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Tittel
Effekten av riss på kloridinitiert korrosjon av armering - en gjennomgang

Undertittel

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Godkjent av
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Emneord
Varige konstruksjoner, tilstandsutvikling bruer, armeringskorrosjon, bestandighet, langtidseffekt, klorid, riss

Sammendrag
Denne rapporten går gjennom studier på hvilken effekt riss i betong har på kloridinitiert korrosjon av armering. Riss kan føre til tidligere kloridinitiert korrosjon, men nåværende kunnskap tyder på at effekten av riss avtar med tiden, og muligens kan neglisjeres på lang sikt. Det finnes imidlertid få veldokumenterte, realistiske langtidsforsøk. Det ble også oppdaget store mangler i kunnskapen om effekten av rissfrekvens. I følge dagens forståelse av korrosjonsmekanismene i opprisset betong, vil korrosjonshastigheten på den lokale anoden i risset reduseres med økende rissfrekvens, men det er få undersøkelser som underbygger dette. Det er ikke mulig basert på nåværende kunnskap å angi kritiske rissvidder, verken for initiering av korrosjon eller for korrosjonshastighet. Dette understreker det store behovet for videre forskning.

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Title
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Key words
Durable structures, existing bridges, reinforcement corrosion, durability, long-term performance, chloride, cracks

Summary
This report reviews studies about the effect of concrete cracks on chloride-induced corrosion of reinforcement steel. Cracks can lead to earlier corrosion initiation, but the current state-of-the-art suggests that the effect of cracks on corrosion diminishes with time and may on the long-term even be negligible. Nevertheless, well-documented long-term studies under realistic field exposure are scarce. A further major lack of knowledge was identified with respect to crack frequency. According to current understanding of the working mechanism of corrosion in cracked concrete, it is expected that the corrosion rate at the local anodic site at the crack should decrease with increasing crack frequency. There is, however, very little experimental data confirming this suggested effect of the crack frequency. The current state-of-knowledge does not permit general recommendations for critical crack widths, neither for corrosion initiation nor for corrosion propagation. This clearly illustrates the urgent need for more research.

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Forord

Denne rapporten inngår i en serie rapporter fra **etatsprogrammet Varige konstruksjoner**. Programmet hører til under Trafikksikkerhet-, miljø- og teknologiavdelingen i Statens vegvesen, Vegdirektoratet, og foregår i perioden 2012-2015. Hensikten med programmet er å legge til rette for at riktige materialer og produkter brukes på riktig måte i Statens vegvesen sine konstruksjoner, med hovedvekt på bruer og tunneler.

Formålet med programmet er å bidra til mer forutsigbarhet i drift- og vedlikeholdsfasen for konstruksjonene. Dette vil igjen føre til lavere kostnader. Programmet vil også bidra til å øke bevisstheten og kunnskapen om materialer og løsninger, både i Statens vegvesen og i bransjen for øvrig.

For å realisere dette formålet skal programmet bidra til at aktuelle håndbøker i Statens vegvesen oppdateres med tanke på riktig bruk av materialer, sørge for økt kunnskap om miljøpåkjenninger og nedbrytningsmekanismer for bruer og tunneler, og gi konkrete forslag til valg av materialer og løsninger for bruer og tunneler.

Varige konstruksjoner består, i tillegg til et overordnet implementeringsprosjekt, av fire prosjekter:

Prosjekt 1: Tilstandsutvikling bruer
Prosjekt 2: Tilstandsutvikling tunneler
Prosjekt 3: Fremtidens bruer
Prosjekt 4: Fremtidens tunneler

Varige konstruksjoner ledes av Synnøve A. Myren. Mer informasjon om prosjektet finnes på vegvesen.no/varigekonstruksjoner

Denne rapporten tilhører **Prosjekt 1: Tilstandsutvikling bruer** som ledes av Bård Pedersen. Prosjektet vil generere informasjon om tilstanden for bruer av betong, stål og tre, og gi økt forståelse for de bakenforliggende nedbrytningsmekanismene. Dette vil gi grunnlag for bedre levetidsvurderinger og reparasjonsmetoder. Innenfor områdene hvor det er nødvendig vil det etableres forbedrede rutiner og verktøy for tilstandskontroll- og analyse. Prosjektet vil også frembringe kunnskap om konstruktive konsekvenser av skader, samt konstruktive effekter av forsterkningstiltak. Prosjektet vil gi viktig input i forhold til design av material- og konstruksjonsløsninger for nyere bruer, og vil således ha leveranser av stor betydning til Prosjekt 3: Fremtidige bruer.

Rapporten er utarbeidet av *A. Carolina Boschmann Käthler (ETH Zurich, Institute for Building Materials), Ueli M. Angst (ETH Zurich, Institute for Building Materials og Swiss Society for Corrosion Protection), Matthias Wagner (Tecnotest AG), Claus K. Larsen (Statens vegvesen) og Bernhard Elsener (University of Cagliari, Department of Chemical and Geological Science)* på oppdrag fra Varige konstruksjoner.

Effect of cracks on chloride-induced corrosion of reinforcing steel in concrete – a review

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Abstract

This report reviews literature studies about the effect of concrete cracks on reinforcement steel corrosion in chloride exposure environments. While there is general agreement that cracks can lead to earlier corrosion initiation, the current state-of-the-art suggests that the effect of cracks on corrosion diminishes with time and may on the long-term even be negligible. Nevertheless, the vast majority of literature results are from short-term studies (< 2 y); well-documented long-term studies (> 10 y) under realistic field exposure are scarce. A further major lack of knowledge was identified with respect to crack frequency. According to current understanding of the working mechanism of corrosion in cracked concrete, it is expected that the corrosion rate at the local anodic site at the crack should decrease with increasing crack frequency. There is, however, very little experimental data confirming this suggested effect of the crack frequency. The current state-of-knowledge does not permit general recommendations for critical crack widths, neither for corrosion initiation nor for corrosion propagation. This clearly illustrates the urgent need for more research.

Contents

1 Introduction

Reinforced concrete is in practice often cracked. Concrete cracking occurs whenever the tensile strength or the tensile strain capacity is exceeded. There are many different reasons: cracks induced by structural loads, cracks arising from a restricted volume change of the concrete (e.g. due to shrinkage or temperature changes) or cracks resulting from expansive pressure generated inside the concrete (e.g. due to alkali-silica reaction, sulphate attack). Cracks may appear within the very first hours after casting, but also only after several decades. A systematic overview of different types of cracks may be found in Ref. [1-3].

Cracks are recognised to impair the durability of reinforced concrete as they provide ingress pathways into the concrete for moisture and various aggressive species associated with different degradation mechanisms, e.g. reinforcement corrosion, frost damage, alkali-silica reaction, sulphate attack, etc. For the specific case of reinforcement corrosion, local shortcuts through the cover concrete particularly for chloride ions and CO_2 are of concern, but also local fast access of oxygen may have an impact. It has been experimentally shown that cracks promote chloride penetration, viz. that the chloride concentration and distribution in vicinity of the crack is comparable to the one at the exposed, free concrete surface [4-7]. Also higher carbonation depths at cracks have been experimentally shown in [7, 8].

This enhanced local carbonation or accumulation of chloride ions at the reinforcement steel is likely to give rise to corrosion initiation at the crack. Once corrosion has initiated, the crack may affect the corrosion rate by providing easier access to chlorides, moisture, and oxygen as compared with the uncracked zones of the cover concrete [7]. Using Tuutti's well-known schematic service life model [9], **Fig. 1** qualitatively shows the possible effect of cracks when reinforcement corrosion is the likely degradation mechanism.

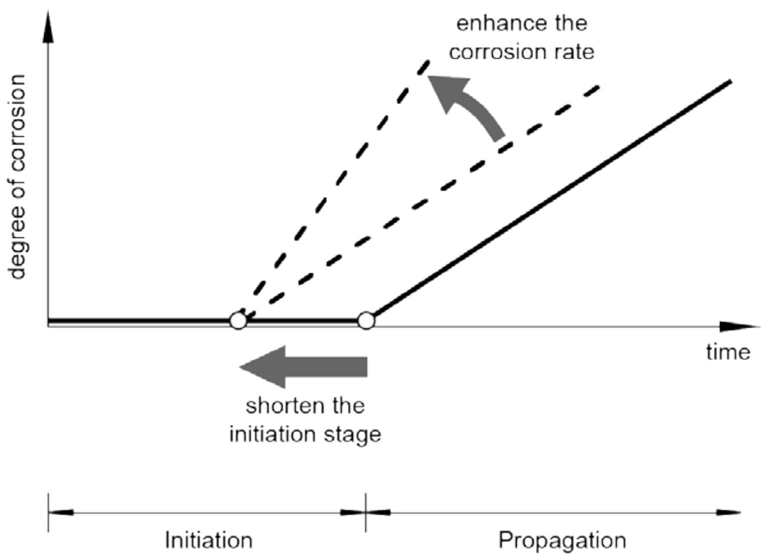

Fig. 1 Possible effects of cracks on the service life of reinforced concrete structures. The dashed line represents cracked concrete in comparison with uncracked concrete (solid line)

In previous literature reviews [7, 10-14] studies concerning the influence of cracks on corrosion were summarized, and the existence of a critical crack width was discussed. Even if no clear answer was found on the question of a critical crack width, design codes typically impose limits for crack widths and provide tools for crack width control [15, 16]. One may argue that crack width limits may be beneficial not only concerning reinforcement corrosion, but also with respect to other degradation mechanisms. Nevertheless, the *fib* model code for concrete structures 2010 [17], for instance, gives crack limits depending on exposure conditions. For chloride bearing environments or conditions promoting carbonation, thus for the specific case of reinforcement corrosion, the *fib* model code imposes crack width limits of 0.2 – 0.3 mm (depending on exposure condition and whether reinforcement steel or pre-stressing steel is considered).

By taking into account the most recent literature on the issue of cracks and their effect on steel corrosion and service life of reinforced concrete structures, the current literature review attempts to answer the following questions:

- Is there evidence that cracks shorten the initiation stage and, if yes, to what extent and under what conditions?
- Is there evidence that cracks enhance the corrosion rate during the propagation stage? If yes, to what extent and under what conditions?
- Do critical crack widths exist for initiation and propagation stages? What is the effect of the crack shape and the crack frequency?
- What type of experiments is the current state of the art based on? Evaluate the practical relevance of these and identify need for further research.

The present work focuses on chloride-induced reinforcement corrosion; the effect of cracks in the absence of chloride, viz. when only carbonation-induced corrosion may occur, is not considered. Moreover, only cracks perpendicular to the exposed concrete surface are here considered, thus, situations with cracks parallel to the exposed concrete surface (delamination, spalling) are not discussed.

2 Literature review

Tables 1–6 in the Appendix summarize the experimental work reviewed in this report; the used abbreviations are explained in Table 7. In Table 1, details about the specimens are given. This includes the number of investigated specimens, the size of the specimens and the cover depth. Concerning the number of specimens, in many studies different sets of experimental parameters were used. Thus, whenever the information was available, the number of parallel specimens, viz. the number of specimens with *identical experimental parameters*, is indicated by “per configuration”. In this work, configuration is defined as a set of identical parameters (parallel samples), including material properties (concrete mix design, cement type, etc.), specimen geometry, crack formation method (loading conditions, etc.), and exposure conditions. Special cases are those studies where destructive testing was done over time [18-21]. This led to a reduction in number of parallel samples over time, and at the end of the study, only a limited number of samples were left to draw final conclusions.

The cover depth, also given in Table 1, is considered as a relevant information, as for bending cracks – and in most studies, cracks were introduced by bending – the extent to which the crack narrows down at the steel depth compared with the crack width at the concrete surface depends on the cover depth.

Table 2 and Table 3 summarize material specific details: for concrete the maximum aggregate size, binder type, w/b-ratio, and curing conditions are reported, and the reinforcement steel properties for unsegmented (Table 2) and segmented (Table 3) steel specimens are given. Typically, studies with high degrees of freedom – for instance many different concrete mixes, different cover depths, different exposure conditions, etc. – did not have the primary aim of investigating the effect of cracks. This potential difficulty with interpreting the results will be addressed later in this review.

Table 4 shows how cracks were introduced, the crack widths that were studied and if they were fixed to ensure a constant width. It is also distinguished between specimens with only one single crack and specimens with several cracks. The column termed “uncracked specimens” indicates whether specimens without any cracks were used as reference specimens. This is considered important since only reference specimens allow assessing the effect of cracks within a given set of experimental parameters, without having to rely on data from crack-free specimens that were tested under different conditions (e.g. literature results or uncracked regions of cracked specimens).

Table 5 summarizes the exposure duration as well as the environmental conditions. Additionally, the type of the measurements and their frequency is shown. As will be apparent from this literature review, the exposure duration is one of the most relevant parameters.

Finally, Table 6 summarises the main conclusions drawn by the authors of the studies.

2.1 Common experimental setups

In most studies, the tested beams were relatively small; only approx. one third of the reported cases had beams that were at least 1 m long (Table 1). The reinforcement steel bars were in the tension zone of the beams. In some studies [22-29], the steel bars were cut in small pieces and electrically isolated from each other in order to form segmented bars. While this permitted more refined measurements such as corrosion potential measurements of individual segments or macro-cell currents, the segmenting may potentially introduce experimental artefacts, for instance the risk of crevice corrosion at the sealing between the segments, or a different mechanical behaviour with respect to tensile strength or ductility characteristics of the segmented rebar. All relevant information concerning these studies is directly listed in Tables 1-6, except Table 2.

Fig. 2 schematically shows a type of experimental setup that was often used in the literature to study the effect of cracks on corrosion. The typical approach was to fix two beams in a test rig subjecting them to either three point bending or four point bending and thus leading to cracking on one side of each beam. While three point bending leads to one crack, the location of which is relatively well defined, four point bending gives rise to several cracks distributed over the length at which the maximum bending moment arises. **Fig. 3** summarises the methods by which cracks were introduced; 75% of all experiments reported were bending tests.

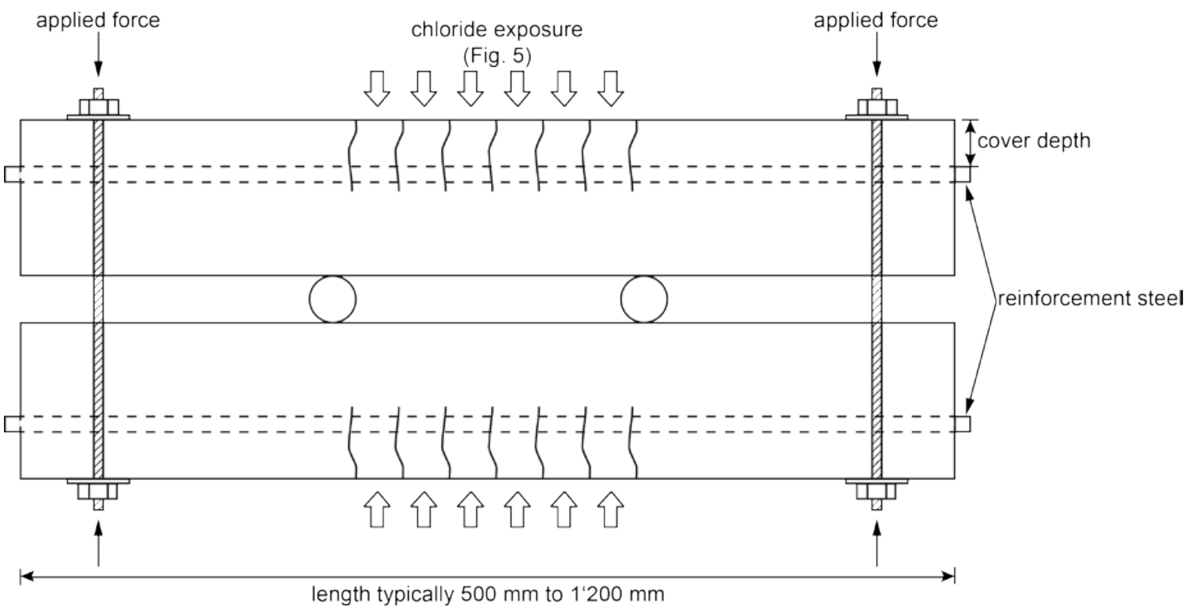

Fig. 2 Schematic illustration of a test setup frequently used in the literature. Here, the case of four point bending (leading to several cracks) is sketched, but also three point bending (leading to one crack) was often used

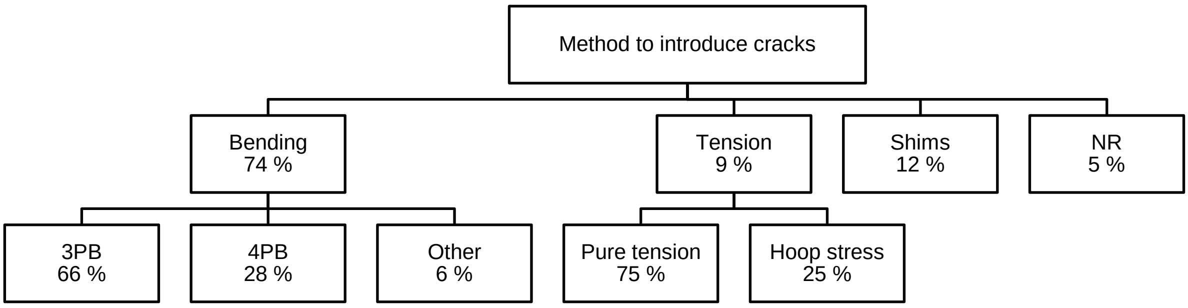

Fig. 3 Common methods to introduce cracks in the specimens. The numbers indicate the percentage with which a certain method was used in the literature

As is apparent from **Fig. 4**, typical investigated crack widths were between 0 and 0.5 mm. In approx. 80% of the reviewed studies, the crack width was below 1 mm; in exceptional cases it reached even 5 mm. Although the commonly used cracking methods (**Fig. 3**) such as four point bending or pure tension led to several cracks in the specimen, most studies unfortunately did not report any information on the crack frequency (number of cracks, distance between the cracks). This was only done in one work [30], where different numbers of cracks along a beam were studied (with the sum of all cracks widths within a specimen held constant). Table 4 shows that 18 investigations did have uncracked reference specimens, while in 19 studies no results of uncracked specimens were reported.

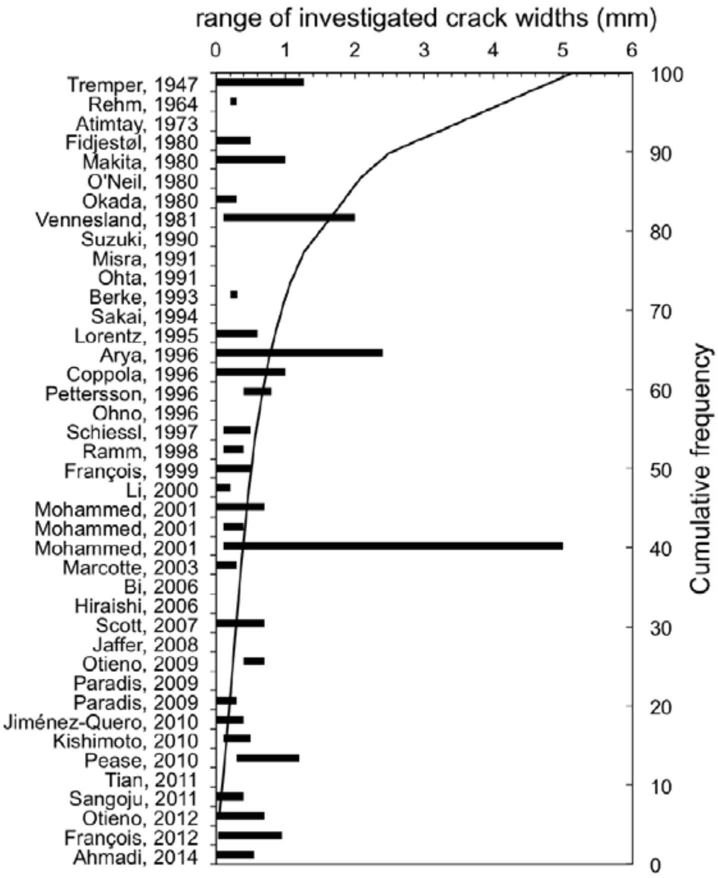

Fig. 4 Ranges of investigated crack widths in the literature. Shown by reference and as cumulative frequency distribution

The exposure conditions were rather variable and included both field exposure (marine environment) and a number of laboratory exposure conditions (**Fig. 5**). Generally, artificial exposure media were realistic or only moderately increased in terms of chloride concentrations, i.e. the chloride ingress was typically not severely accelerated in the laboratory studies.

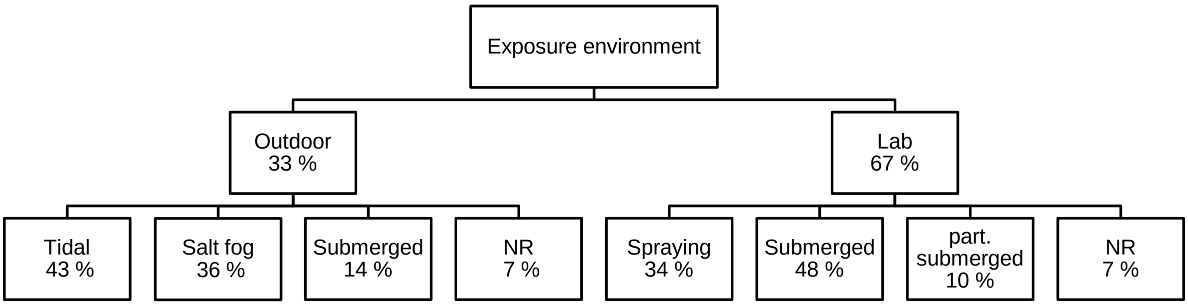

Fig. 5 Common exposure situations. The numbers indicate the percentage with which a certain exposure condition was used in the literature

2.2 Measurement methods and time frame

Fig. 6 shows the number of studies as a function of the exposure duration reported. Note that some studies reported results from several different exposure durations (compare Table 5). For these cases, always the maximum reported exposure duration was used for plotting the histogram shown in **Fig. 6**. In any case, it is clearly apparent that the vast majority of studies employed relatively short exposure times. In fact, in approximately 70% of the cases the effect of cracks on reinforcement corrosion was studied during less than 3 y. In some cases, the experimental duration was even shorter than one year, and amounted for only a few weeks to months [24-26, 31-40]. If as an arbitrary assumption, long-term studies were considered as studies performed over more than 10 y of exposure, it becomes apparent that in total only 7 long-term studies are available (thus only approx. 20% of the reviewed studies).

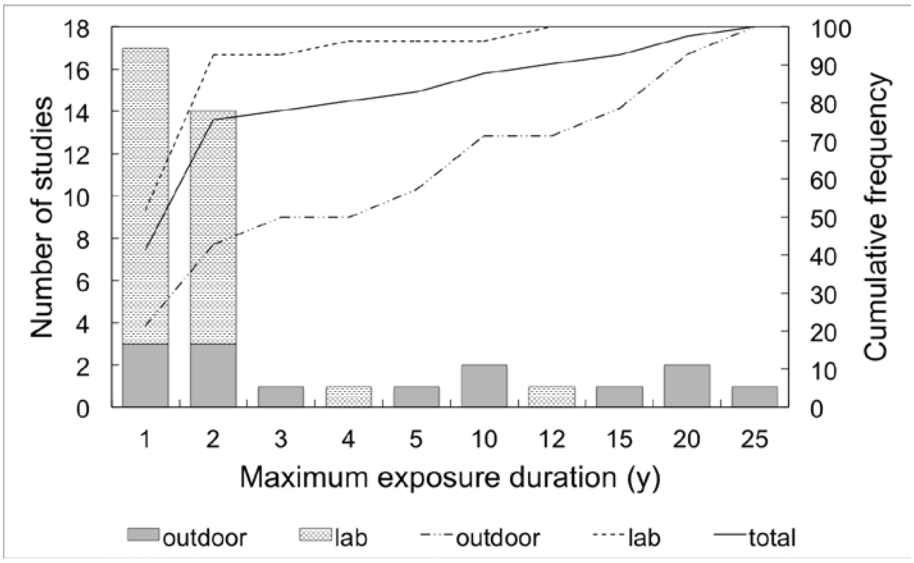

Fig. 6 Histogram (bars) and cumulative frequency (lines) of studies as a function of exposure duration

It may also be worth pointing out that in general more detailed measurements were performed in the short-time studies than in the long-term studies. This applies both to the number of complimentary measurement methods and to the time resolution of the investigations. For instance, in Refs. [25, 41, 42], specimens were exposed to marine environment for 15 y; however, measurements to characterize the effect of cracks on corrosion were only performed at the very end of the tests, viz. at 15 y when the specimens were transferred to the laboratory. In Ref. [43], specimens were subjected to marine field exposure during 25 y and annually investigated by visual inspections. At the end of the field trial, the residual specimens (11 of 82 specimens) were transported to the laboratory and more detailed investigations were performed (such as structural testing, but no electrochemical measurements). Also Ohta [19] reported results from marine field exposure where investigations were performed at two times, viz. after 10 y and 20 y of exposure. This included visual inspections, determination of steel cross section loss and measurements of steel potentials. Thus, in these studies, there is very little information available on the time evolution. Especially the time of corrosion initiation is not well known. Obviously, this limits the conclusions that can be drawn with respect to the effect of cracks in concrete on both corrosion initiation and corrosion propagation.

In the work by François and co-workers [20, 21], cracked beams were exposed to chloride for 12 y and then left (in unloaded condition) in an outdoor, chloride-free environment until the age of 27 y. Crack patterns were visually recorded at different ages; at the end (27 y) selected beams were opened and the loss of steel cross section determined [44, 45]. Nevertheless, the long-term experience of this work concerning the effect of cracks on corrosion may be less than 27 y, since over the last years, in addition to the initially present cracks, there were also numerous cracks *caused by corrosion*. Thus, the cause and effect of cracking cannot be separated easily of the last stage of this study.

One may thus conclude that for the long-term studies, measurements were generally performed at relatively long intervals and that continuous and well time-resolved monitoring is scarce. Often, the conclusions were based on visual assessments only. For the short-term studies, on the other hand, the evolution of the deterioration is generally documented in a more detailed manner, also including electrochemical measurements. Table 5 shows in column 3 and 4 all measurements and their frequency executed during the studies.

2.3 Relevance

Table 6 summarizes the main observations and conclusions drawn by the respective authors in the reviewed studies. However, when evaluating the literature – i.e. assessing trends, general agreements, and controversial issues (compare following sections) – we do not consider all studies equally relevant. The reason for this is that in our opinion, differences concerning a number of experimental details affect the relevance of the observations made and the conclusions drawn on the basis of individual studies.

For instance, the number of parallel specimens, i.e. specimens of identical experimental configuration, is in the present context considered as an important parameter. This varied strongly: while some studies only had one sample per configuration [46, 47], others had three or more [25, 26, 30, 35, 36, 41, 42, 48]. In several studies, however, the number of “parallel samples” was not clear (Table 1). As will be apparent from the discussion, primarily due to the high variability inherent to corrosion in concrete, the number of parallel samples is indeed a crucial parameter when designing an experiment on the role of concrete cracks in reinforcement corrosion. Studies with few parallel samples generally permitted conclusions only with a low level of confidence. As mentioned above, for some long-term studies, the number of specimens decreased with time, which limited the confidence with which conclusions could be drawn at later stages.

Similarly, we also consider the presence of uncracked reference specimens as an essential requirement for experimentally assessing the effect of cracks on corrosion. Table 4 shows that only approximately half of the studies reported the results from uncracked reference specimens. A frequent reason for the absence of uncracked specimens was that the objectives of the studies were not always specifically to investigate the effect of cracks on corrosion, but to study the influence of other parameters on corrosion in cracked concrete, such as w/b-ratios, cover depths, binder types, exposure conditions, etc. Nevertheless, it was in these studies occasionally attempted to draw conclusions also on the effect of cracks on corrosion, such as by comparing cracked and uncracked regions within certain specimens. Since the interaction of cracked and uncracked regions likely affects the corrosion behaviour (e.g. mutual polarization), we do not consider this as an optimum experimental design in order to assess the effect that the presence or absence of cracks has on reinforcement corrosion.

Another criterion of relevance was the method of measurements and the frequency of measurements. Electrochemical methods, such as potential, linear polarisation resistance and macro-cell current measurements, are here considered to be more precise in determining the onset and the rate of corrosion than only visual inspections of the outer concrete surface or destructive sampling at typically long time intervals. This is because electrochemical (non-destructive) measurements can be applied on a frequent basis and thus yield time-resolved information. In the work summarized in Refs. [25, 41, 42], however, electrochemical methods were used but only at the very end of the 15 y exposure duration.

3 Discussion

3.1 Corrosion initiation

It was in practically all studies found that cracks shorten the initiation stage (Table 6). It is noticeable, that this observation was made with all the different experimental setups and configurations, particularly also in both short-term and long-term studies.

While there appears to exist a general agreement that cracks shorten the initiation period, there is no clear trend in the literature about the extent to which cracks influence the time to corrosion initiation. While some authors found “immediate initiation” at cracks [49, 50], most studies did not report about the extent to which cracks shorten the initiation stage. Generally, this is because the time to corrosion initiation in uncracked concrete was unknown due to the absence of uncracked reference specimens, or because the time-resolution of the measurements done was too low to quantify the time-to-corrosion sufficiently accurate.

Fig. 7 summarizes reported times to corrosion initiation as a function of crack width. With the exception of François [20], all other studies [25, 26, 35, 36, 49] showed that the time to corrosion initiation is shorter the wider the crack.

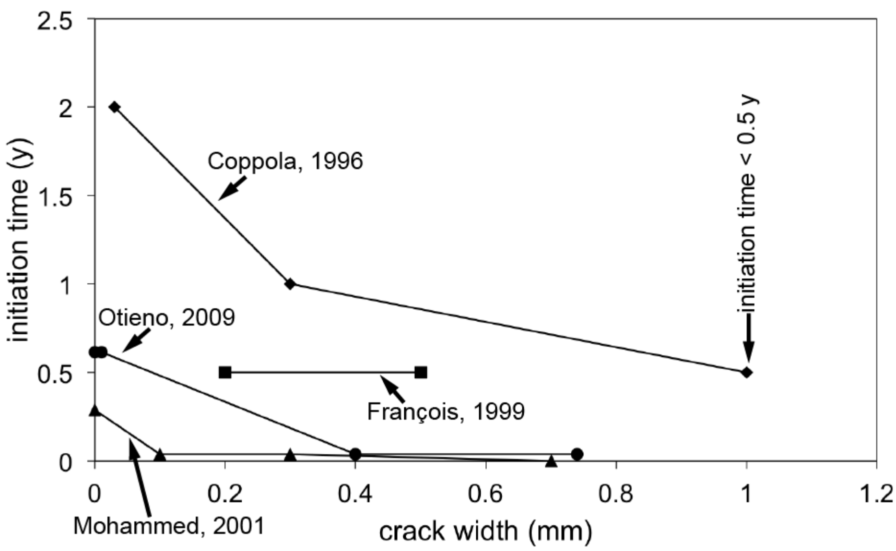

Fig. 7 Influence of crack width on the time to corrosion initiation, reported in different laboratory studies [20, 25, 26, 35, 36, 49]. The initiation time for the smallest crack widths is the minimal initiation time, because the exposure duration ended before the corrosion initiated. The w/b-ratio was in all studies in the range of 0.4-0.5, and the exposure conditions were cyclic ponding or spraying

The reason why cracks shorten the time to corrosion initiation was largely claimed to be the faster ingress of chlorides through cracks compared with uncracked regions [20, 21, 26, 49-52]. It was in several studies observed that corrosion occurred in close proximity of cracks [25, 31, 41-43, 47, 49, 53, 54]. This explanation is supported by additional studies that investigated penetration of chlorides (or the carbonation front) through cracked concrete [4-8], which provide evidence that chlorides can reach the reinforcement faster at cracks. An additional reason for the earlier initiation of corrosion at cracks, occasionally suggested in the literature [43], is that also oxygen and moisture are more easily supplied at cracks, which enhances the corrosion process. Finally, it is also claimed [24, 50] that cracks present a local weakness (defect) at the steel-concrete interface, which may give rise to preferred corrosion initiation.

3.2 Corrosion propagation

Concerning the influence of cracks on corrosion propagation, the literature results are contradictory. While some studies reported an effect of cracks on corrosion propagation, typically in the sense that larger crack widths lead to higher corrosion rates or larger accumulated corrosion damage [18, 30, 35-38, 46, 55], a number of studies indicated that cracks did not enhance the corrosion process or that the enhancing effect diminished with time [20, 21, 26, 47, 50, 53, 56]. This can be illustrated with the selected literature results plotted in **Fig. 8**.

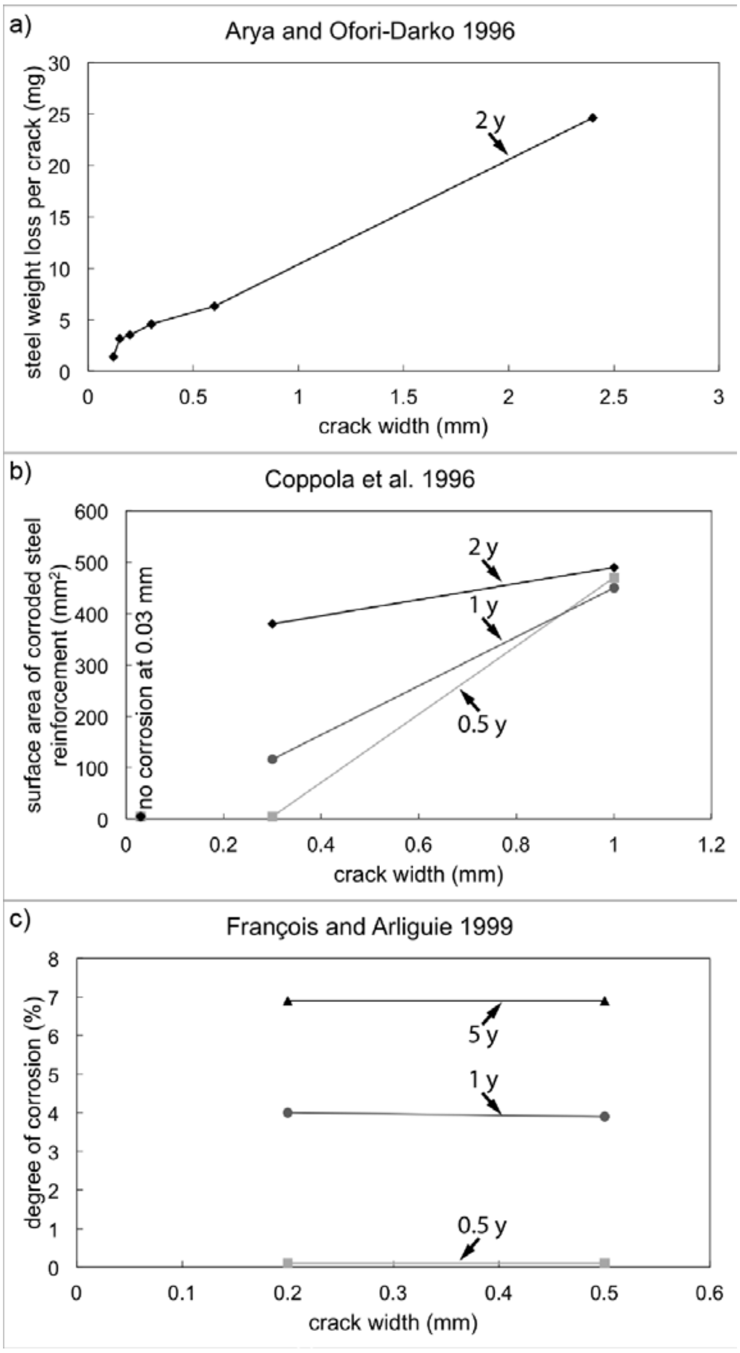

Fig. 8 Effect of crack width on corrosion damage from literature data after 0.5 to 5 y of exposure, illustrating a) a more severe corrosion damage for increased crack widths [30], b) an initially pronounced, but with time diminishing effect of crack width (apparent as a decreasing slope over time; short-term results) [49], and c) no effect of the crack width on the degree of corrosion (percentage of the corroded area in relation to the total area of the longitudinal reinforcement) [20]

An influence of cracks on the corrosion rate is generally seen in short-term studies. However, there are also some short-term studies where it was observed that cracks only have minor influence on corrosion propagation [25, 26, 47, 56]. In most long-term trials, on the other hand, the effect of cracks on corrosion propagation was much less pronounced or even inexistent. For instance, François and co-workers, concluded on the basis of long-term results (> 10 y) that for crack widths below 0.5 mm “the development of corrosion is not influenced by the crack widths or their existence” [20]. The same conclusion was also reached by Mohammed et al. [25, 41, 42] where after 15 y of exposure no influence of cracks (width < 0.5 mm) on corrosion propagation was found for all studied cement types, which was by the authors explained by the ability of the cracks to “self-heal”. Unfortunately, the experimental data presented in other long-term studies [19, 43] hardly permits drawing any conclusions with respect to the effect of cracks on corrosion propagation.

Concerning the studies utilizing segmented steel bars, some results from short-term investigations (1–4 y) indicated that the propagation of corrosion is enhanced in the presence of cracks [23, 27, 28]. Nevertheless, even during short-term tests (0.25–2 y), tests with segmented bars indicated that the influence of cracks on corrosion rate declines with time [22, 25, 26].

There is only little literature data on the extent to which cracks can enhance the corrosion rate, particularly concerning long-term exposure. In Ref. [34] it was reported that cracks increased the corrosion rate by orders of magnitude (exposure duration < 1 y). Less pronounced effects were found in Ref. [55] (exposure duration 1.5 y) where the corrosion rate increased by up to 75% when increasing the crack width from 0.2 to 0.7 mm, and in Refs. [35, 36] (exposure duration < 1 y) where an increase in corrosion rate by up to 210% in the presence of cracks was observed (Table 6). These are, however, without exception results obtained from extremely short-term testing (max. 1.5 y). As mentioned above, despite the lack of detailed data on the time-evolution, long-term studies typically indicated a negligible effect of cracks on the corrosion propagation.

3.3 Critical crack widths

3.3.1 Definitions

It has long been appealing to engineers to define critical crack widths, i.e. threshold values of crack widths below which the impact of the crack on reinforcement corrosion is considered uncritical. Such critical crack widths have been proposed to exist for the following situations:

- corrosion initiation ($w_{crit,ini}$), i.e. no earlier corrosion initiation at < $w_{crit,ini}$ compared to uncracked concrete,
- corrosion propagation ($w_{crit,prop}$), i.e. that corrosion may initiate, but does not propagate significantly in cracks below a critical width $w_{crit,prop}$,
- self-healing ($w_{crit,heal}$), i.e. below $w_{crit,heal}$ cracks are able to self-heal.

An alternative suggestion was to use the ratio of crack width to cover depth as threshold values [39], but no critical threshold ratio was described. Nevertheless, critical crack widths have been subject to controversial discussion already in the 1970s [11, 12].

In the literature reviewed in this work, some authors reported critical crack widths for corrosion initiation $w_{crit,ini}$, typically in the range 0.1–0.5 mm [18, 23, 32, 33, 43, 47, 49, 52] (Table 6). **Fig. 9** summarizes the reported $w_{crit,ini}$ as a function of cover depth, clearly illustrating an overall influence of cover depth on threshold crack widths. Numerous authors [25, 26, 35, 36, 50] also stated that no critical crack widths can be recommended, i.e. that such a threshold does not exist.

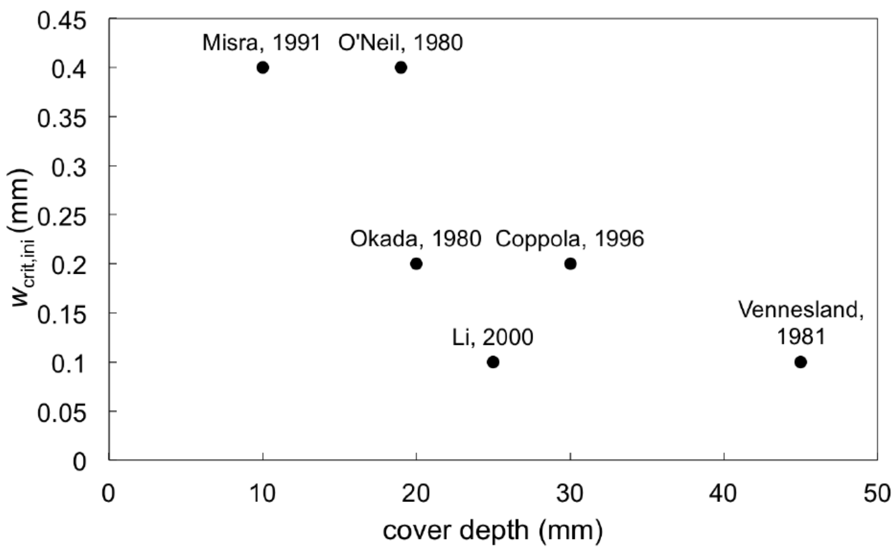

Fig. 9 Reported $w_{crit,ini}$ as a function of cover depth. All cracks were introduced by bending (3PB:[33, 43, 49, 52]; 4PB:[47]), with exception of Okada [32], where tension cracks were studied

3.3.2 Self-healing

Crack self-healing describes the clogging of the crack due to precipitation of corrosion products or deposits such as ettringite or calcite. As a crack gets blocked by precipitates, transport processes become more restricted. This applies to the transport of ferrous ions away from the steel as well as to the supply of chloride ions to the pit, which is needed to sustain pitting corrosion. Thus, as a result of crack self-healing, the corrosion rate decreases and may eventually reach a negligible level.

A number of researchers investigating corrosion in cracked concrete showed experimental evidence for crack self-healing [25, 33, 41, 42, 53, 57] or assumed [30, 52] possible self-healing of cracks to explain the decrease of the corrosion kinetics over time. A few authors also suggested critical crack widths for self-healing, namely in Ref. [33] $w_{crit,heal}$ = 0.4 mm and in Refs. [25, 41, 42] $w_{crit,heal}$ = 0.5 mm.

Another study that led to the suggestion of a value for $w_{crit,heal}$ is Ref. [58]. In this work, autogenous healing of cracks exposed to water pressure was investigated, however in the absence of corrosion (no precipitation of corrosion products possible). Calcite formation was the main reason for autogenous healing; minor ones were swelling of concrete and mechanical processes, such as blocking the crack with loose concrete particles. Critical crack widths were defined up to $w_{crit,heal}$ = 0.25 mm, depending on hydraulic gradient and crack motions.

Two studies [52, 57] reported crack healing under sustained load; in Ref. [52] it was mentioned that healing of cracks is more difficult under sustained load. Two of the studies [25, 33, 41, 42] measuring self-healing did not fix the samples during exposure or did not report about fixing conditions, which means that the crack widths can also decrease due to release of load.

3.3.3 Relation of crack width to crack shape and to loading conditions

Most cracks investigated in the reviewed studies were induced by bending, therefore they are V-shaped and reach the reinforcement level. An important consequence of this is that the crack width apparent and measureable at the concrete surface is wider than the crack width at the reinforcement steel surface. In contrast to bending cracks, cracks arising from pure tension have essentially parallel crack walls, and thus the crack width at the concrete surface corresponds to the one at the reinforcement steel level. In short, there is no direct and generally valid – i.e. independent of loading conditions – relationship between surface crack widths and crack widths at the steel surface. It has early been noted that this makes it difficult to relate surface crack widths to corrosion phenomena [11, 12]. As is apparent from Table 4, there were no systematic studies that compared bending and tension cracks in terms of their effect on corrosion in the literature.

In some of the reviewed studies the loading conditions were sustained or the cracks were fixed and thus kept open during the exposure, while in other studies, the load that induced cracking was released prior to exposure, which allowed the cracks to close to some extent. Unfortunately, it was not always clear at what time the crack widths were measured, i.e. before or after releasing the load, which further complicates the interpretation of the results.

One investigation [52] was concerned about the influence of loading conditions on corrosion and concluded that corrosion does not only depend on crack width but also on the loading conditions. It was found that for crack widths < 0.1 mm the steel potential was less negative when the load was released compared to sustained loading conditions. However the crack widths were always measured under sustained load, i.e. upon releasing the load the crack widths have likely decreased, which may explain the less negative steel potentials. Only one investigation [54] considered also dynamic loading. Nevertheless, the conclusion of [54] was that type of loading had less impact on corrosion than the type of concrete or exposure conditions.

3.4 Crack frequency

The effect of the crack frequency (or the distance between cracks) has received very little attention in the literature. Although numerous researchers used four point bending or tension to introduce cracking and thus had specimens with several cracks, little or no information is generally reported on the crack frequency. One exception is the work of Arya et al. [30] where the crack frequency was explicitly taken into account as an experimental parameter. Beams of 1.36 m in length were manufactured and different numbers of cracks were introduced with shims. The number of cracks within a specimen varied from 1 to 20 (Table 4), but the sum of all crack widths within a specimen was kept constant at 2.4 mm. This was achieved by casting in plastic shims of appropriate thickness. This means that the width of the individual cracks varied from 0.12 mm in the specimens with 20 cracks to 2.4 mm in the specimens with only one crack. In addition, uncracked reference specimens were prepared. The beams consisted of two mild steel rebars and one stainless steel rebar. Each configuration was tested with four parallel specimens. Upon exposure to chlorides the weight loss of the mild steel bar was measured with different electrochemical techniques, i.e. with help of corrosion rate and macro-cell current measurements, integrated over time. This indicated that with increasing crack frequency more steel weight loss occurred. From the original data by Arya et al. [30], the total steel mass loss can be related to the number of cracks, which yields an average steel mass loss occurring in individual cracks (**Fig. 10**). This shows that with increasing crack frequency the average corrosion rate occurring in an individual crack decreases, or in other words, that the average corrosion rate at a certain crack tends to increase with increasing crack distance.

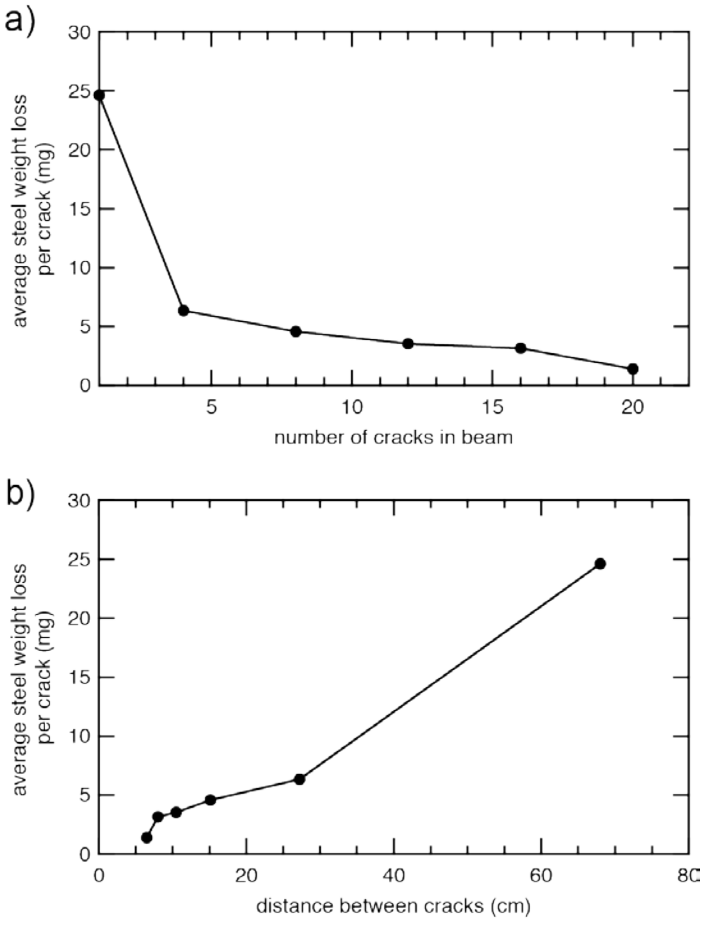

Fig. 10 The effect of crack frequency illustrated by an evaluation of the original data reported by Arya et al. [30] after 24 months of chloride exposure: a) the average cumulative weight loss at individual cracks as a function of the number of cracks, and b) as a function of the distance between the cracks. Note that the sum of all crack widths within each specimen was held constant at 2.4 mm (with help of shims)

This is in agreement with a numerical study by Schiessl and Raupach [22] that demonstrated that the corrosion rate of a corroding site located at a crack decreases with decreasing distance between cracks. Thus, increasing the crack frequency decreases the local corrosion rate at each corroding spot. This was in Ref. [22] explained by the fact that shorter crack distances limit the cathodic area available to contribute to the macro-cell current.

3.5 Main influencing factors on the corrosion performance of cracked concrete

3.5.1 The w/b-ratio

All studies investigating different w/b-ratios agreed that decreasing the w/b-ratio has a positive impact on concrete quality and therefore on the corrosion performance [22, 26, 32, 35, 36, 38, 40, 41, 49, 51, 56, 59-61], concerning both initiation and propagation of corrosion. Most studies also agreed that the relationship between w/b-ratio and corrosion rate is clearer than the relationship between crack width and corrosion rate [26, 32, 41, 60, 61]. Three studies [32, 60, 61] stated that a correlation between crack width and corrosion rate exists only for lower w/b-ratios. Typical w/b-ratios in infrastructure construction are around or below 0.4. However, some studies used relatively high w/b-ratios above 0.6, and even up to 0.8 (**Fig. 11**). At low w/b-ratios, the contrast in concrete permeability between cracked and uncracked zones is expected to be more pronounced than at high w/b-ratios (porous concrete). The results from studies with a relatively high w/b-ratio should thus be interpreted with caution when drawing conclusions for practical situations.

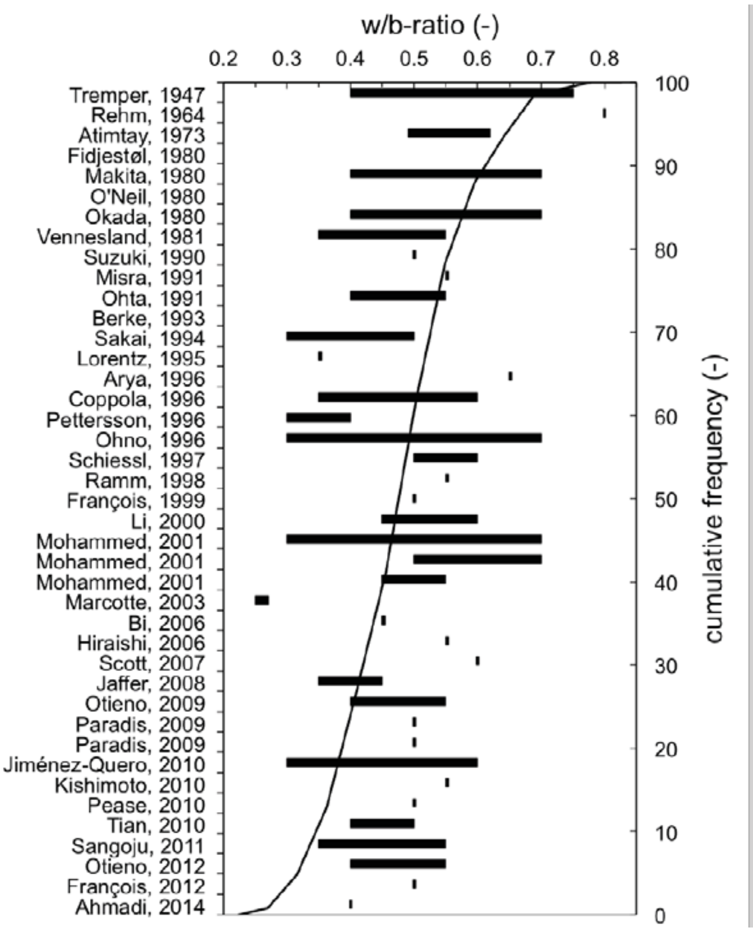

Fig. 11 Ranges of investigated water-binder ratios in the literature. Shown by reference and as cumulative frequency distribution

3.5.2 Binder type

It is widely known that the binder type has an effect on corrosion initiation and propagation in general [62], and most studies also agreed with this for the case of cracked concrete. To mention some examples: Mohammed et al. [25, 41, 42] investigated the influence of slag and fly ash blended cement compared to ordinary Portland cement (OPC). It was concluded, that all supplementary cementitious materials (SCM) showed a better performance concerning the extent of corrosion compared to OPC for crack widths of 0.1 mm. Slag blended cement had a strongly beneficial effect on corrosion performance in cracked concrete, whereas for fly ash the effect was more moderate. Scott et al. [55] compared silica fume, blast-furnace slag and fly ash blended cement with OPC. It was concluded that the binder type (and also the cover depth) had more influence on the corrosion rate than the crack width. The increase of corrosion rate for crack widths varying from 0.2 to 0.7 mm was highest for silica fume blended cement and lowest for fly ash and Portland cement. For slag blended cement the increase of corrosion rate due to increasing crack width from 0.2 to 0.7 mm was moderate. Otieno et al. [35, 36] investigated the influence of different crack widths (0 to 0.7 mm) on different concrete compositions (slag cement and OPC, w/b-ratios 0.4 and 0.55). It was concluded that increasing the crack width leads to increased corrosion rates. This increase was smaller for slag cement than for OPC specimens. For slag cement the corrosion rate was generally lower than for OPC. Also a lower w/b-ratio led in general to decreasing corrosion rates.

3.5.3 Damage at the steel/concrete interface

Another aspect of the influence of cracks on corrosion is the interfacial damage resulting from the crack. Cracks introduced by loading are accompanied by secondary cracks at the steel-concrete interface (SCI) due to transmission of tension forces between the concrete and the steel (**Fig. 12**a) [63, 64]. Several authors [24, 50, 65] have recognized the influence of such secondary cracks and the damaged SCI. For instance, François et al. [50] described in their study that the corrosion initiated at the root of the crack, “and then propagates along the length of the steel-concrete interface on a few mm in the zone which is damaged by the creation of the crack”. Paradis [65] proposed that there was a difference between cracks introduced with shims and with bending with respect to the steel-concrete interface. For cracks introduced with shims (**Fig. 12**b), even if they reach the reinforcement level, a small cement paste layer might exist on the steel surface, which will protect the steel better than if the layer was destroyed due to bending introduced cracks.

However, Mohammed et al. [25, 41] reported that corrosion initiation does not necessarily occur at the root of the crack but also at voids at the steel-concrete interface. This means that the crack and the related interfacial damage is not necessarily the weakest point at the steel-concrete interface.

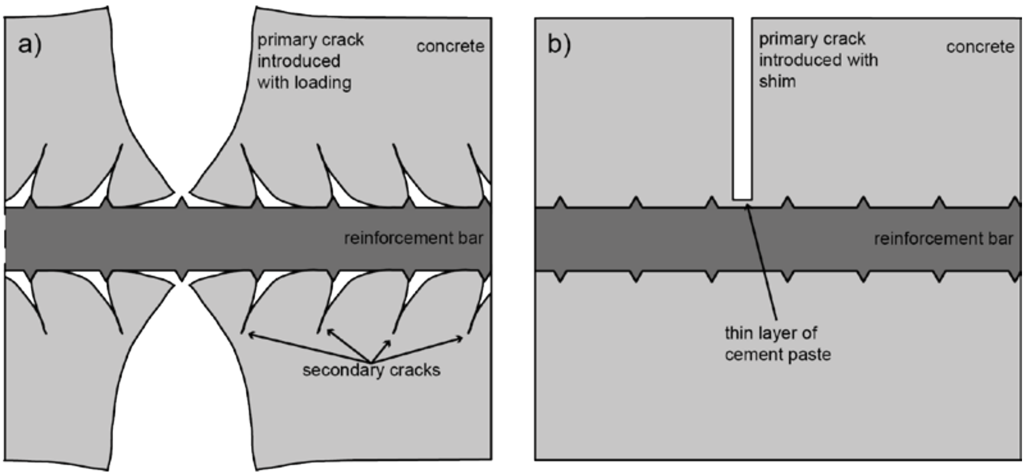

Fig. 12 Schematic illustration of the differences arising from load-induced (a) and shim-induced (b) cracks, respectively. As shown in Figure a), applied bending and tension forces induce a primary crack, and secondary cracks due to load transmission between reinforcement and concrete. Corrosion can occur at the root of the primary crack and/or at the secondary cracks. Figure b) shows that cracks introduced with embedded shims possibly provide a thin, protective layer of cement paste at the crack root

3.6 Corrosion mechanism in cracked concrete

Schiessl et al. [7, 22] proposed two distinct mechanisms for the effect of cracks on the corrosion process in general:

- In mechanism 1, both the cathodic process (oxygen reduction) and the anodic process (iron dissolution) occur at the root of the crack. The oxygen for the cathodic reaction is mostly provided through the crack. It may be noted that this requires that the crack is not continuously saturated with solution.
- In mechanism 2 (**Fig. 13**), the anodic process takes places at the root of the crack, while the passive steel between the cracks acts as cathode; supply of oxygen occurs through the concrete cover between the cracks, and may also be facilitated through the crack.

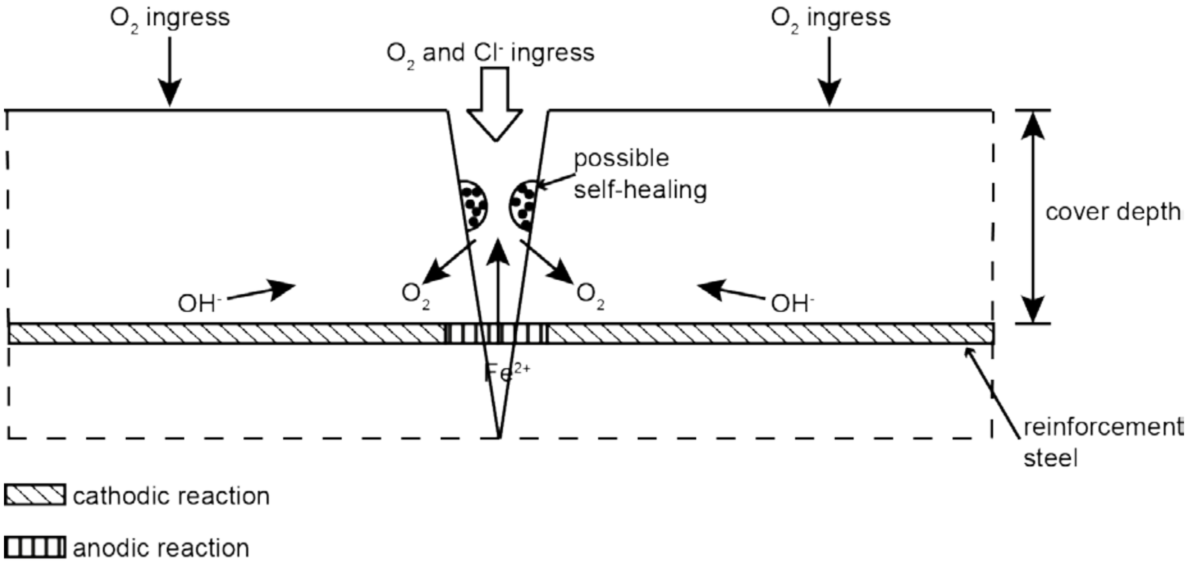

Fig. 13 Schematic illustration of possible corrosion mechanisms at cracks (subjected to cyclic wetting/drying or continuously submerged). The cathodic reaction takes place between the cracks, whereas the anodic reaction takes place in the region of the crack. Possible self-healing may clog up the crack and decrease the corrosion rate by affecting the anodic reaction

Based on theoretical reasoning and on experimental data, Schiessl et al. [7, 22] suggested that mechanism 2 is the relevant one in practice, rather than mechanism 1. Corrosion rates were expected to be relatively low in mechanism 1 because i) the cathodic area supplied with oxygen through the crack is small, and ii) because precipitation of corrosion products in the crack will suppress the corrosion process over time. Assuming that mechanism 2 is operating, the authors thus suggested that the *quality of the concrete between the cracks* is governing the corrosion rate. In this regard, two parameters are important: the oxygen permeability of the concrete strongly affects the cathodic reaction kinetics [7, 55], and the electrical concrete resistivity may limit the macro-cell corrosion current by ohmic control. Experimental evidence for the hypothesis on the mechanism of corrosion in cracked concrete proposed by Schiessl and co-workers primarily comes from studies using segmented steels [27, 28], where it was generally documented that the steel segment in the cracked region acts as anode while the other segments act as cathode (short-term studies with maximum exposure duration 2 y). The proposed hypothesis, that the concrete quality between the crack plays a dominant role, is also supported by the general literature findings on the effect of binder type and w/b-ratio (see section 3.5.1 and 3.5.2).

In [66] the investigated multi-crack specimens led to the conclusion that the widest crack induced corrosion earliest. It was found that these so-called “major cracks” were the site of anodic iron dissolution, while the rest of the rebar acted as cathode. Interestingly, many of the “minor cracks” were free from corrosion and it was suggested that these sites were mutually protected by polarization from the anode at the “major crack”. This observation was also made by Rehm and Moll [18] and O’Neil [43] where multi-crack specimens were studied and corrosion initiated only at some of the cracks. Thus, the assumption made by Schiessl and Raupach when modelling the effect of crack frequency [22], where it was assumed that at each crack an anode is formed, is not always found in real samples.

4 Conclusions and suggestions for further research work

In this report, a number of literature studies were reviewed to assess the effect of cracks on reinforcement steel corrosion in chloride exposure environments. The following conclusions can be drawn:

1. There is agreement that cracks can lead to earlier initiation of chloride-induced corrosion and thus shorten the initiation stage. This was reasonably explained and underpinned by experimental evidence of faster ingress of chlorides through cracks compared with uncracked regions. There is, however, no agreement in the literature concerning the extent to which cracks shorten the initiation stage. It is also unclear whether initiation of corrosion at a crack can lead to similarly stable pitting corrosion as in uncracked concrete or if the corrosion process may intermediately stop due to the more variable conditions (moisture, chemistry, etc.) at the crack.
2. Concerning the corrosion propagation stage, it was in some short-term studies found that cracks enhance the corrosion rate and that wider cracks lead to higher local corrosion rates. However, this could not be confirmed in the few long-term studies that are available in the literature, and it was also contradicted by some short-term studies. Based on the current state-of-the-art, the extent to which cracks enhance the corrosion rate on the long-term is difficult to be quantified but can be considered moderate.
3. Several studies indicated that cracks may undergo self-healing, i.e. clogging of cracks due to precipitation of corrosion products or phases such as ettringite or calcite, and that this may on the long-term lead to corrosion kinetics similar to uncracked concrete. The conditions under which self-healing is possible, however, are not yet fully understood. Parameters that have been identified to play a role are the crack width, the type of loading (e.g. static vs. dynamic), the binder type, and the exposure conditions.
4. Based on the current state-of-the-art, no general recommendations can be made for critical crack widths, neither for corrosion initiation, nor for corrosion propagation and for self-healing. The existence of critical crack widths has also been controversially discussed in the literature, amongst other reasons because it is hardly possible to relate the crack width measured at the concrete surface to the width at the reinforcement steel.
5. There is experimental evidence supporting the corrosion mechanism in cracked concrete proposed by Schiessl and Raupach [7, 22], namely that the anodic iron dissolution occurs at the location of the crack (sustained by easier supply of chloride), while the cathodic partial reaction occurs on the passive steel surface between the cracks. However, experimental evidence from literature showing that the location of pits does not always correspond to the location of the cracks means that pits initiate and grow at the weakest point at the interface steel/concrete, and that this must not necessarily be at cracks.
6. The proposed corrosion mechanism of anodes at the cracks and cathodic zones between the cracks implies that the corrosion rate is controlled by the quality of the concrete between the cracks and by the concrete cover thickness. Decisive parameters are the electrical resistivity and the oxygen permeability. A further implication of this concerns the crack frequency, namely that an increasing crack frequency leads to a decreasing corrosion rate at the local anodic site at the crack. There is, however, very little experimental data in the literature confirming this suggested effect of the crack frequency.
7. The current state-of-the-art suggests that the effect of cracks on corrosion diminishes with time and may on the long-term be negligible. Despite this, the vast majority of studies were performed over relatively short times (< 2 y). The conclusions that can be drawn from these investigations may thus have little relevance for practice. Additionally, one of the main shortcomings of the few long-term studies currently available is that the time

evolution of corrosion initiation and propagation is not well documented, primarily due to a lack of continuous, non-destructive measurements.

8. There is a clear need for further research on the effect of cracks and on the long-term behaviour (> 10 y) of reinforced concrete in chloride exposure. It is recommended to perform field tests with specimens of realistic dimensions and to use materials representative for infrastructure construction. Parameters that need to receive particular attention are the crack frequency, the shape of cracks (bending vs. tension), the loading conditions (constantly loaded, unloaded, and dynamic), and the conditions under which self-healing is possible. With respect to measurement methods it is suggested to make use of electrochemical techniques and modern instrumentation that permits continuous data logging (monitoring).

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Appendix

Table 1 Specimen details for reviewed publications (abbreviations see Table 7)

Reference	Year	Specimen			
		No.	Dimensions LxWxH (mm)	Cover (mm)	Notes
Tremper [53]	1947	64 tot. 2 per config.	200x200x64	31	-
Rehm & Moll [18]	1964	36 tot. 6 per config.	1950x250x150	23	some damaged specimens
Atimtay & Ferguson [56, 59]	1973	18 flex. beams, 15 unloaded slabs	7000xNRxNR	25, 51	-
Fidjestøl & Nilsen [57, 67]	1980	70 tot.	NR	20, 50	-
Makita et al. [46]	1980	23 tot. 1 per config.	L = 1500, 750 diff. widths/heights	20, 70, 120	-
O'Neil [43]	1980	82 tot. in general 2 per config.	NR	19, 50	only 13 were in testable condition
Okada & Miyagawa [32]	1980	NR	900x50x50	20	-
Vennesland & Gjørv [33]	1981	8 tot.	500x100x100	45	sealed, only cracks left open
Suzuki et al. [66]	1990	43 tot. min. 2 per config.	700x160x140 600x140x120	20, 30, 60	-
Misra & Uomoto [47]	1991	6 tot. for Series III 1 per config.	2100x200x100	10	Series III: 4 uncracked, 2 cracked
Ohta [19]	1991	149 pairs of beams	1000x150x150	20, 30, 40, 50, 68	-
Berke et al. [48]	1993	20 tot. 4 per config.	762x152x152	25, 38	-
Sakai & Sasaki [51]	1994	16 tot.	1000x150x150	20, 30	-
Lorentz & French [34]	1995	96 tot. 3 per config.	1200x300x180	25	3 config. with normal uncoated rebars
Arya & Ofori-Darko [30]	1996	28 tot. 4 per config.	1360x135x100	42	-
Coppola et al. [49]	1996	NR	400x150x100	30	noched area has a cover of 20mm
Pettersson et al. [68]	1996	25 tot.	1000x300x80	15, 30	-
Ohno et al. [61]	1996	NR	450x93x111, 131	20, 40	-
Schiessl & Raupach [22]	1997	NR	700x150x97	15, 35	lateral surfaces sealed, embedded cathode segments
Ramm &Biscoping [23]	1998	42 tot.	700x180x180 600x300x300 600x600x300	NR	-
Francois & Arliguie [20, 21]	1999	68 tot.	3000x280x150	40, 10	-
Li [52]	2000	model: 255 tot. full-size: 16 tot.	model: NRx62x62 and NRx82x82, full-size: NRx200x120	model: 25, 35 full-size: 31	-
Mohammed et al. [25, 26]	2001	36 tot. 3 per config.	400x100x100	45	-
Mohammed et al. [26]	2001	6 tot. 3 per config.	1250x150x150	25	-
Mohammed et al. [25, 41, 42]	2001	36 tot. 3 per config.	600x100x100	46	one specimen is missing (only 35 tested)
Marcotte & Hansson [27]	2003	4 per mix	500x100x100	NR	-
Bi & Subramaniam [28]	2006	NR	1219xNRxNR	25	-
Hiraishi [69]	2006	NR	NRx100x100	50	-
Scott & Alexander [55]	2007	66 cracked sp. NR uncracked sp. 3 per config.	375x120x120	20, 40	-
Jaffer & Hansson [54]	2008	36 tot. 8/2 per config.	1200x120x70	29	-
Otieno et al. [35, 36]	2009	48 tot. 3 per config.	500x100x100	40	-
Paradis [65]	2009	20 tot. 10 per config.	312x100x100	20	-
Paradis [65]	2009	12 tot. 1 per config.	U-shaped h: 1220x100x89 v: 1018x150x89	20	
Jimenez-Quéro et al. [60]	2010	40 tot. 2 per config.	300x200x55	20	-
Kishimoto [31]	2010	NR	NRx100x100	NR	-
Pease et al. [24]	2010	8 tot. 1 per config.	600x100x100	24, 40	-
Tian & Schiessl [29]	2010	28 tot. 0-4 per config.	500x200x150	20, 35, 50	-
Sangoju et al. [40]	2011	NR	U-shaped h: 600x100x100 v: 200x150x100	20	strongly accelerated corrosion by impressed voltage (10V)
Otieno et al. [37, 38]	2012	210 tot.	375x130x120	20, 40	-
Francois et al. [50]	2012	25 tot. 10 (2008), 15 (2009)	ring shaped øa=150, øi=50 h=50	21, 22	-
Ahmadi et al. [39]	2014	36 tot. 2 per config.	600x150x150	30, 50, 75	12 uncracked control samples

Table 2 Material specifications for unsegmented steels (abbreviations see Table 7)

Reference	Year	Concrete			Curing		Steel		
		Binder type	w/b	d_{max} (mm)	Time (d)	Type	Type	ø (mm)	Length (mm)
Tremper [53]	1947	NR	0.4, 0.60, 0.75	< 19	14	moist curing	diff. wires	NR	NR
Rehm & Moll [18]	1964	OPC	0.8	15	7	Covered with wet towels	RIB, SM, CL	8	NR
Atimtay & Ferguson [56, 59]	1973	OPC	0.49, 0.55, 0.62	38	NR	NR	tension bars	6, 8, 11	NR
Fidjestøl & Nilsen [57, 67]	1980	NR	NR	NR	NR	NR	RIB, CL	16, 25, 32	NR
Makita et al. [46]	1980	OPC	0.55, 0.7, 0.5, 0.4	25	NR	NR	Black steel	16	610, 1360
O'Neil [43]	1980	NR	NR	NR	NR	NR	different	16, 19	NR
Okada & Miyagawa [32]	1980	NR	0.4, 0.5, 0.6, 0.7	10	NR	NR	NR	NR	NR
Vennesland & Gjørv [33]	1981	OPC	0.5	16	56	Water, 20°C	RIB	10	400
Suzuki et al. [66]	1990	OPC	0.35, 0.45, 0.55	10	28	wet cured	RIB	13	NR
Misra & Uomoto [47]	1991	NR	0.55	NR	14	under wet gunny bags	NR	16	NR
Ohta [19]	1991	OPC, FA, GGBS	0.41...0.53	25	NR	NR	RIB, CL	13	NR
Berke et al. [48]	1993	NR	NR	NR	28	fog room	Black steel	13	NR
Sakai & Sasaki [51]	1994	OPC	0.32…0.48	25	7+21	moist-cured + NR	RIB, UC, EC	13	940
Lorentz & French [34]	1995	OPC, cSF	0.35	NR	28	NR	UC, EC	13	NR
Arya & Ofori-Darko [30]	1996	OPC	0.65	10	28	RH90%, 20°C	2x stainless steel, 1x SM	8	1340
Coppola et al. [49]	1996	OPC, FA, SF, PIC	0.35, 0.5, 0.6	19	7+45	Plastic sheets + air cured at 20°C	Steel plate 320x130x1	NR	NR
Pettersson et al. [68]	1996	SRPC, 5-15% MSF	0.3, 0.4	16	5+NR	RH100%, 20°C + outside sheltered	NR	12	1000
Francois & Arliguie [20, 21]	1999	OPC HP	0.5	15	NR	NR	RIB	16	NR
Li [52]	2000	OPC, FA, BFS	0.45, 0.6	NR	NR	NR	NR	NR	NR
Mohammed et al. [26]	2001	NR	0.5, 0.7	20	28	wet jute bags	SM, RIB	13	NR
Mohammed et al. [25, 41, 42]	2001	OPC, FA, GGBS	0.45, 0.55	20	28	standard curing	SM	9	500
Scott & Alexander [55]	2007	OPC, FA, GGBS, SF	0.58	9	14	Water, 23°C	Cl, SM	16	315
Jaffer & Hansson [54]	2008	OPC, GGBS, SF	0.46, 0.35	12	OPC: 2 HPC: 7	wet curing	RIB	11	1200
Otieno et al. [35, 36]	2009	OPC, BFS	0.4, 0.55	19	28+10	submerged + dry air	CL	10	NR
Paradis [65]	2009	OPC, SF	0.49	sand	120	submerged	RIB, CL	15	100
Paradis [65]	2009	OPC, SF	0.49	sand	90	submerged	NR	15	NR
Jimenez-Quéro et al. [60]	2010	OPC	0.3, 0.4, 0.5, 0.6	13	28	RH 95%, 23°C	NR	NR	NR
Kishimoto [31]	2010	OPC	0.54	NR	NR	NR	RIB	22	NR
Sangoju et al. [40]	2011	OPC, PPC	0.37, 0.47, 0.57	20	28+30	submerged + air	RIB	12	NR
Otieno et al. [37, 38]	2012	OPC, SF, GGBS, FA	0.40, 0.55	13	28	submerged	RIB	10	NR
Francois et al. [50]	2012	OPC	0.48	2	NR	NR	RIB	6, 8	341
Ahmadi et al. [39]	2014	OPC, SF	0.40	19	1+28+84	plastic sheet + submerged + air	CL	10	NR

Table 3 Material specifications for segmented steels (abbreviations see Table 7)

Reference	Year	Concrete			Curing		Steel		
		Binder type	w/b	d_{max} (mm)	Time (d)	Type	Type	ø (mm)	segmentation
Ohno et al. [61]	1996	OPC	0.3, 0.5, 0.7	10	NR	NR	SM	14	48mm long SM, which ends (each side 5mm) were EC, 2 150mm long stainless steel rods placed as cathode, 13mm ø steel bar (EC) as reinforcement
Schiessl & Raupach [22]	1997	OPC	0.5, 0.6	NR	28	NR	NR	14	steel bar EC except 20mm in central section, 3 reinforcing steel sections (75mm) on each side of the crack
Ramm & Biscoping [23]	1998	OPC	0.55	16	NR	NR	NR	16	continuous steel bar in the centre, three steel segments on each side of the crack
Mohammed et al. [25, 26]	2001	OPC	0.3, 0.5, 0.7	sand	28	RH 80%, 20°C	SM, RIB	NR	steel segments (7x45mm) were connected with epoxy for electrical isolation, EC steel bar as dummy
Marcotte & Hansson [27]	2003	HPC, SF	0.25, 0.27	10x5	14+84	plastic sheets + outdoor	NR	15	conventional rebar as backbone, overlapping position of 5x30mm steel bars with at least 20mm distance to its neighbouring element
Bi & Subramaniam [28]	2006	OPC	0.45	NR	90	RH 100%	CL	13	central segment 51mm, one more (533mm) at each side, electrically isolated along their length
Hiraishi [69]	2006	OPC	0.56	20	NR	NR	SM	9	3 segments (10mm) and 2 segments (50mm) at the end of the steel bar, all segments connected with epoxy-resin
Pease et al. [24]	2010	PC	0.5	8	1	plastic sheet	RIB	10	17 1.5mm holes spaced 11mm apart for sensors
Tian & Schiessl [29]	2010	OPC, GGBS	0.4, 0.5	16	7-14, 180	RH 85%, 20°C	NR	12	EC steel except 4mm in cracked section, parallel 3x50mm and 1x150mm without electrical contact

Table 4 Information about cracks for reviewed publications (abbreviations see Table 7)

Reference	Year	Cracks					
		No.	Cracking	Crack width (mm)	Fixing	Uncracked specimen	Remarks
Tremper [53]	1947	single	3PB	0.13, 025, 0.51, 1.27	brass screws and strips	yes	-
Rehm & Moll [18]	1964	multi	4PB	0.2-0.3	two beams yoked together	no	-
Atimtay & Ferguson [56, 59]	1973	multi	4PB	NR	NR	no	-
Fidjestøl & Nilsen [57, 67]	1980	multi	4PB	0…0.5	two beams yoked together	yes	-
Makita et al. [46]	1980	multi	NR	> 0 ... 1.0	NR (no)	yes	-
O'Neil [43]	1980	multi	3PB	NR	two beams yoked together	yes	Different load levels -> different crack width
Okada & Miyagawa [32]	1980	multi	Tension	0;0.1-0.2;0.2-0.3	NR	yes	-
Vennesland & Gjørv [33]	1981	single	3PB	0.1...2.0	NR	no	-
Suzuki et al. [66]	1990	single, multi	3PB, 4PB	NR	NR	yes	-
Misra & Uomoto [47]	1991	multi	4PB	NR	no	yes	-
Ohta [19]	1991	multi	NR	“Reported”	NR	no	-
Berke et al. [48]	1993	single	3PB	0.2-0.31	shims	NR	-
Sakai & Sasaki [51]	1994	NR	4PB	0.2	two beams yoked together	no	-
Lorentz & French [34]	1995	NR	4PB	0.25-0.60	no	yes	-
Arya & Ofori-Darko [30]	1996	multi	shims	0.12...2.4	no	yes	0, 1, 4, 8, 12, 16, 20 cracks (sum of all crack widths per specimen = 2.4 mm)
Coppola et al. [49]	1996	single	3PB	0, 0.03, 0.3, 1.0	no	yes	-
Pettersson et al. [68]	1996	multi	4PB	0.4, 0.8	sustained load	NR	Aim: single crack, but multi cracks resulted, sealing: NR
Ohno et al. [61]	1996	single	3PB	NR	yes	yes	-
Schiessl & Raupach [22]	1997	single	3PB	0.1…0.5	sustained load	NR	crack initiator with plastic strip
Ramm &Biscoping [23]	1998	single	shims	0.1, 0.2, 0.3, 0.4	NR	NR	-
Francois & Arliguie [20, 21]	1999	multi	3PB	0.05-0.2 0.05-0.5	sustained load	yes	2 different loading levels
Li [52]	2000	NR	3PB	0.05,0.1, 0.2	sustained and released load	yes	sustained load, loaded to crack and released, unloaded
Mohammed et al. [25, 26]	2001	single	3PB	0.1, 0.3, 0.7	shims	yes	-
Mohammed et al. [26]	2001	multi	4PB	0.1...0.4	two beams yoked together	NR	-
Mohammed et al. [25, 41, 42]	2001	single	3PB	0.1...5.0	no	no	-
Marcotte & Hansson [27]	2003	single	3PB	0.3	stabilized	yes	-
Bi & Subramaniam [28]	2006	single	3PB	NR	NR	NR	-
Hiraishi [69]	2006	single	3PB	NR	NR	NR	-
Scott & Alexander [55]	2007	single	3PB	0.2, 0.7	NR	yes	-
Jaffer & Hansson [54]	2008	multi	3PB	NR	two beams yoked together	yes	8 static, 8 dynamic, 2 unloaded
Otieno et al. [35, 36]	2009	single	3PB	0.4, 0.7, incipient	remained in loading rigs	NR	unwanted cracks were sealed + reloading in week 9/19
Paradis [65]	2009	single	shims	0.3	NR (no)	no	-
Paradis [65]	2009	multi	shims, spez.	0.3	sustained and released load	yes	50-70 mm distance between shims
Jimenez-Quéro et al. [60]	2010	single	rubber sheet	0, 0.1, 0.2, 0.3, 0.4	no	yes	-
Kishimoto [31]	2010	single	tension	0.1, 0.3, 0.5	shims	no	-
Pease et al. [24]	2010	single	3PB	0.3…1.2	sustained load	NR	-
Tian & Schiessl [29]	2010	single	tension	NR	NR	NR	-
Sangoju et al. [40]	2011	multi	spez.	0, 0.2, 0.4	yes	NR	stressing and fixing the tie rods with nuts
Otieno et al. [37, 38]	2012	single	3PB	0, 0.4, 0.7	remained in loading rigs	yes	-
Francois et al. [50]	2012	multi	hoop stress	0.025…0.946	no	NR	-
Ahmadi et al. [39]	2014	single	3PB	0.09...0.55	two beams yoked together	yes	-

Table 5 Used measurement methods and exposure conditions (abbreviations see Table 7)

Reference	Year	Measurements		Exposure	
		Type	Frequency	Duration (a)	Environment
Tremper [53]	1947	VI, CA, PD	NR	10	marine environment
Rehm & Moll [18]	1964	VI, CA	annually	1, 2	tidal zone, (exposure in industry & city)
Atimtay & Ferguson [56, 59]	1973	VI, CA	after 19 months	1.6, 2	daily spraying of NaCl-solution
Fidjestøl & Nilsen [57, 67]	1980	VI, E, PDP	approx. every 90 days	10	submerged in natural seawater
Makita et al. [46]	1980	VI, CA, E, MC, ER, ClC	NR	2.7	tidal and air exposure at seaside
O'Neil [43]	1980	VI	annually	25	twice daily tidal cycles, freeze-thaw in winter
Okada & Miyagawa [32]	1980	E, MC, ER	2h, 1d, 1w, 1m, 3m	0.25	spraying NaCl-solution
Vennesland & Gjørv [33]	1981	VI, E, LPR, MC (galvanic current)	daily, VI after 4m	0.33	submerged art. seawater, external cathode
Suzuki et al. [66]	1990	HCP, LPR, WL, CA	every 10 days	0.5	cyclic wetting with NaCl-solution
Misra & Uomoto [47]	1991	VI, CA, E	after 1, 3, 6 months	1, 2	marine environment
Ohta [19]	1991	VI, CSL, E, ClC	NR	20	seaside air exposure
Berke et al. [48]	1993	VI, CA, E, LPR, MC, ClC	monthly	1.33	cyclic ponding NaCl-solution
Sakai & Sasaki [51]	1994	ClC, HCP	NR, after 5, 10 y	5, 10, 20	seaside air exposure
Lorentz & French [34]	1995	VI, E, MC, ER, HCP	continuous	0.77	cyclic ponding saltwater-solution
Arya & Ofori-Darko [30]	1996	LPR (WL), Galvanic current (WL)	monthly	2	cyclic spraying
Coppola et al. [49]	1996	VI (Microscope), E, ER	every 6 months	2	submerged nat. seawater
Pettersson et al. [68]	1996	VI, CA, PD, LPR, ER, HCP, ClC	1.) monthly 2.) twice weekly	1.06	1.) partially submerged in marine environment 2.) laboratory NaCl-solution
Ohno et al. [61]	1996	HCP, LPR, WL, CA	every 10 days	0.38	cyclic ponding in NaCl-solution
Schiessl & Raupach [22]	1997	MC	continuous	2	cyclic ponding in NaCl-solution
Ramm &Biscoping [23]	1998	MC	every 2 weeks	2	varying pH (7, 6.1, 5.2) and pressures, no exposure to chloride environment
Francois & Arliguie [20, 21]	1999	VI, E, ClC	NR	12 (27)	cyclic and permanent salt fog
Li [52]	2000	HCP, LPR, ClC, VI	after each cycle	NR	cyclic spraying salt solution
Mohammed et al. [25, 26]	2001	MC, LPR, oxygen permeability, VI, WL	NR	0.25	cyclic spraying salt solution
Mohammed et al. [26]	2001	MC, LPR, oxygen permeability, LPR, Rho, ClC, VI	NR	1.33	cyclic salt solution spraying
Mohammed et al. [25, 41, 42]	2001	WL, LPR, ClC, MC, VI	after 15 years	15	tidal seawater pool
Marcotte & Hansson [27]	2003	LPR, E	every 2 months	4	partially immersed in sea water
Bi & Subramaniam [28]	2006	E, LPR, MC, HCP	NR	1.33	cyclic wet sponge with NaCl-solution
Hiraishi [69]	2006	MC	continuous	< 0.01	varying T, wind, moisture and sprayed salt-solution
Scott & Alexander [55]	2007	LPR, E, Rho	continuous	1.5	cyclic ponding NaCl-solution
Jaffer & Hansson [54]	2008	LPR, PDP, MC with electrochemical noise measurements, ER, E	monthly	1.5	partially submerged in salt solution
Otieno et al. [35, 36]	2009	E, LPR, Rho, HCP	weekly	0.6	cyclic ponding NaCl-solution
Paradis [65]	2009	E, VI	every 2 days	1.4	cyclic fog NaCl-solution
Paradis [65]	2009	E, VI	every 2 days	1.4	cyclic fog NaCl-solution
Jimenez-Quéro et al. [60]	2010	VI, E, LPR	annually	5	marine environment
Kishimoto [31]	2010	VI, ClC	every 3 weeks	0.06, 0.12, 0.17	1. cyclic submerged in Cl-solution at 60°C 2. cyclic spraying salt-solution at 60°C
Pease et al. [24]	2010	E, ClC, VI	daily	0.17	ponding in NaCl-solution in cracked region
Tian & Schiessl [29]	2010	E, Rho, corrosion current	daily	2	cyclic spraying NaCl-solution, cyclic spraying tap water, outdoor
Sangoju et al. [40]	2011	E, Rho, charge passed, WL, HCP	every 2 days	0.06	submerged in NaCl-solution, impressed voltage (10 V)
Otieno et al. [37, 38]	2012	E, LPR, HCP, ER	weekly	0.6	cyclic ponding NaCl-solution, external current
Francois et al. [50]	2012	VI, LPR	NR	2.19	cyclic submerged in NaCl-solution
Ahmadi et al. [39]	2014	Rho, MC, E, HCP	1, 2, 8, 14 months	0.01, 0.2, 0.7, 1.2	outdoor tidal zone

Table 6 Summary of conclusions drawn by the authors (abbreviations see Table 7)

Reference	Year	Do cracks shorten the initiation stage?	Do cracks enhance the corrosion rate during propagation?	Evidence for self-healing?	Critical crack width?	Remarks
Tremper [53]	1947	NR	no	yes	$w_{crit,heal}$ = 0.12 mm	[a]
Rehm & Moll [18]	1964	yes	yes	NR	$w_{crit,ini}$ = 0.1 mm	[c]
Atimtay & Ferguson [56, 59]	1973	NR	no	NR	NR	-
Fidjestøl & Nilsen [57, 67]	1980	minor importance	NR	yes	NR	[a]
Makita et al. [46]	1980	NR	yes	NR	$w_{crit,prop}$ = 0.1 mm	[d]
O'Neil [43]	1980	yes	yes	NR	$w_{crit,ini}$ = 0.4 mm	[d]
Okada & Miyagawa [32]	1980	yes	yes	NR	$w_{crit,ini}$ = 0.1–0.2 mm	[d]
Vennesland & Gjørv [33]	1981	NR	NR	yes	$w_{crit,ini}$ = 0.1 mm	[a], [c], $w_{crit,heal}$ = 0.4 mm
Suzuki et al. [66]	1990	yes	no	NR	NR	-
Misra & Uomoto [47]	1991	yes	no	NR	$w_{crit,ini}$ = 0.5 mm	[d]
Ohta [19]	1991	NR	(no)	NR	NR	-
Berke et al. [48]	1993	NR	NR	NR	NR	-
Sakai & Sasaki [51]	1994	yes	NR	NR	NR	leaching out at wider cracks
Lorentz & French [34]	1995	yes	yes	NR	NR	pronounced increase of corrosion current densities for cracked specimens compared to uncracked specimens
Arya & Ofori-Darko [30]	1996	NR	yes	yes	$w_{crit,heal}$ = 0.12 mm	[b], [c]
Coppola et al. [49]	1996	yes	NR	NR	$w_{crit,ini}$ = 0.2 mm	[d]
Pettersson et al. [68]	1996	NR	NR	NR	NR	-
Ohno et al. [61]	1996	yes	NR	NR	NR	-
Schiessl & Raupach [22]	1997	NR	no	NR	NR	-
Ramm &Biscoping [23]	1998	yes	yes	yes	$w_{crit,ini}$ = 0.1 mm	[b], [d]
Francois & Arliguie [20, 21]	1999	yes	no	NR	$w_{crit,prop}$ = 0.5 mm	[c], investigated: w_{cr} < 0.5 mm
Li [52]	2000	yes	NR	yes	$w_{crit,ini}$ < 0.1 mm with sustained load; $w_{crit,ini}$ >0.1 mm for all loading conditions	[b], [c], with sustained load less self-healing
Mohammed et al. [25, 26]	2001	yes	no	NR	does not exist	-
Mohammed et al. [26]	2001	yes	no	NR	does not exist	-
Mohammed et al. [25, 41, 42]	2001	NR	yes	yes	NR	[a], only observed for $w_{crit,heal}$ < 0.5 mm
Marcotte & Hansson [27]	2003	yes	yes	NR	NR	-
Bi & Subramaniam [28]	2006	NR	yes	NR	NR	-
Hiraishi [69]	2006	NR	NR	NR	NR	-
Scott & Alexander [55]	2007	NR	yes	NR	NR	increase of corrosion rate by up to 75% by increasing w_{cr} from 0.2 mm to 0.7 mm
Jaffer & Hansson [54]	2008	yes	yes	NR	NR	-
Otieno et al. [35, 36]	2009	yes	yes	“not measured”	does not exist	[c], increase of corrosion rate by up to 210% caused by the presence of cracks
Paradis [65]	2009	yes	yes, moderately	NR	NR	-
Paradis [65]	2009	yes	NR	yes	NR	[b]
Jimenez-Quéro et al. [60]	2010	NR	yes, for w/b=0.3/0.4 no, for w/b=0.5/0.6	NR	NR	-
Kishimoto [31]	2010	yes	NR (yes)	NR	NR	“corroded area largest near to crack”
Pease et al. [24]	2010	NR	NR	NR	NR	-
Tian & Schiessl [29]	2010	NR	NR	NR	NR	-
Sangoju et al. [40]	2011	yes	yes	NR	NR	-
Otieno et al. [37, 38]	2012	NR	yes	NR	NR	“corrosion rate increased with increasing crack width”
Francois et al. [50]	2012	yes	yes, but negligible in long-term	NR	does not exist	-
Ahmadi et al. [39]	2014	yes	yes	NR	NR	ratio: crack width/cover

[a] Explicit experimental evidence for self-healing.
[b] No experimental evidence for self-healing, but it was used as explanation why the corrosion rate decreased with time.
[c] The given value for the critical crack width is evident from the original data.
[d] A critical crack width was recommended by authors, but the given value is not clearly evident from the original data.

Table 7 Abbreviations used in Tables 1-6

Abbreviation	Meaning
3PB	three point bending
4PB	four point bending
BFS	blast-furnace slag
CA	corroded area
CL	cleaned / degreased
ClC	Chloride content
cSF	condensed silica fume
CSL	cross section loss
E	potential
EC	epoxy-coated
ER	electrical resistance
FA	fly ash
GGBS	ground granulated blast furnace slag
HCP	half-cell potential
LPR	linear polarisation resistance
MC	macro-cell current
MSF	micro silica fume
NR	not reported
OPC	ordinary Portland cement
OPC HP	ordinary Portland cement high-performance
PD	pit diameter
PDP	potentiodynamic polarisation
PIC	polymer impregnated concrete
PPC	Portland pozzolana cement
Rho	resistivity measurements
RIB	ribbed steel bars
SCM	supplementary cementitious materials
SF	silica fume
SM	smooth steel bars
SRPC	sulphate resistant Portland cement
UC	uncoated
VI	visual inspection
w_{cr}	crack width
$w_{crit,heal}$	Critical crack width for self-healing of cracks
$w_{crit,ini}$	Critical crack width for corrosion initiation
$w_{crit,prop}$	Critical crack width for corrosion propagation
w/b-ratio	water/binder-ratio
WL	weight loss

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