow this release of  $\text{Fe}^{2+}$  into the cementitious matrix has revealed that under accelerated conditions, precipitation of corrosion products are strongly favoured immediately at the steel surface. This is because the acceleration leads to supersaturation of  $\text{Fe}^{2+}$  in close proximity of the rebar, whereas, at natural corrosion rates,  $\text{Fe}^{2+}$  saturation is hardly reached, which permits the diffusion of  $\text{Fe}^{2+}$  away from the rebar (up to millimetres) before it precipitates. This “relaxation” under natural conditions is the reason why corrosion products can also precipitate relatively far away from the steel. The larger pore volume of the concrete that is in this case filled with corrosion products can thus accommodate more corrosion products before expansive stresses arise, which, ultimately, will lead to cracking.

To be able to forecast concrete cracking due to reinforcing steel corrosion, mechanistic models, considering all relevant processes at the fundamental level, still have to be developed. Fig. 4 shows a schematic illustration of all the processes that need to be considered in future model approaches.

This includes the electrochemical reactions occurring at the metal surface (release of  $\text{Fe}^{2+}$  and  $\text{OH}^-$ , consumption of oxygen, etc.), followed by reactive transport in the pore system of the concrete in order to predict the location of precipitation of the corrosion products. This involves on the one hand formation of compounds with the ferrous ions (such as  $\text{Fe}(\text{OH})_2$ ) and on the other hand further oxidation to ferric ions, which then precipitate virtually immediately due to their very low solubility [25]. Deposition of corrosion products in the concrete pores will also affect the porosity and thus the transport properties of the matrix, which influences the

electrochemical processes at the metal surface. Thus, future models should be able to consider this feedback. Finally, predicting the precipitation of corrosion products in the pore system will allow for poromechanical considerations in order to predict cracking, which has been successfully used e.g. in the context of sulphate and frost attack of concrete or salt damage in stone [28-30].

### 3.5 Implications for engineering practice

The mechanisms leading to cracking of the cover concrete due to internally ongoing corrosion, discussed above, have implications for engineering practice.

For inspection of reinforced and prestressed concrete structures, it is crucial to acknowledge that in the case of chloride-induced corrosion, cover concrete cracking may be significantly delayed. This is important because inspection strategies are generally based on first visual inspection, where cracks, rust stains, etc. are taken as indication for corrosion or other degradation mechanisms. Since cracks may only become visually apparent at the concrete surface after considerably sectional loss of reinforcing steel has occurred, visual inspection may underestimate the damage. Practical cases illustrating this are documented in [24].

For carbonation exposure classes, on the other hand, the limit state is typically not the sectional loss of the steel, but rather the cracking of the concrete cover. Thus, for the design of new structures and to ensure that no such cracking occurs within the design life, the current limitations in modelling concrete cracking due to corrosion present a challenge. Since it is not possible to reliably predict the concrete cover cracking for practice-related conditions, engineers continue relying on empirical values for the amount of sectional loss required to lead to concrete cover cracking ( $D_{\text{cr}}$ ). While also these numbers may scatter widely from one case to another,  $D_{\text{cr}}$  is typically assumed around 50–100  $\mu\text{m}$ .

## 4 CONCLUSIONS

This contribution has highlighted a number of aspects related to 1) cracks as cause for corrosion of reinforcing steel in concrete, and 2) corrosion leading to cracking of the concrete cover. The discussion of the state-of-the-art knowledge has revealed that a number of questions still need to be answered in order to deliver reliable engineering concepts and models. Referring to Fig. 1, the most salient points are:

- There is agreement that cracks shorten corrosion initiation, but the actual extent to which this happens ( $\Delta t_{\text{ini}}$ ) depends on too many influencing factors to be generalized.
- Once initiated, the rate of corrosion in cracked concrete seems to decrease over time (due to crack-healing and other effects). Current knowledge seems to indicate that on the long-term, the effect of cracks on the corrosion performance is moderate ( $\Delta t_{\text{long-term}}$  may be small).
- It is currently not possible to stipulate critical crack widths below which corrosion would not occur. Even very thin cracks are zones where transport of chlorides or the carbonation front is accelerated compared to the bulk concrete. Once corrosion starts, the crack width seems to be a minor influencing factor as the kinetics of the corrosion process depend also on a number of properties related to the zones between the cracks (cover depth, concrete transport properties, exposure conditions, etc.).
- To predict concrete cover cracking due to rebar corrosion, only empirical models are currently available. They are limited in terms of reliability, which can be traced to the fact that they are generally calibrated against accelerated laboratory testing, which is not representative for natural conditions, and because models and experiments often mix chloride-induced and carbonation-induced corrosion, although it is well documented that these are fundamentally different in terms of concrete cover cracking mechanism. For carbonation-induced corrosion,  $D_{\text{cr}}$  – the amount of sectional loss required to lead to

concrete cover cracking – is in engineering typically assumed around 50–100  $\mu\text{m}$ . Current knowledge does not allow to improve this.

- For science-based mechanistic concrete cracking models to be developed, a number of aspects need to be addressed in detail. These include the oxidation kinetics of  $\text{Fe}^{2+}$  in concrete pore solutions, the diffusion of  $\text{Fe}^{2+}$  in the cementitious matrix, pore clogging due to precipitation of corrosion products and the related effect on transport properties and the corrosion kinetics, the arising crystallization pressure, and the poromechanical cracking mechanism.



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