TITLE:

A REVIEW OF LITERATURE AND CODE REQUIREMENTS FOR THE CRACK WIDTH LIMITATIONS FOR DESIGN OF CONCRETE STRUCTURES IN SERVICEABILITY LIMIT STATES

RUNNING HEAD:

A REVIEW OF SLS DESIGN OF CONCRETE STRUCTURES

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ABSTRACT

This paper includes a critical review of literature related to design of reinforced concrete structures in the serviceability limit state (SLS), and attempts to tie the results to code requirements. The main attention is given to so-called controllable cracks, defined as cracks due to imposed loads or deformations on hardened concrete or restrained volume changes in young hardening concrete that the structural engineer has the knowledge and tools to predict and control. It is suggested that more consistent procedures are introduced into the future SLS-design. It is found necessary to more clearly distinguish the crack width requirements for aesthetic reasons from durability and tightness reasons, and clarify the use of the term *durability* in the code text. The research leading up to current leakage prediction formulas and tightness formulas is questioned based on the literature review. It is discussed that a differentiation of the crack width through a cross section will facilitate a more consistent treatment and limitation of crack widths.

Keywords: Concrete structures, Serviceability limit states, Cracks, Durability, Corrosion, Tightness, Aesthetics.

1 INTRODUCTION

In an on-going Norwegian research project, Durable Advanced Concrete Solutions (DaCS), literature related to research on serviceability limit state (SLS) crack width requirements has been critically reviewed. This article is an outcome of a wide review of the state-of-the-art performed in the research project and is also based on the combined experience between the authors. It was attempted to tie the findings to the code requirements in Eurocode 2 [1], NS3473 [2] and *fib* Model Code 2010 [3]. In the process, the need for logical explanations for several SLS-related topics became evident. The various interpretations of research results and regulative requirements throughout the history are special for the SLS-design, and it can be argued that an approach through verification of hypotheses rather than objective interpretation of research results has governed the evolution of SLS-design [4]. It seems like the research mostly have been executed to verify expected results, and that the test methods frequently appear to be made to facilitate for this.

In an attempt to establish a clear terminology, it is referred to Figure 1. As both illustrations indicate, the crack width is narrowest close to the reinforcement and increase in width with increasing distance from the reinforcement. In addition, small horizontal cracks along the reinforcement bars close to the main crack, can be observed. Crack widths a) to e) can be defined as follows: **a)**, defined as the maximum crack width at the surface, anywhere on a given structural member, exemplified in [5]. The illustration on the right hand side further includes the following; **b)**, crack width at the surface directly over the load bearing reinforcement; **c)**, crack width at a distance from the reinforcement surface corresponding to the minimum cover from a durability perspective and; **d)**, crack width in the vicinity of the reinforcement. Finally **e)**, the narrowest through-crack passage between the reinforcement bars, is also defined, however not shown in Figure 1.

[FIGURE 1]

This paper is limited to *controllable cracks* defined as cracks the engineer doing the structural design has the knowledge and tools to predict and control [4]. The crack width of such cracks is determined by equilibrium calculations including the effects of imposed loads and deformations on hardened concrete or restrained volume changes in young hardening concrete. Cracks from other effects, e.g. plastic shrinkage, alkali-silica-reaction, sulphate attacks, expansive oxides (corrosion) [e.g. 8] and freeze/thaw deterioration, are here defined as *non-controllable cracks*. Non-controllable cracks are those that originate from causes not currently included in the crack width estimation proposed by the codes. In addition to the aforementioned limitations of the current crack width calculation formula, the crack width estimates assumes that the concrete quality is adequate, the concrete cover is as intended, the execution is performed according to the requirements in the relevant execution standard, the design is correct and that no excessive loading or other unintended effects occur.

In the early versions of the design codes, after the limit state principles were introduced early in the 1970-ties, the requirements in the serviceability limit states was usually verified by distribution of minimum reinforcement in addition to deformation control. In the late 1970-ties and early 1980-ties, reinforcement stress requirements were introduced in various reference codes and client requirements. In Norway, the first crack width requirements were introduced to the design code NS3473 in 1989. In the first version of the requirements, the maximum crack width for concrete in severely aggressive environments was set to 0.10 mm for pre-stressed reinforcement, and 0.20 mm for ordinary reinforcement. In a revised version of the Norwegian standard from 1998, the requirements were relaxed to 0.2 mm and 0.3 mm, respectively, which constitutes a considerable relaxation of the requirement. It should be noted that the early requirements were given with two significant digits, and that the required cover thickness was increased over the same period of time. Other European countries introduced and adjusted the requirements in the same time period. With the introduction of the Eurocodes around 2010, the crack width criteria for ordinary reinforcement remained about the same as in the last version of the Norwegian standard. On the other hand for the pre-stressed reinforcement the additional requirement denoted decompression was introduced, which definitely was a step in conservative direction often leading to about 20% increased pre-stressed reinforcement in bridge superstructure design. Note that, even though the crack width criteria for ordinary reinforcement have not changed significantly the later years, the crack width calculation methods have changed, and the required cover thickness has further increased. For a brief introduction to the historical development of crack width requirements prior to 1983, see also Beeby's contribution to [9].

The literature review described above revealed inconsistencies in the crack width regulations of the codes, which the current paper attempts to enlighten. It is recommended to clearly distinguish between crack width requirements related to aesthetics, durability and tightness. Until the influence of the reinforcement layout on the crack pattern and the effect of the crack width and the crack pattern on the corrosion process is better understood, it is recommended to go one step back and simply limit the crack widths by distribution of minimum reinforcement and allowable steel stress in a corrosion protection perspective. A main reason for this is that today's crack width regulations favour small concrete cover and dense placement of small reinforcing bars which clearly not are good solutions to prevent corrosion. Further refinement of durability and serviceability related requirements should thereafter be based on a step-wise refinement.

2 LITERATURE REVIEW

2.1 General

The content of this paper is derived from a critical literature review where several simplifications regarding test set up and time period, e.g. accelerated tests, are critically reviewed with focus on how it relates to, or deviates from the validity range of the code requirements and common practice. The literature review results are categorized in the topics; aesthetics, reinforcement corrosion and tightness, similar to Beeby in 1978 [10].

Recent comprehensive literature reviews on the crack width formulas can be found in [11, 12]. The paper from Borosnyói & Balázs [11] concludes that despite of 100 years with enormous research activity on the topic, there is no globally accepted formulation for crack width nor crack spacing prediction for reinforced concrete. Lapi et al. [12] recently drew a similar conclusion, where it was concluded that the results presented in the paper show that the prediction capacity does not necessarily increase as the cracking model is refined. One could also find that all the calculation models focus on prediction of the crack width at point **a)** or **b)**, ref Figure 1, since the calculation models assume that plane sections remain plain after deformation, and the fact that the formulas are empirically adjusted to the surface crack widths [11, 12]. A refined crack width calculation model, based on experimental behaviour and analytical solutions of the bond-slip relation near the cracks, seems to be more consistent in predicting the surface crack width for large scale concrete structures, and is recently published by Tan et al. [13, 14]. Beeby [10] states that the corrosion protection relevant crack width should be **d)**, if any relevance between crack width and corrosion can be found.

2.2 Aesthetics

In this paper, aesthetics is related to what the user of the structure can see, and not to the reliability or durability of the structure. Aesthetics is probably the SLS-related topic which is receiving least research attention, possibly since it often is considered to be outside the scope of the codes [15]. However, some studies have been performed and *fib* [16] highlights a study performed by Campbell-Allen [17]. The study investigated the relationship between the acceptability of crack widths and the distance from the viewer for nine categories of structures. The idea behind was that the less strict category “little-used building” would yield a higher acceptance for surface imperfections than “Monumental buildings”, which is the category with highest prestige. The approach is shown in Figure 2.

[FIGURE 2]

In addition to the Campbell-Allen study, a study performed by Haldane [18] is found in the literature which covers the public opinion regarding acceptable concrete surface crack widths. In this study a group of individuals were presented with cracked concrete surfaces, and asked to judge whether or not the crack could give rise to appreciable dissatisfaction. Beeby [10], however, points out some aspects related to the survey which may influence the validity of the results. Beeby suggest that the correct way of discovering the width which give rise to appreciable dissatisfaction, would be to make a study of situations where cracking has given rise to complaint, as a substitute for the surveys which points out the crack directly and asks for an opinion. It should also be noted that the survey of Haldane [18] does not have restriction on the viewing distance, and conclude that surface cracks exceeding 0.25 mm are likely to give rise to appreciable dissatisfaction. According to the conclusion of Haldane, the public can be expected to put stricter criteria for the surface crack width than what the engineering community does [18].

Introduction of restrained imposed deformations from e.g. young hardening concrete into the crack width calculations, as proposed by the upcoming revision of the Eurocode 2 [19, 20], are necessary in order to adequately describe the final impression of a given concrete surface.

Based on Figure 2, *fib* [16] states that a crack width limit of 0.3 mm may be applied to achieve acceptable appearance. It is stated that it is generally accepted that under normal lighting and associated conditions, a crack width of 0.3 mm will be visible to the naked eye at a distance of about 3 meters.

fib [16] recognizes and points out that although Figure 2 might indicate that a certain crack width is found to be adequate, the visibility can be enhanced on a drying surface after being wet. It is also stated that the “visibility” of cracks can be greater for cracks contaminated with dirt. *fib* further states that crack widths larger than 0.3 mm might be tolerated from an aesthetic perspective if it’s found acceptable and the viewing distance is significantly exceeding 3 m.

2.3 Corrosion

The concrete society [21] examined two viewpoints regarding the impact of cracking in concrete on reinforcement corrosion: 1) cracks significantly reduce the service life of the structure, accelerate corrosion initiation and provide space for the deposition of the corrosion products, and 2) corrosion initiation may be accelerated by cracks, but the influence on the subsequent rate of corrosion is minimal and limited to zones where the cracks cross the reinforcement. The concrete society [21] does not provide a clear conclusion, but both viewpoints recognize that the initiation phase seems to be reduced with the presence of cracks [21], which appear to be a common conclusion for the reviewed literature explicitly commenting the initiation phase [4, 10, 21, 22, 24]. Furthermore the subject is made more complicated by an ongoing discussion within the standardization bodies where it still is being debated what should be the definition of the end of working life for the structures, i.e. whether it should be synonymous with de-passivation of the reinforcement, or if a part of the propagation period with active corrosion should be included. In any case, the limit state should be accompanied by an accepted probability of exceedance.

While there seems to be consensus in the literature about the influence of cracks on the corrosion initiation time, several authors point out that there seems to be a lack of common understanding of the consequences in a service-life perspective [10, 21, 23]. West et al. [24] summarised an experimental database, given in Table 1. The critical crack width is defined as an upper bound limit for when corrosion is prevented or assumed to be negligible [24].

[TABLE 1 AND TABLE 2]

From Table 1 and Table 2 it can be seen that variations in the conclusions regarding critical crack width can be found based on the studies included. More than half of the short-term studies conclude with a critical crack width, which ranges from 0.1 mm to 0.5 mm. For the long-term studies, however, only O´Neil [35] concludes with existence of a critical crack width. Some of the long-term studies do not support the need to limit the crack width as currently done to assure adequate durability, based on the idea of a critical crack width [4, 10, 39-43]. Misra and Uomoto recommend a critical surface crack width applicable for cracks parallel to the reinforcement in question [29].

According to most code requirements, the designer shall today calculate the crack width at the surface, i.e. position **b)** from Figure 1 [46]. According to NS 3473 [2] and the Norwegian annex to EC2 [1], the crack width requirement is linearly increased with the ratio between the distance from the reinforcement to **b)** and **c)** if one chooses to increase the cover relative to the minimum cover requirement from a durability perspective. The increase is however limited to 30%. Approaches with similar effects are given in other codes, e.g. [52] where a maximum value of 50 mm is used for the cover in the crack spacing calculation, [52] where the critical crack width is given as a linear function of the cover and [53] where a constant value of 20 mm is used for the cover in the crack spacing calculation. Several studies [4, 10, 21, 24, 47-50] point out location **d)** in Figure 1 as the most relevant crack width position with respect to corrosion prevention. However, this is a complicated topic, since both experimental and theoretical research point out that this crack width approaches zero [14,54].

Danner and Geiker [55] investigated the influence of exposure and orientation of surface and cracks on ingress and self-healing for selected long-term in-situ exposed cracked Norwegian concrete structures. A summary of finding on drilled cores is given in Table 3. The experience from this study is broad and interesting, however spans beyond the scope of this paper [55]. The overall experience from the assessments confirms the previous research, i.e. it is not straight-forward to conclude on the corrosion state of the reinforcement based on the surface crack width. The in-situ investigations also show that there is no direct link between the surface crack width, **b)**, and the chloride ingress/profile.

[TABLE 3]

2.4 Tightness

Research on tightness of concrete generally concludes that through-cracks are required to achieve mention-worthy leakages [56, 57, 58], which is aligned with Beeby’s view from 1978 [10]. It is noted that some of the research groups seem to have tested either un-reinforced concrete discs [60], or a small portion of the structural member, e.g. the tensile zone of a beam under flexural loading [59]. Both cases represent through-cracks, and are further used as basis for recommendations on a more general basis. Desmettre and Charron [59] tested an RC-tie, and concluded for flexural cases based on the through-cracked results. Ziari and Kianoush [58] tested pure tension elements and explicitly limited the relevance of the findings to pure tension load situations with through-cracks.

A study outlined in [60], indicates how much the reinforcement density influences the leakage rates. Mivelaz [61] found that an increase in reinforcement amount from $\rho=0.5\%$ to $\rho=1.5\%$ yielded a scaling of leakage per crack meter in the order of magnitude 10^{-3}, whilst the global flow rate, i.e. flow rate per surface area, showed a scaling of 10^{-2} under equal reinforcement strain, i.e. increased load intensity for the higher reinforcement density specimen in order to reach equal strain. No studies of how leakage rates vary from un-reinforced to $\rho=0.5\%$ are found.

Table A.9 in NS3473's informative Annex A, is the current guideline for offshore structures where tightness is desired, see Table 4 below. The proposed leakage calculation formula from NS 3473 can be derived using results from un-reinforced discs under uniform tension load.

[TABLE 4]

In the main part of NS3473 [2], Section 15.5.2, it is stated that structures exposed to a pressure difference in fluid or gas, harmful flow shall be prevented by fulfilling one or more of the following; Minimum thickness, maximum tensile stress, minimum compression zone or maximum crack widths. This seems incomplete and ambiguous, and can cause confusion as long as it is not further elaborated how this statement shall be interpreted in conjunction with Table 4.

A similar table has been worked out for Eurocode 2, part 3 as well, see Table 5 below. This part of Eurocode 2 addresses design of concrete structures which fall under the category of liquid retaining and containment structures. The parameter h_D in Table 5 is defined as the hydrostatic pressure and h is defined as the wall thickness of the containment structure.

[TABLE 5]

3 DISCUSSION

3.1 General

As for the result section, the three relevant topics are treated in sub-sections and the discussion of the results from the literature review are supplemented with input and experience from offshore structures and the Norwegian infrastructure.

3.2 Aesthetics

From an aesthetic perspective it is recommended to incorporate the hardening phase effects as proposed in the planned EC2 revision. From a user point of view, it is irrelevant what caused the crack, and it is clear that it is the surface crack width which should be limited. It is however emphasized that the crack width limitation should only include controllable cracks, i.e. cracks resulting from plastic shrinkage or deterioration cannot be captured through standard design methods.

If aesthetics is found to be within the scope of the codes, opposed to the suggestion in [5], the codes should develop methods corresponding to the research findings [17], i.e. facilitate for methods which differentiate requirements based on viewing distance, prestige level and climatic conditions for the respective member. It should also be clearly emphasized that the requirement is aesthetic, such that the owner can evaluate the impact of the limitation chosen for a given project.

The experience from the offshore industry is that the aesthetic arguments alone are of secondary importance. The main philosophy is to assure adequate structural behavior, e.g. proper fluid retaining properties throughout the service life for expected operational loads. On the other hand bridges should be treated at buildings with crack width requirements related to viewing distance and location of the structural member in question.

3.3 Corrosion

The effect of crack widths on the corrosion process has been a topic for research for several decades and only simplified and/or accelerated tests have concluded that the methodology with surface crack width control used today is relevant. Some studies even claim that the present methods are counteractive in a durability perspective [21], since most of the present crack width calculation methods favour a small cover and closely spaced bars with small diameter.

It is, in all cases, necessary to agree on where in the cross section the crack width is of interest, if relevant at all. Most research indicates that the crack width at the vicinity of the reinforcement is most relevant [4, 10, 21, 25, 47-50]. A pragmatic reason for controlling the surface crack widths, as done today, can be accessibility for measuring after testing. However, it can be argued that one of the most important properties of the cross-section with respect to durability, is the ability of the cover to protect the reinforcement from harmful substances. If a critical crack width exists, and it is defined as the width beyond which the cover loses the protective properties, crack width control at location **b)** or **c)** in Figure 1 seems most relevant. If a critical crack width is found, a differentiation based on exposure condition and corrosion sensitivity of the reinforcement may seem redundant, since a critical crack width, as defined, will imply that the reinforcement is protected against detrimental substances regardless of the exposure condition and the corrosion sensitivity of the reinforcement. The surface crack width can also be linked to the debonding length [22], which is by some researchers claimed to be the governing measure [62]. The debonding length is variable and defined as the distance, along the reinforcement bar, between the surface crack and the location where maximum shear stresses are reached between the concrete and the reinforcement. To determine the debonding length, the total confinement provided is believed to be of importance. How confinement from reinforcement influences the debonding length is presently not fully understood [14].

Since there are still large uncertainties related to how accurately the surface crack pattern can be predicted and how the reinforcement layout influences the corrosion process, it is difficult to recommend a surface crack width requirement. As a pragmatic solution to take care of the more general requirements in SLS and before the influence of the surface cracks on the durability are understood and the current surface crack width calculation methods are improved, it can be recommended to only limit the reinforcement steel stress calculated based on a cracked section. Further work can be related to a step-wise improvement of methodology and physical understanding of the influence of predictable surface cracks on durability.

For offshore concrete structures exposed to harsh environment, early age surface cracks above 0.4 mm crack width are repaired or sealed within the construction period. Also any surface defects caused by poor compaction or the construction method are normally repaired during the construction phase. Although specific offshore projects generally are confidential, the experience from the offshore industry among the authors is that none of the locations where the durability is questioned could have been prevented with stricter crack width limitations, since there is no experienced correlation between highly tension-utilized structural components and corrosion. Marginal corrosion is found to a limited extent on some structures. Possibly since measures for avoiding corrosion are given significant attention in construction and design. However, durability concerns are rarely reported to originate from cracks that are determinable with today's crack width calculation methodology.

Similarly as for offshore structures, it is the experience among the authors that no known durability problems within the Norwegian infrastructure have their cause in controllable cracks. It is especially interesting that this statement holds for time periods when there have been no crack width requirements, and where the requirements have been both very strict and more liberal. The durability problems are usually due to alkali-silica-reactions or bad execution, insufficient concrete cover and/or initial chloride content resulting in reinforcement corrosion.

3.4 Tightness

One can argue that some researchers are interested and focus on global tightness, whilst other focus their interest on tightness in the sense of the concrete being able to protect the reinforcement. I.e. tightness seems in some research to be the understanding of chloride ingress to the reinforcement rather than the structures ability to contain gas or liquid. If tightness experiments are to be related to durability, as in e.g. [57, 58, 59], it should only tie its findings to durability if conformity with durability experience and durability specific literature is assured. Tightness, in a code perspective, is related to the capability of a structure to contain gas or liquids in an adequate manner.

Neither the assumption that results from experiments on tension prisms can be transferred directly to the behaviour of beams, nor the ability to tie a certain leakage-rate to a given reinforcement stress or surface crack width on a general basis [10, 59], seems to be justified. The assumption of e.g. [59] seems more rational if the concern is the ability of the cover to maintain tightness for the reinforcement, or in the situation of a pure tension load scenario as investigated by Ziari and Kianoush [58]. However, only the latter comply with the requirements of the codes, which seem rather univocal that the goal is to assure containment capabilities for the member in question.

The design codes give guidance for tightness control according to different tightness classes, see Table 4 and 5, reproduced from NS3473 and Eurocode 2, but the criteria are ambiguous because they are not necessarily more strict for the higher tightness classes than for the others. As an example, if a structure is subject to the normal requirements from Table 4, the requirements to the surface crack width is 0.2 mm, which can under certain circumstances be exceeded, although the particularly strict requirement for tightness is fully satisfied.

The general experience from the offshore industry is that the tightness has been adequate throughout the history, although the requirements have changed over the decades. It should be noted that the introduction of restrained volume changes from young hardening concrete and other previously neglected effects could lead to overly conservative structural solutions, depending on the degree of refinement of the calculation method, when compared to previously satisfactory behaviour.

4 CONCLUSION

4.1 General

It has been discussed and argued that it is necessary to reconsider how the influence of surface crack widths on durability, aesthetics and tightness is treated in the regulations. The conclusion is aligned with Beeby's recommendations [10]. It is recommended to improve the crack width calculation methods, and is stated that it is questionable to keep today's surface crack width limitations in a corrosion perspective, and that the aesthetic perspective seems to lack conformity with research.

4.2 Aesthetics

It is concluded that crack width requirements due to aesthetics should be in accordance with research experience, e.g. [17]. If the codes choose to take responsibility for the aesthetics, the limitation should be a function of anticipated viewing distance, prestige level of the structural member in question, and the climatic conditions. Furthermore it is concluded that it is the surface crack widths that are of importance for aesthetics, and that the young hardening concrete effects should be included.

4.3 Corrosion

Until the physical understanding of how the reinforcement layout and crack pattern influence the corrosion process and the crack width calculation methods are improved, it is recommended to limit the control with respect to durability to limiting the reinforcement steel stress. This represents an improvement compared to the present practice where a small cover and closely spaced reinforcement with small diameter are favoured. Further work should be related to a step-wise improvement of the physical understanding and the calculation method.

4.4 Tightness

The idea that leakage rates can be tied to surface crack width is in conformity with several studies. However, these studies are typically performed on un-reinforced concrete discs where plane sections remain plain after deformation. It is evident that for e.g. an hour-glass, one would estimate the leakage based on the narrowest free passage. Following this philosophy, **e)** is marked out as the relevant parameter for leakage estimations.

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Table 1: Summary of a literature review of crack width studies and the influence of crack width on reinforcement corrosion with a duration less than 3 years – rewritten after [24].

Investigator (Cover)	Critical crack width	Conclusion*
Kahhaleh, K.Z [25] (50 mm)	-	<u>No specific information on critical crack widths:</u> Tests on epoxy coated reinforcement. The test results indicate that the cracked members experienced corrosion initiation much earlier than uncracked members. However, specific crack widths did not show an influence on corrosion initiation and progression.
Schiessl and Raupach [26] (15 and 35 mm)		<u>Crack width has little influence on corrosion:</u> Investigated crack widths of 0.1 mm, 0.2 mm, 0.3 mm and 0.5mm. The results of the laboratory tests clearly indicate that the problem of reinforcement corrosion in cracked zoned cannot solely be solved by crack width limitation in the range from roughly 0.3 to 0.5mm; corrosion protection must be assured primarily through adequate concrete quality and cover.
Lin, C.Y. [27] (-)		<u>No conclusive remarks about critical crack widths:</u> For the tested specimens with sustained loading, the crack width did not affect the amount of corrosion for the range of crack width considered (0.10 mm to 0.18 mm). Cracked and subsequently unloaded specimens resulted in less corrosion than the sustained loaded specimens. The author recommends that cracks should not be permitted under sustained or frequent loads.
Makita, Mori, & Katawaki [28] (-)		<u>No conclusion about critical crack widths:</u> The measured surface crack width after exposure was 0.5 mm to 0.3 mm. Autopsy of the specimens after 1000 days of exposure led to a reported conclusion that there did not appear to be a correlation between surface crack widths and corrosion, for both the pre cracked specimens and the initially un cracked specimens.
Misra & Uomoto [29] (10 mm)	0.5 mm	<u>Critical crack width for longitudinal corrosion induced cracks. Smaller crack widths clearly yielded less observed corrosion:</u> The study mainly focuses on the potential critical crack width for cracks induced by corrosion, in the longitudinal direction. In the study it is reported that once corrosion induced longitudinal cracks reaches 0.5 mm or more in width, the amount of corrosion observed is clearly higher than the corrosion observed in areas with lower longitudinal crack widths. It was also shown that the presence of shear reinforcement gave sufficient confinement to avoid the vicious effect of longitudinal cracking.
Vennesland & Gjorv [30] (-)	0.4 to 0.5 mm	<u>Critical crack width (under sea water):</u> Smaller crack widths yielded corrosion which was deemed not significant.
Okada & Miyagawa [31] (15 and 20 mm)	0.1 to 0.2 mm	<u>Critical crack width:</u> Two series were tested, where only the second series, Series 2, are deemed relevant for this manuscript. Series 2: loaded to produce cracks up to 0.3 mm, or provided with pre-formed cracks of 10 mm and some specimens had 3.13% NaCl added to the mixing water. It was concluded that as the water-cement ratio of the cement increases, corrosion of reinforcing steel accelerates. The experimental results suggest that the critical crack width is between 0.1 and 0.2 mm. However, as the water-cement ratio increased there was less correlation between crack width and potential difference.
Swamy, R.N. [32] (50 to 70 mm)	0.1 to 0.15 mm	<u>Critical crack width:</u> Tested concrete surface crack widths from 0.11 to 0.25 mm with a steel stress adjusted to approximately 200MPa. Reinforcement types of plain, epoxy coated and galvanized steel were included with three thicknesses of the epoxy coating (100 µm, 200 µm and 300 µm). None of the reinforcement protective measures could overcome the negative impact of insufficient cover, poor concrete quality or excessive cracking. It was concluded that cover to steel is the most critical factor in preserving the electrochemical stability of steel. The results suggest that the concrete cover, quality and the crack width all play an interactive role in the durability of reinforced concrete structures.
Berke, Dalliare Hicks & Hoopes [33] (38 mm)	0.2 mm	<u>Critical crack width:</u> Test setup based on ASTM G109 macrocell specimens. The tests were conducted to evaluate the effectiveness of calcium nitrite as corrosion inhibitor. A total of 8 specimens, of which four had calcium nitrite added. Loaded to an average crack width of 0.2mm, shim introduced before offloading and consecutive exposure. The beams without corrosion inhibitor showed more severe corrosion and spreading of the corrosion several diameters from the crack. Overall, the results showed that a crack width of 0.2 mm was insufficient to prevent corrosion with a 38 mm cover in the absence of calcium nitrite. The specimens with calcium nitrite proved effective in limiting corrosion at a crack width of 0.2 mm.
Houston, Atimtay, & Ferguson [34] (25 to 75 mm)	0.13 mm	<u>Critical crack width for normal cover thickness:</u> In many cases corrosion was initiated at flexural crack widths greater than 0.13 mm. However, for specimens with 25 mm cover a limitation of crack width less than 0.10 mm did not ensure prevention of corrosion.

* Elaborated descriptions of the conclusions relative to the table from reference [24], the elaborated explanation is taken from appended descriptions of the studies from [24], and mainly consist of direct or indirect quoting.

Table 2: Summary of a literature review of crack width studies and the influence of crack width on reinforcement corrosion with duration of 3 years or more – rewritten after [24].

<table>
<tr><th>Investigator (Cover)</th><th>Critical crack width</th><th>Conclusion*</th></tr>
<tr><td>O´Neil, E.F. [35]
(19 and 50 mm)</td><td>0.4 mm</td><td>Critical crack width:
82 reinforced concrete beams exposed for 25 years were included in the test. The purpose of the test was to evaluate the long term weathering of air-entrained and non-air-entrained concretes with several variables. The following was observed for the remaining beams after 25 years of exposure, where 11 beams were tested and autopsied.
-Corrosion of the reinforcing steel in beams stressed to 138MPa could not be matched with the flexural cracks in the beams.
-Beams stressed to 345MPa did show corrosion of steel matched with the flexural cracking in those regions. Since 345MPa beams had crack widths of 0.4 mm or greater, it was concluded in the study that crack widths of 0.4 mm or higher were necessary to produce corrosion at flexural cracks.
The results did not show a definite relationship between the steel stress levels and corrosion that would indicate more or less corrosion for lower stress levels. The study generally did find that steel at higher stress levels produced larger flexural cracks, and therefore would allow for greater penetration of water and oxygen.</td></tr>
<tr><td>Ohta, T. [36]
(20 to 68 mm)</td><td rowspan="6">-</td><td>No conclusive crack width value:
One hundred and forty nine pairs of reinforced concrete beams with open cracks were exposed for two to twenty years to sea air. The following conclusions were stated after 10 years of exposure
-Crack widths did not correlate with significant loss of the cross sectional area of the steel for 20 mm cover after 10 years.
-For 40 mm cover, it appeared to be a relationship between the amount of corrosion and crack widths.
-Corrosion was only slight for covers of 50 mm and 68 mm.
It was further concluded that the rate of progress of depassivation depends on the crack width when the cover is thick, and when the term of exposure is short.
The following are conclusion after 20 years of exposure.
-The effect of crack widths on corrosion disappeared for even the specimens with 40 mm cover.
-Specimens with 20 mm cover were very heavily corroded, and longitudinal cracks along the reinforcement were observed.
It was concluded from these tests that the cover, and not the crack width played the most important role in the control of corrosion of reinforcing steel in concrete.</td></tr>
<tr><td>Francois & Arliguie [37]
(-)</td><td>No conclusive crack width value:
Sixty-eight reinforced concrete beams were tested and covered a test period of ten years. The study did not report how different values of crack width influenced corrosion, but only said that crack widths were less than 0.5 mm. The results in the study suggest that the existence of cracks, and not their width, is the significant parameter in the corrosion of reinforcing steel. The idea from Ohta et al. [30] is supported, that the concrete cover is directly related to the rate of corrosion.</td></tr>
<tr><td>Tremper, B. [38]
(28.6 mm)</td><td>Cracks within the range of the study did not promote serious corrosion:
Crack widths from 0.127 mm up to 1.27 mm were tested for a period of 10 years. The autopsy after ten years found that all reinforcement was free from corrosion, except in the regions of cracks. However, this corrosion was deemed minor. It was concluded that for the tested specimens and the tested crack range that cracks did not promote serious corrosion of reinforcement steel.</td></tr>
<tr><td>Schiessl, P. [39 and 40]
(25 and 35 mm)</td><td>No critical crack width given – does not support the idea of a critical crack width
Schiessl does not propose any specific limiting value of crack widths because his research shows that, there is no value of crack width below which protection against corrosion could be guaranteed. Test results show that, for even a specimen with 25 mm cover, and a crack width of 0.15 mm, there still exists a 40% probability that corrosion will appear.</td></tr>
<tr><td>Tuutti, K. [41]
(-)</td><td>Cracks have only a local effect and do not change corrosion mechanisms:
Tuutti concludes that cracks in the concrete cover do not change the basic mechanisms of corrosion, but instead only have local effects. It is theorized that corrosion initiates when a threshold concentration of an initiating substance is achieved at the surface of the steel. The rate of corrosion is then determined by the flows of these substances to the area. So basically it is only the local flows of the substances that are changed by the crack widths.</td></tr>
<tr><td>Beeby, A.W. [4, 42 and 43]
(-)</td><td>Crack widths have little long-term influence – current guidelines unnecessary:
Beeby says that the current guidelines for controlling crack widths are unnecessary based on his own studies and Schiessls studies. This is based on the argument that cracking influence the onset of corrosion locally, but has a negligible long term effect.
Beeby also support his claim that crack control guidelines are unnecessary based on the work by Husain and Furguson [44] regarding crack widths at the level of steel in concrete. In this study, it was found that there does not exist a definite relationship between the widths of surface cracks, and the widths of cracks at varying steel depths. Therefore it is argued, that the use of a surface crack width</td></tr>
</table>

		value can be completely arbitrary with regard to the actual situation in the field.
Käthler, Angst, Wagner, Larsen and Elsener [45] (10 to 120 mm) **	-	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.

* Elaborated descriptions of the conclusions relative to the table from reference [24], the elaborated explanation is taken from appended descriptions of the studies from [24], and mainly consist of direct or indirect quoting.
** Reference in addition to the ones given in [24]. [45] is a separate review.

Table 3: Norwegian field studies, description of cases, after [55]

Structure	Cecilie Bridge	Tåsen Tunnel	Moholt Bridge	DNV Field Station	Hafrsfjord Bridge
Type	Beam (Box-girder) Bridge	Culvert	Slab bridge	Concrete column	Beam Bridge, NIB
Location	Trondheim	Oslo	Trondheim	Bergen	Stavanger
Structural component	Edge beam	Tunnel wall	Edge beam	Column	Foundation
Age (Years)	16	20	25	33	50
Exposure	De-icing salt (minor)	De-icing salt (minor)	De-icing salt (minor)	Tidal seawater (heavy)	Tidal seawater (heavy)
Climate	Inland	Inland	Inland	Marine	Marine
Concrete	C55, SV-40		C45	C60	B35
Cover (mm)	55	50	50	50	90

Table 4: Tightness Criteria table from NS 3473 (Table A.9), where f_{tn} is the nominal in-situ tensile strength of the concrete.

Design criterion	**a) Normal requirements for tightness**	**b) Particularly strict requirements for tightness**
Requirement for minimum cross sectional dimension, h	-	200 mm
Requirements for minimum compressive zone	[1] -	The smallest of 0.25 h and 100 mm
Maximum tensile stress from axial tension; σ_n=N/A	f_{tn}	[2] [3] 0

[1] If the compression zone of the cross section is less than 0.2h and 50 mm respectively, the crack widths shall be limited to 0.2 mm
[2] Values of σ_n above zero are permitted if minimum depth of the compression zone is increased to 200 mm
[3] If a prestress resulting from external fluid or gas pressure *p* is included in σ_n, *p* is added to σ_n.

Table 5: Tightness criteria from EN1992-3

Design criterion	**1) Leakage to be limited to a small amount. Some surface staining or damp patches are acceptable.**	**2) Leakage to be minimal. Appearance not to be impaired by staining**	**3) No leakage permitted**
Requirement for minimum cross sectional dimension, h	-	-	2)
Requirement for minimum compressive zone	-	The smallest of 50 mm and 0.2h	2)
Through-cracking requirements	If through cracked, the surface crack width shall be limited to 0.2 mm for $h_D/h<5$ and 0.05 for $h_D/h>35$. 1)	Should be avoided unless appropriate measures (e.g. liners or water bars) have been incorporated. i.e. Maximum tensile stress from axial tension; $\sigma_n=N/A< f_{ct,eff}$	2)
1) Interpolation of the surface crack width requirement is suggested for h_D/h between the 5 and 35. 2) Generally, special measures (e.g. liners or prestress) will be required to assure tightness.			

*These requirements are also found in Annex H of the draft version of the new Eurocode 2. It should however be noticed that the new method includes calculations procedures of fresh concrete effects, which in principle will be a conservative adjustment of the calculations compared with the current version of the Eurocode 2.

List of figure captions	
Fig. 1	Principle sketch after Goto [6] (left), and illustration to establish terminology, based on [7] (right).
Fig. 2	Acceptable crack width vs viewing distance for various buildings [17].

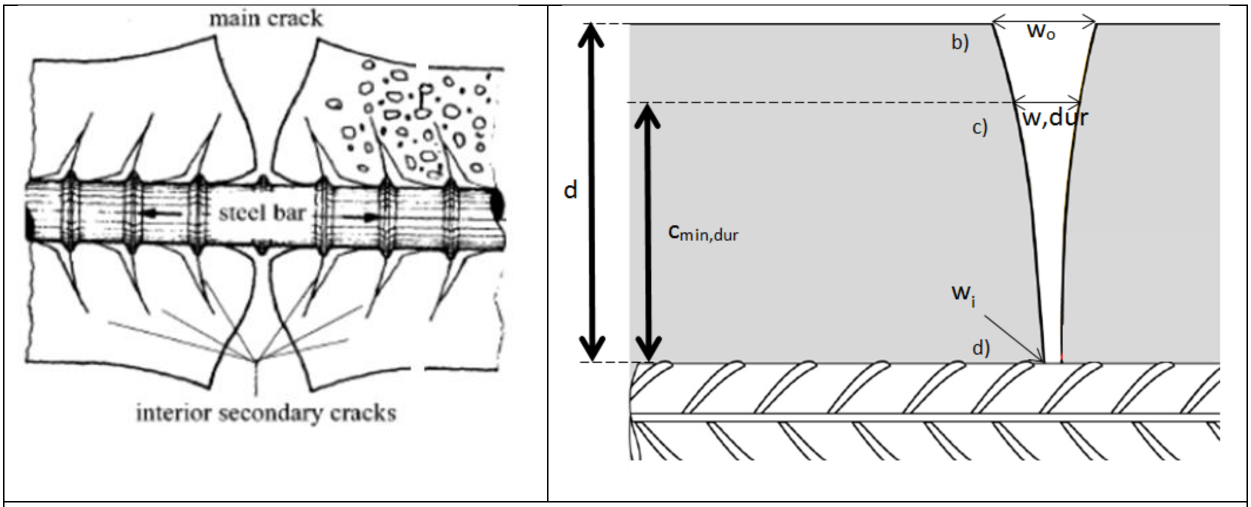

Fig. 1

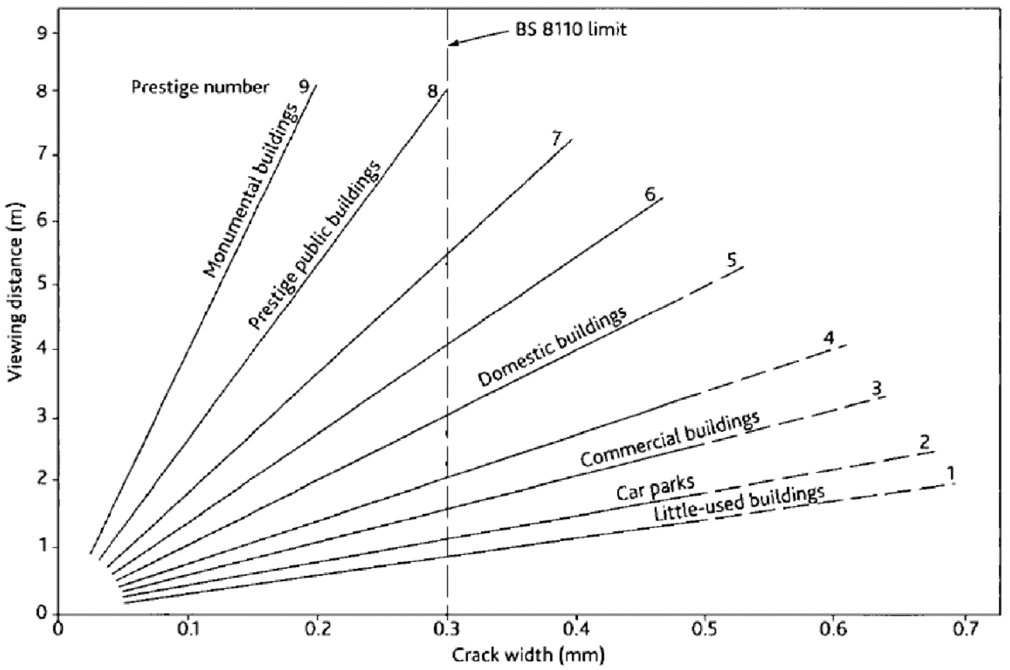

Fig. 2