



DAMAGE OF CONCRETE STRUCTURES DUE TO AGING

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1. Introduction

For many decades concrete structures have been designed for predominantly two criteria: structural safety and serviceability. There was no doubt about the service life of structures. The observation of substantial damage to concrete structures developing in time due to deterioration processes became however gradually a point of large concern, predominantly because structural safety of structures turned out to be endangered. In the new *fib*-Model Code for Concrete Structures 2010 design for service life is a central issue. According to this principle structures are designed to satisfy the demands for safety and serviceability for a specified period of time. Structures serving the infrastructure of a country, like tunnels and bridges, are mostly designed for a low maintenance service life of 100 years. In structures like buildings and offices the functional service life is often the governing criterion, so that a physical service life of 50 years is most often required. Nowadays we dispose about considerable knowledge with regard to deterioration processes, and we are able to design our new structures for service life by making sure that we use the right materials, that the concrete cover is sufficient and that the cracks are well controlled. We dispose of adequate quality control procedures for the material concrete and its constituents to avoid unforeseen problems like alkali aggregate reaction.

A problem which is left, however, is that most of the existing structures have not been designed based on service life considerations. The actual structural safety of those structures is a point of concern. If the actual safety turns out to be sufficient, the question is for how long this is guaranteed. That this uncertainty is justified is often illustrated by unexpected and unwanted surprises, like the collapse of gallery slabs in an apartment building in Leeuwarden, the Netherlands, 2012, Fig. 1



Fig. 1. Unannounced collapse of gallery plates in an apartment building in Leeuwarden, The Netherlands, 2013

The collapse turned out to be caused by corroded reinforcement. A remarkable observation was that the structure fulfilled the code criteria of today. There were no cracks and no signs of upcoming problems due to for instance spalling of the concrete cover. That illustrates that obviously our knowledge about deterioration processes is not yet good enough. In this paper it is argued that we meanwhile know a lot about deterioration, but as well that crucial knowledge is missing.

2. Important deterioration mechanisms endangering structural safety in time

2.1. Alkali aggregate reaction

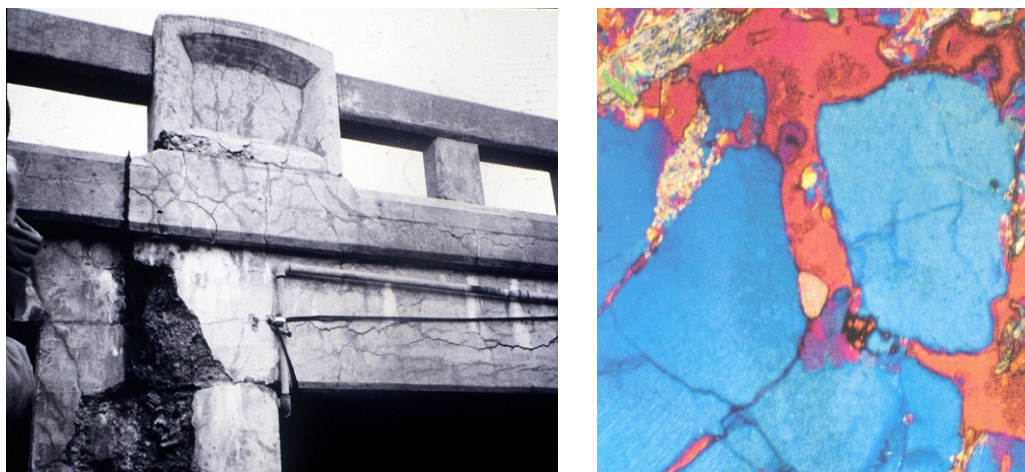


Fig. 2 a. Part of a bridge structure attacked by alkali aggregate reaction, b. Physical reaction leading to expansion of the concrete

Alkali aggregate reaction (AAR) is an internal chemical reaction between the alkaline components in the cement and certain active mineral constituents in some aggregates. AAR reactions are subdivided into alkali silica (ASR) and the alkali-carbonate (ACR) reactions. However, ASR is more widespread, and is more harmful to the mechanical properties of concrete. ASR is an internal chemical reaction between the alkaline components in the cement and active silica-based constituents of some aggregates. The reaction results in the formation of a gel that absorbs water, and therefore exerts internal pressure which sometimes can be far in excess of what concrete can sustain, thereby causing the formation of microcracks. There are three main components necessary to facilitate ASR, namely: sufficient alkalis in the pore solution, a critical amount of reactive material phases in the aggregate particles, and sufficient moisture.

An important phenomenon is the effect of restraint of the concrete swelling by reinforcement. Fig. 2a shows that the cracks in the beam tend to develop in the horizontal direction. The reason is that that in the axial direction the longitudinal reinforcement counteracts the swelling and in this way generates a type of prestressing. In the vertical direction often no or limited reinforcement is present, so that the expansion can develop freely, resulting in horizontal crack formation. These cracks may reduce the shear capacity of the bridge [1].

2.2. Carbonation

Due to carbonation the protective function of the depassivation layer on the reinforcing steel is destroyed due to the penetration of CO_2 which increases the probability of rusting of the reinforcing steel. The three stages in the carbonation process are:

– Diffusion of CO_2 through the capillary pores,

- Dissolution of CO_2 in the water film at the pore wall: $\text{CO}_2 + \text{H}_2\text{O} \rightarrow \text{H}_2\text{CO}_3$.
- Neutralization of $\text{Ca}(\text{OH})_2 + \text{H}_2\text{CO}_3 \rightarrow \text{CaCO}_3 + \text{H}_2\text{O}$,

Due to the carbonation process the pH of the concrete reduces from about 13 to a value of 8–9. If there is sufficient oxygen and moisture the corrosion process can start.

The largest probability that corrosion occurs is in areas which are outside but shut, with limited moisture variations. In that case there is fast CO_2 penetration, but as well supply of water. This situation occurs in the area A in Fig. 3b.

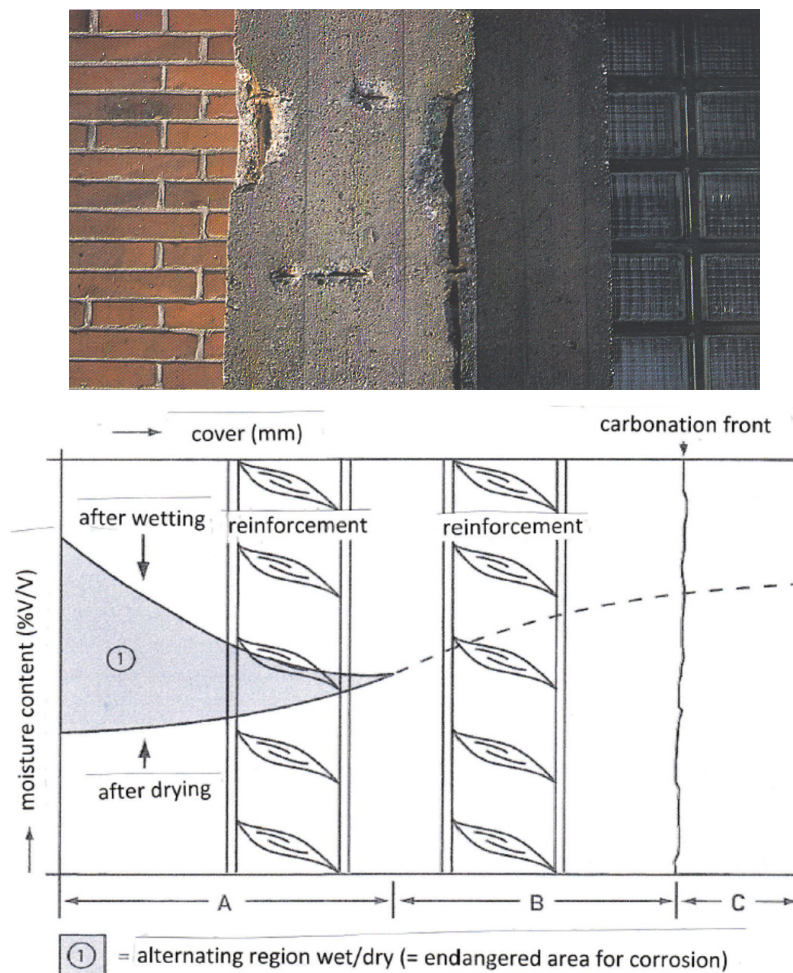


Fig. 3. a) Reinforcement corrosion due to carbonation of concrete in a column, b) Carbonation process leading to corrosion of reinforcing bars

2.3. Chloride penetration

Chloride induced corrosion is by far the most important cause for deterioration and as such reduction of structural safety in time. If the concrete cover is too small and the concrete quality too low, chlorides penetrate until the reinforcement. Corrosion starts as the chloride concentration at the reinforcement has reached a limit value. Fig. 4 shows the result of such a process and a desperate attempt to stop it by applying a coating (Fig. 4 right), which was deemed to fail. A well known representation of the deterioration process as a function of time is shown in Fig. 5. Until the end of the initiation time no damage is assumed to occur, since the corrosion process starts only after the chloride content at the reinforcement has reached a critical value. The events after the start of corrosion (so after reaching point 1 in the diagram in Fig. 5) were considered to be quite uncertain with regard to their timely development.

Moreover the general idea was that the initiation time was substantially longer than the propagation time. As a practical approach the end of the initiation time was therefore defined as the end of service life.

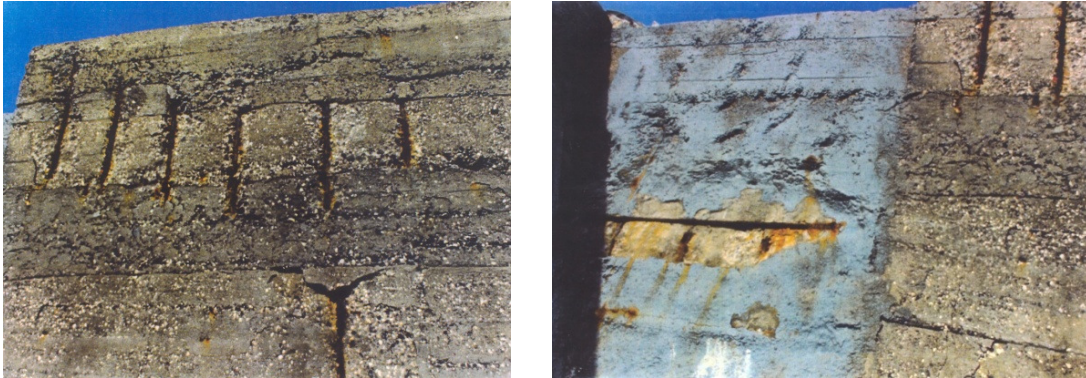


Fig. 4. a) Damage due to chloride induced corrosion in a harbour jetty in Sicily, Italy, b) Unsuitable solution with a coating

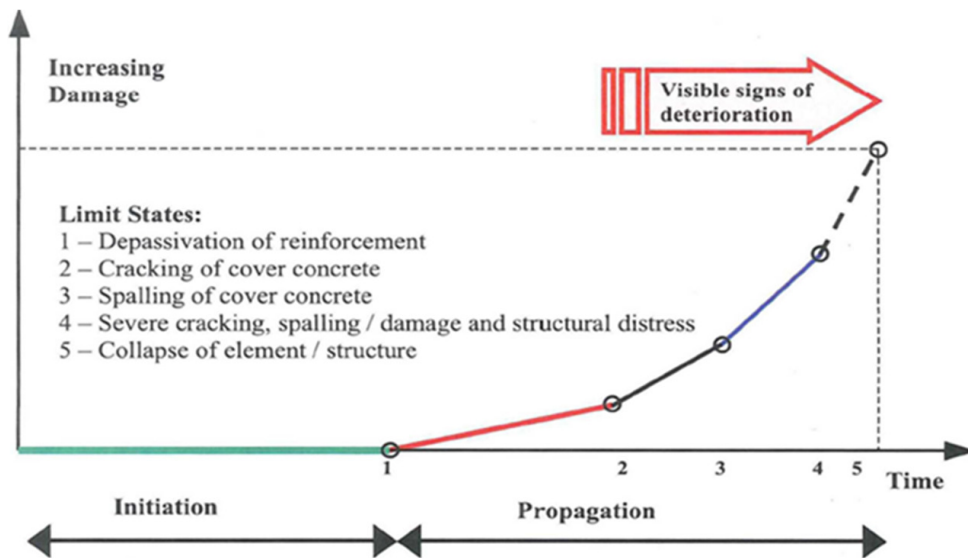


Fig. 5. Deterioration due to chlorides in reinforced concrete

3. Development of a probabilistic approach to service life prediction

3.1. General basis

It was mentioned before that in the fib Model Code for concrete structures 2010 service life was integrated in design, by requiring that the a defined structural safety and serviceability should be maintained for a defined period. For the case of chloride ingress that means that it should be determined with which probability the critical chloride content is not reached in that time. This led to the requirement that the following limit state function must be fulfilled:

$$P\{\} = P_{dep} = P\{C_{crit} - C(c, t_{SL}) < 0\} < P_0 \quad (1)$$

where:

$P\{\}$ – is the probability that passivation occurs,
 C_{crit} – is the critical chloride content to achieve depassivation of the reinforcement [wt.-% /binder content],

$C(c, t_{SL})$ – is the chloride content at depth c and time t [wt.-%/binder content],
 C – is the concrete cover [mm],
 t_{SL} – is the service life [years],
 P – is the target failure probability.

The variables c , C_{crit} and $C(C, t_{SL})$ must be quantified in a probabilistic approach.

Moreover the ingress of chlorides in a marine environment may be assumed to obey Fick's second law of diffusion.

$$C(x, t) = C_S - (C_S - C_i) \cdot \left[\operatorname{erf} \frac{x}{2 \cdot \sqrt{D_{app}(t) \cdot t}} \right] \quad (2)$$

where:

$C(x, t)$ – is the content of chlorides in the concrete at a depth x (structure surface: $x = 0$ m) and time t [wt.-%/binder content],
 C_S – is the chloride content at the concrete surface [wt.-%/binder content],
 C_i – is the initial chloride content of the concrete [wt.-% binder content],
 x – is the depth with a corresponding content of chlorides $C(x, t)$ [mm],
 D_{app} – is the apparent coefficient of chloride diffusion through concrete [m²/sec.] at time t ,
 t – is the time (years) of exposure,
 erf – error function.

$$D_{app}(t) = D_{app}(t_0) \left(\frac{t_0}{t} \right)^\alpha$$

where:

$D_{app}(t_0)$ – is the apparent diffusion coefficient measured at a reference time t_0 is the aging factor giving the decrease over time of the apparent diffusion coefficient. Depending on the type of binder and the micro environmental conditions the ageing factor is likely to be between 0,2 and 0,8.

3.2. Trial calculation

A trial calculation of the probabilistic determination of the deterioration of a structure was made based on [2]. The calculation was made for a bridge in the Netherlands, with 54 beams with a length of 20 m. Fig. 6 shows a top view of the bridge.

The assumptions for the most important parameters in the analysis are given in Table 1.

Fig. 7. shows the results of the calculation for the deterioration progress. It shows that there is a substantial difference between the lower bound and the upper bound of the prediction, due to the uncertainties involved. The calculation was carried out starting in 1985, the year of construction. The calculation could be calibrated based on the results of an inspection in 2002. This calibration made it possible to determine which curve fits to the bridge considered. On this basis the behaviour of the bridge for the following years could be predicted and decisions could be made with regard to the maintenance strategy.

Table 1. Properties assumed in trial calculation for chloride induced deterioration development [2]

Variable	Distribution	Mean	Stand. Dev.	Ref.
$C_{Cl,s}$	Normal	0,89%	0,37%	Tests
$C_{Cl,cr}$	Normal	0,48%	0,15%	Duracrete
D_{Cl}	Normal	$2,7 \times 10^{-5}$ (m ² /year)	$1,1 \times 10^{-5}$ m ² /year	Tests
c	Lognormal	0,032 m	0,01 m	Measurements

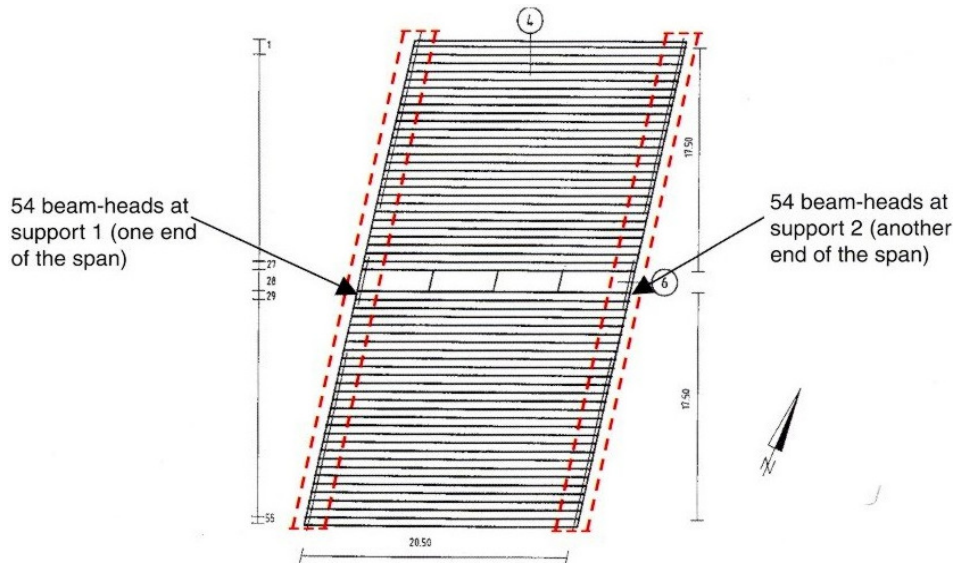


Fig. 6. Top view of bridge Wilpsedijk

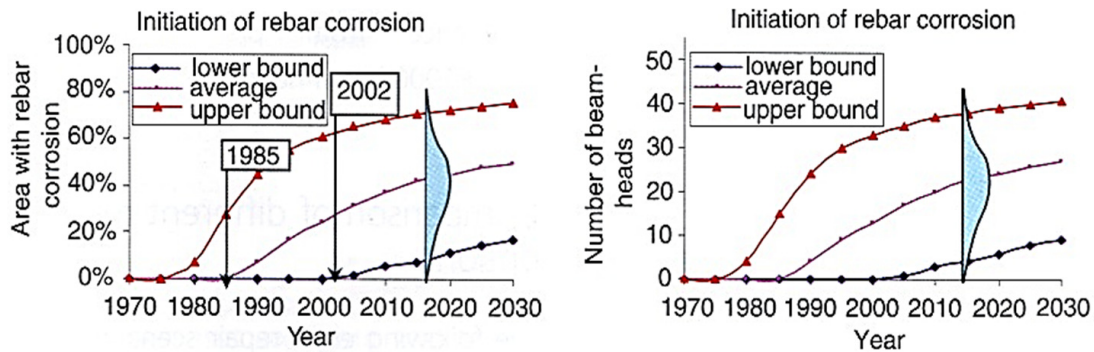


Fig. 7. Prediction of area of rebar corrosion and number of damaged beam heads [2]

In the previous chapter it was shown that a scientific basis was developed for the prediction of the deterioration process. This model, however, has to be handled with care. As an example the determination of the coefficient of diffusion with the Rapid Chloride Migration test is a simplification of reality. It is to some extent opportunistic to predict the service life of a structure (e.g. 100 years) on the basis of accelerated tests on young concrete, lasting only a few months. Moreover it should be noted that close to none of the cements on the market today have 50 or 100 years of records. Therefore the use of probabilistic methods for new structures at this stage of development is doubtful. On the other hand a full probabilistic approach, might be very well suited to extrapolate long-term performance based on the assessments of medium-term field-exposed structures. Then the influence of uncertainties in data and models will be reduced dramatically.

3.3. The critical crack width: an anomaly in modeling:

It is remarkable that in the model explained before the role of crack width is not recognized. In our codes like EN 1992-1-1 limit values for the crack width are given related to the environmental classes. For reinforced concrete structures the maximum crack width should be determined under the quasi-permanent load. For prestressed structures with bonded prestressing steel, the frequent load combination has to be adopted. On the other hand, in the Model Code for Service Life Design [3], only half a page of the totally 110 pages deals marginally with the role of crack width on service life. No indications of limit values are given. The same holds true for

the *fib* Model Code 2010. In CEB Bulletin 182 the following sentence about the role of crack width is found: “The influence of crack width on the corrosion rate at the reinforcement has turned out to be relatively small within the common range of crack width (up to 0.4 mm). Self-healing as a result of calcium-, dirt- and rust-deposits within the cracks, can be frequently observed. In this case, any ongoing corrosion at the reinforcement is likely to come to a halt”. It is remarkable that the role of crack width on service life is not better clarified. In order to get a better understanding of the role of crack width for the service life of reinforced concrete structures, at TU Delft a series of tests was carried out, concentrating on the role of crack width on the development of corrosion. Altogether 32 concrete beams (1500×100×150 mm) were tested, see fig. 8 and 9. Parameters were the mean crack width, the number of cracks, the concrete cover and the type of loading (static versus variable). The cracked reinforced beams were exposed to alternately wetting and drying cycles once a week for 2 days ponding using a 3,5% NaCl solution and a 5 days drying phase to simulate an aggressive environment. The half-cell potential and linear polarization resistance method were applied for assessing the corrosion of the steel reinforcement.

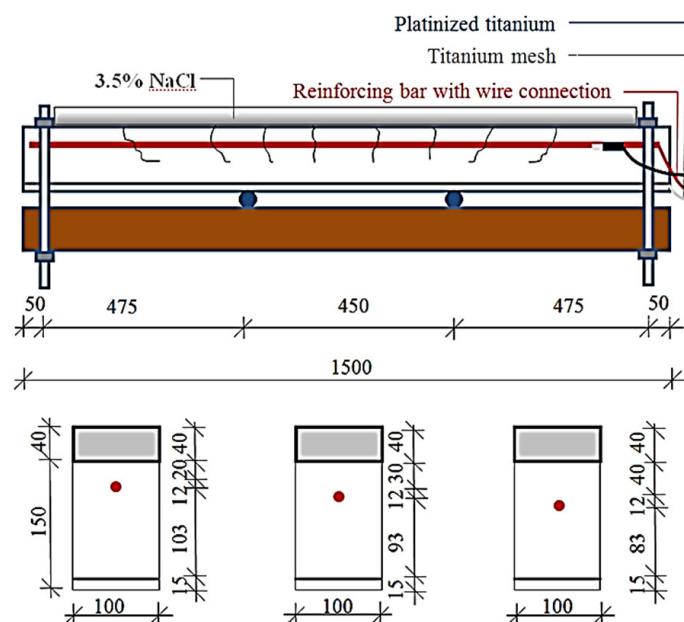


Fig. 8. Test specimens for the determination of the role of crack width on durability carried out at TU Delft

In the following some of the results are given. In Table 2 the parameters are given of the beams, the behaviour of which is compared in Fig. 9.



Fig. 9. PhD researcher Andrija Blagojevic in front of some of his 32 beams

Table 2. Data for beams, the corrosion development of which is compared in Fig. 9

Series	Concrete cover (mm)	Mean crack width (mm)	Number of cracks	Type of loading
14B	40 mm	0.15–0.25	6	Variable
15B	40 mm	0.25	6	Static
16A	40 mm	0.20	6	Static
15A	40 mm	0.15	6	Static

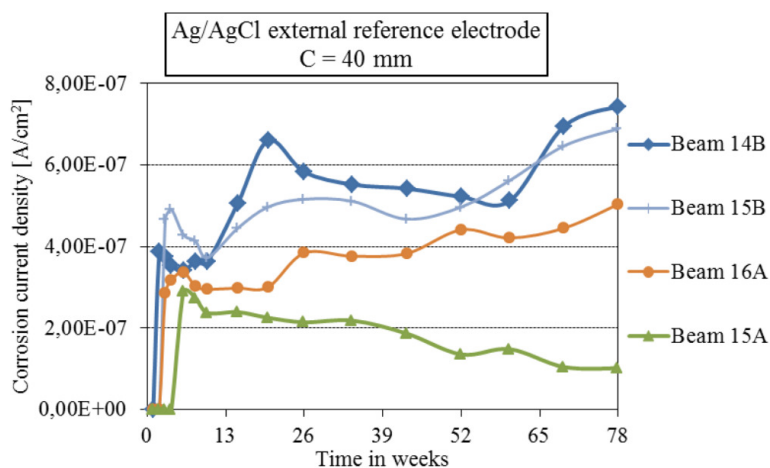


Fig. 10. Corrosion current density as a function of time in weeks for beams with various crack widths

The diagram shows that there is clear tendency of a higher corrosion current density with increasing average crack width. Another interesting observation is made in Fig. 11, where the behaviour of two beams is compared with similar geometry. Beam 6A has a mean crack width of 0.15 mm and is subjected to a static load. Also beam 7A has a mean value of 0.15 mm, but the crack width varies between 0.10 mm and 0.20 mm due to the variable load.

Table 3. Data for beams, the corrosion development of which is compared in Fig. 11

Series	Concrete cover (mm)	Mean crack with (mm)	Number of cracks	Type of loading
6A	30	0.15	8	static
7A	30	0.10–0.20	8	variable

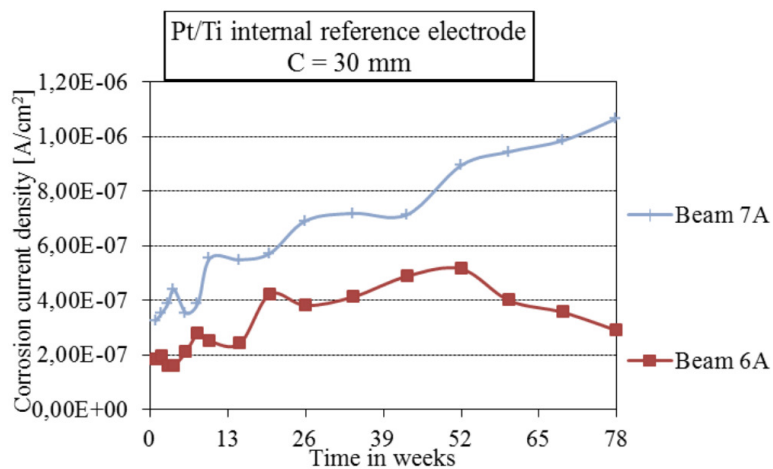


Fig. 11. Corrosion current density as a function of time in weeks for beams with various types of loading

It is seen that the second beam shows a significantly higher corrosion current than the first one. There may be two reasons for this observation. The first is that under the variable load no crack healing can occur. The second is that the maximum crack width is governing for the rate of corrosion, as suggested also by Fig. 10.

The last comparison is given Fig. 12, in combination with Table 4.

Table 4. Data for the beams the behaviour of which is compared in Fig. 12

Series	Concrete cover (mm)	Mean crack width (mm)	Number of cracks	Type of loading
2B	20	0.15 + longitudinal	9	static
3A	20	0.15	9	static
4B	20	0.15	6	static
1A	20	0	0	unloaded

In Fig. 12 it is seen that beam 2B, with a mean crack width of 0.15 mm, but with an additional longitudinal crack, shows a larger corrosion current density as the similar beam 3A, without a longitudinal crack. The beam 4B, with 6 cracks shows a smaller corrosion current density than the similar beam with 9 cracks (in beam 4B 3 cracks were coated, so corrosion is prevented): this can be explained on the basis of the measuring system, which included the resistance over the full length of the bar, so that 9 cracks offer a larger resistance due to a larger number of corroded bar areas.

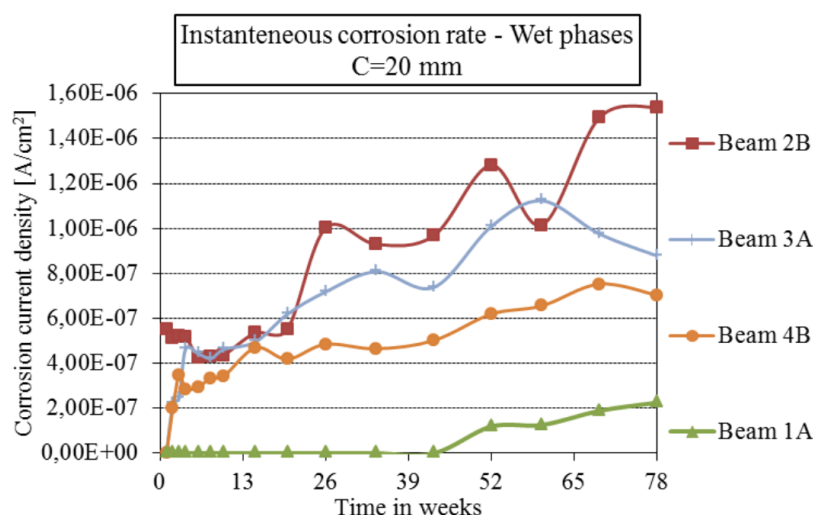


Fig. 12. Corrosion as a function of time for beams with different types and number of cracks.

Finally, as expected, the beam without cracks shows the lowest corrosion activity. The first signs of corrosion are observed after about 40 weeks.

From the measurements it is clear that cracks influence the rate of corrosion and that the magnitude of the crack width plays a role. Corrosion does not stop after a few weeks of initiation due to self healing, and calcium-, rust- and dirt deposits, as suggested in [5]. In a number of cases even after 1,5 year the “early propagation corrosion” continues. This is especially true for the beams subjected to variable loading. After two years all beams were sawn-open and the areas into which chlorides have protruded were highlighted. Fig. 13 shows this the chloride intrusion is substantially influenced by the presence of cracks. The bending cracks facilitate the ingress of chlorides over a large area, because the crack faces function as areas in direct contact with the environment. Also the areas where corrosion is observed are related to the crack pattern.

In a number of cases the corrosion occurs at the reinforcing bar where the bar intersects the crack, but in other cases at some distance from the crack, showing that chlorides, oxygen and water can infiltrate the concrete to a more remote location from the crack along the bar across the area adjacent to the crack, damaged by microcracking.

These observations show that the theoretical models, used for the probabilistic determination of service life, based on the assumption that serious damage only occurs after the end of the initiation time, should be revised.

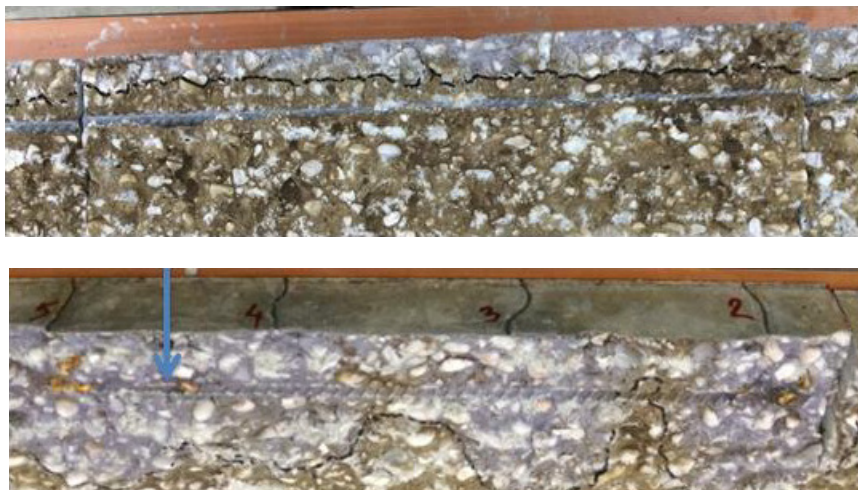


Fig. 13. a) Chloride-intruded area in beam without bending cracks (light area), b) Chloride-intruded area in beam with bending cracks (light area)

4. Determining the remaining service life when assessing existing structures

One of the tasks of assessing the condition of an existing structure is to determine the remaining service life. That means that the period must be determined in which the structural safety still satisfies the demands. A number of new questions show up like:

- how is the bending moment capacity influenced by the occurrence of rusted bar areas? In other words: do all corrosion pits occur in one cross section or is it reasonable to assume a certain degree of spreading so that not necessarily the worst possible scenario has to be adopted?,
- how is the shear and the punching capacity influence by local corrosion of the rebars which are part of the bearing mechanism,
- how is the anchorage of bars influenced by corroded bar areas?

Here certainly probabilistic considerations play a role. It is important to realize that structures have mostly been designed with models which hide some inherent conservatism. Code models should be easily applicable for designers and therefore offer a simplified representation of the real behaviour. If more advanced models can be mobilized, it might be possible to demonstrate that the bearing capacity is larger than follows from the original design rules. An example of this relates to the decks of prestressed concrete bridges, where taking into account the effect of compressive membrane action, leads to the conclusion that the bearing capacity has sufficient reserve capacity and no strengthening is required [6].

A stunning example of a structure which turned out to be substantially stronger than expected was the Maas Tunnel in Rotterdam, The Netherlands. This tunnel, constructed between 1937 and 1942, is the first immersed tunnel in the world. During visual inspection of the bottom of the tunnel (Fig. 13) it turned out that there was major concrete damage. The concrete cover was loose over an extended area and substantial corrosion of the reinforcement was observed.

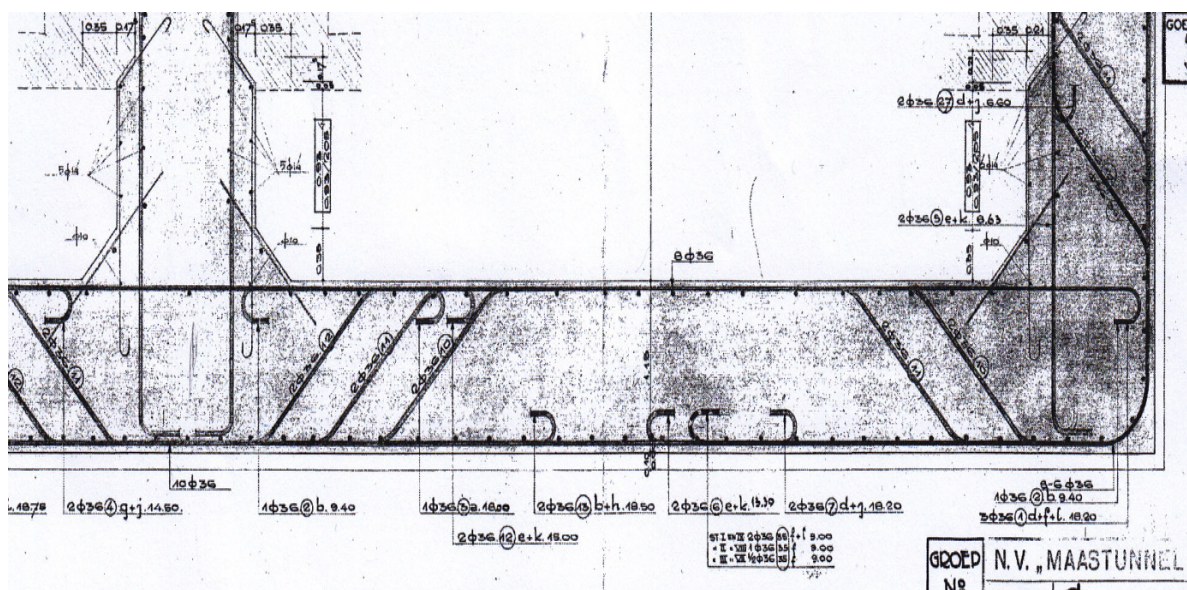


Fig. 13. Original design drawing of bottom slab of Maas Tunnel Rotterdam, 1937

In the river Maas, the tunnel floor has to carry a maximum upward water pressure of 18 meters, and there was sever doubt about the structural reliability of this floor in the corroded situation, given the observation that the cross section of a large number of the reinforcing bars was largely reduced as a result of chloride attack. The structural safety was investigated with different nonlinear FEM programs Fig. 14. The analyses were carried out in steps, assuming that an increasing part of the cross section of the reinforcing bars was reduced due to corrosion. The result of the analysis was remarkable. It was demonstrated that even if the cross section of the reinforcement of the slab would have been reduced to zero, the structural safety would still be sufficient. Fig. 14 shows the stresses calculated with the NLFEM program ATENA. It demonstrates the generation of concrete arches as a bearing mechanism, under the action of the water pressure in vertical and horizontal direction.

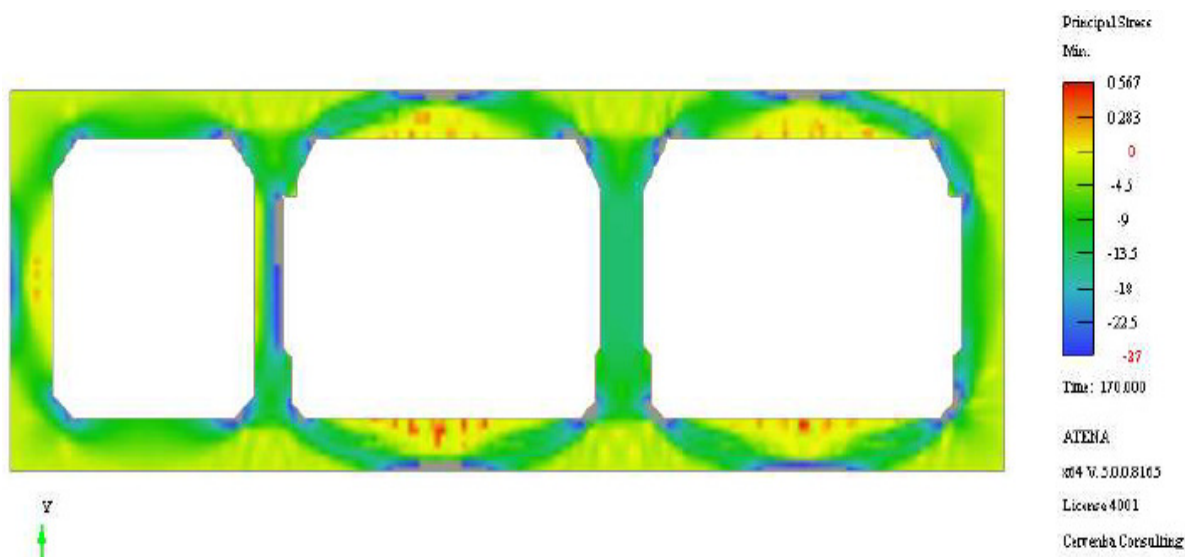


Fig. 14. Analysis of the structural safety of the Maas tunnel Rotterdam, with corroded bottom slab reinforcement, by the NLFEM program ATENA

5. Conclusions

Aging of concrete structures is developing to a major cause of damage in concrete structures.

Although with regard to knowledge of the aging effect of concrete structures substantial steps have been made, there are still essential questions waiting for an answer.

Contrary to many references in literature, which claim that the end of the initiation time of an uncracked structure determines the (beginning) of the end of service life, it could be demonstrated that the role of cracking should not be ignored.

The assessment of the residual capacity of structures built decades ago is an area which deserves major attention.

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