Applicability of existing crack controlling criteria for structures with large concrete cover thickness

Chavin N. Naotunna [1], S.M Samindi M.K Samarakoon [1] and Kjell T. Fosså [1]

[1] Department of Mechanical and Structural Engineering and Material Science, Faculty of Science and Technology, University of Stavanger, Stavanger, Norway.

Abstract

Adverse effects from the cracks in Reinforced Concrete (RC) structures are controlled at the structural design stage. Cracks due to service load are controlled by limiting the ‘calculated crack width’ to a ‘maximum allowable crack width’. With the understanding of social and economic advantages of long design life structures, there is a trend of constructing structures up to 300 years of design life. To enhance durability, such structures require relatively large concrete cover thickness. The existing ‘crack width calculation models’, have to be validated before using on such large cover structures. The predictions of crack width calculation models in Eurocode 2, Model Code 2010, Japanese Code, American Code and British code were compared with the results of recent experiments with large cover specimens. It could identify that the aforementioned models have to be improved to predict the crack widths of large cover structures. The necessary improvements of each model have been identified. Next, a literature survey was conducted to check the applicability of the existing ‘allowable crack width limits’, for the structures with large concrete covers. To effectively use the existing allowable limits on such structures, the necessary improvements and future works have been identified considering the durability, aesthetic and tightness criteria.

Keywords – reinforced concrete structures, service load, crack width, durability, aesthetic, tightness, concrete cover thickness

1. Introduction

Cracks in concrete occur when the tensile stress on concrete exceeds its tensile strength [1]. Cracks are classified into three categories, according to the applied tensile load on the structure. They are tensile stresses induced by service loads (i.e. axial load, bending moments, shear and torsion, etc.), imposed deformations (i.e. differential settlements, shrinkage/creep, and temperature differences, etc.) and environmentally induced loads (i.e. frost action, carbonation and chloride penetration, etc.). These cracks in reinforced concrete (RC) structures create many adverse effects on the durability, aesthetic view and liquid or gas tightness of the structure. To avoid the discussed adverse influences from cracks, it is necessary to repair the generated cracks, resulting in a high level of repair cost [2]. Therefore, it is always preferable to limit cracks at the structural design stage. However, it is not possible to control every crack at this stage. Depending on the controllability of the cracks at the structural design stage, Beeby [3] has classified cracks as controllable cracks (load-induced cracks) and non-controllable cracks (plastic shrinkage, alkali-silica reaction, freeze/thaw deterioration).

To avoid the generation of cracks due to service load, the ‘stress of the tensile steel’ has to be limited to a very low value (for deformed bars, the tensile stress due to permanent loads has to be limited to 120 N/mm^2 [4]). In order to do this, it requires a large amount of reinforcement. This tends to drastically increase the cost of the structure and reduce the ease of construction. Therefore, in general, the cracks are allowed to occur, and they are controlled by limiting their widths. At the design stage the ‘calculated crack width’ is limited to an ‘allowable crack width’. However, the method of calculating the crack width and the values of allowable crack widths are different from region to region. Therefore, widely used crack width controlling methods in Eurocode 2 (EC2) [5], Model Code 2010 (MC2010) [6], Japanese Society of Civil Engineers (JSCE) code [4], American Concrete Institute (ACI) code [7] and British Standards (BS) code [8] have been investigated. The differences and the background behind each code is discussed, as it can be beneficial to decide, which model is to be used for the design of the specific structure. For example, the MC2010 crack width calculation model has a limitation for the concrete cover thickness as 75 mm. But the empirical-based ACI and BS codes have been developed considering the experimental results of 84 mm and 89 mm, respectively.

Recently, it has identified the economic and social benefits over the long-term of the structures with very long service life (200 or 300 years) [9, 10]. Concrete cover thickness is mainly increased to improve the lifetime of an RC structure from the durability aspect. The current requirement of concrete cover thickness is as high as 120 mm (Norwegian Public Road Administration guidelines [11]); as an example, Hafrsfjord Bridge in Norway is constructed with a concrete cover thickness of 90 mm [12, 13]. However, when using the existing crack width calculation models for such structures, it has to make special attention to check whether the existing models are applicable. As mentioned in the above paragraph, some ‘crack width calculation’ models have mentioned their limitations for the concrete cover thickness. However, it is important to check the applicability of other models, as they have not validated for such structures with large concrete cover thickness.

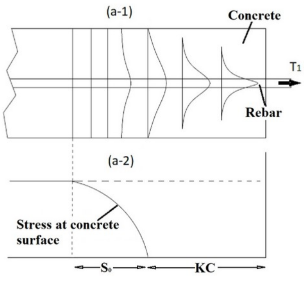
At the same time, it is important to check the applicability of existing ‘allowable crack width limits’, for the aforementioned structures, which requires large concrete covers. Because, when the concrete cover thickness increases, the crack width also increases. Therefore, if the crack width of a structure with a large concrete cover thickness is controlled to the allowable limits, which are prescribed for lower cover thicknesses, additional tensile reinforcement tends to be required. For this reason, it is necessary to identify how the existing allowable limits are decided and what improvements need to be made for them to apply to structures with higher cover thickness.

As mentioned, this paper is focusing on the applicability of existing ‘crack width calculation models’ and existing ‘allowable crack width limits’, for the structures with large concrete cover thickness. The manuscript starts with discussing the cracking phenomenon. Then it explains why the different codes suggest different models to calculate the crack width. By comparing the recent experimental results with the aforementioned model predictions, this study emphasizes the necessity of improving existing crack width calculation models, to effectively predict the crack width of the structures with large concrete covers. The next objective is to identify the applicability of existing ‘allowable crack width limits’ for the structures with large concrete covers. A literature survey has been carried out on how the existing limitation has been appointed based on durability, aesthetic view and liquid tightness criteria. It has identified the required improvements and required further studies to decide the ‘allowable crack width limits’, to apply for the structures with large concrete cover thickness. However, although much has been published on cracking, such a study, which discusses the necessary improvements required to effectively control crack widths of structures with large concrete covers, has not been conducted.

2. Cracking phenomenon of RC members subjected to axial tension and flexure

To understand the cracking phenomenon in flexure, a reinforced concrete tie in pure tension is considered, as it can represent the tensile region of a bending member with or without any axial force [14]. Many previous researchers [15-19] have explained the cracking behavior of specimens subjected to pure tension. When the stress is transferred from the reinforcement to the surrounding concrete, a shear lag occurs (Saint-Venant's principle [20]), and this is clearly explained in Figure 1, as per Beeby’s explanation [21]. The applied stress from the rebar starts affecting the concrete surface after ‘KC’ (‘K’ is a constant and ‘C’ is cover) distance [22-24], and it takes another S_0 distance to uniformly transfer the stress along the cross section. When the applied force increases from zero, the highest stress occurs at the concrete surface after KC+S_0 distance (transfer length) and beyond. According to this explanation, the crack spacing would increase with the increase in concrete cover.

The aforementioned theoretical explanation matches with the Borges’s explanation in 1965 [25], which combines two theories: ‘no-slip’ theory [15, 26] and ‘bond-slip’ theory [27-29]. When the stress in the concrete cross section reaches the tensile strength (f_{ct}), the first crack appears. After the first crack, tl ... :arranges, as, at the crack, the concrete can no longer withstand tensile s ... ck face. When the load is further increased, another crack occurs after the ... on proceeds until the last crack occurs at the stabilized cracking



(b)

stage. Afterwards, the increased strain due to the further increased load would accumulate at the cracks that have already occurred.

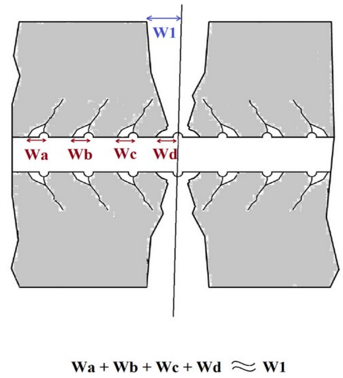


Figure 1. (a-1) Internal stress distribution of an RC tie, (a-2) Stress variation of concrete surface, (b) Internal stress distribution of cracked RC tie.

Figure 2. Crack width variation along the concrete cover.

The aforementioned explanation of the cracking is a simplified approach. As can be seen in Figure 1(b), the transferred stress from the rebar to the concrete, exceeds the concrete strength closer to the rebar. This leads to occur internal cracks (secondary cracks) at the rebar surface [14, 19, 30, 31]. Even though these cracks do not completely discontinue the concrete material, a partial discontinuity occurs [32]. This leads to a complicated stress/strain distribution throughout the specimen. However, it can be stated that the internal cracks cause the stress transfer from the reinforcement to the surrounding concrete to drop. This causes the effective length in actual conditions to increase more than in the simplified condition shown in Figure 1.

Concrete is an inhomogeneous material. The tensile strength of concrete would not be the same, even in different samples of the same batch of concrete [33]. Naotunna et al [34] have suggested including the lower and upper fractile values of concrete tensile strengths, to identify the minimum and maximum crack spacing values. The cracking phenomenon would be more complicated with conditions like effective concrete area, inhomogeneous behavior of concrete tensile strength, tension stiffening effect, internal cracking (Goto cracks), slip-bond stress behavior of reinforcement and concrete, and so on. Further, when considering the concrete members in practice, the effect of surrounding tension reinforcement, shrinkage and creep effect, stirrups, etc. must be considered.

3. Calculation of crack widths

There are various types of crack width calculation models in the existing literature. The theoretical concept of crack width is the integration of the actual strain difference of reinforcement and concrete between two cracks [35]. The crack width at the tensile reinforcement can be calculated by using Equation (1). However, due to the nonlinear behavior of strain variation in both concrete and reinforcement between two cracks, obtaining the crack width explicitly is a complicated process [36]. Therefore, in order to make the crack width calculation model less complicated or more user-friendly, many codes use simplified approaches or semi-analytical approaches. Examples of such models are in Eurocode 2 [5], Model Code 2010 [6], JSCE [4] code and so on. On the other hand, codes like ACI [7] and BS [8] use crack width calculation models based on empirical approaches. Such models are developed by curve fitting of a considerable amount of experimental data. The ACI and BS codes were developed by the experimental investigation of Gergely and Lutz [37] and Beeby [38], respectively.

$$w = \int_{0}^{S_r} \varepsilon_s - \varepsilon_c \, dx \quad (1)$$

where 'w' is the crack width, 'S_r' is the crack spacing, and 'ε_s' and 'ε_c' are the strains of reinforcement and concrete in the x-direction (the direction of axial tensile load).

Semi-analytical models developed from Equation (1) predict the crack width at the tensile reinforcement surface. It is assumed that the crack width propagates similarly, along with the concrete cover thickness, and therefore the same model is used to predict the crack width at the concrete surface [5, 6]. However, the experimental investigations in [39-41] have identified that the crack width at the concrete surface is two to ten times higher than the crack width at the rebar. Beeby [30] observed that the reason for this crack width difference is the effect of shear lag, which occurs along with the concrete cover. However, the authors in [42] proved that the effect of shear lag is considerably smaller than the crack width difference at the reinforcement and at the concrete surface. The authors in [41, 42] explained that the reason for the crack-width difference is the presence of Goto cracks [19] (secondary cracks). These secondary cracks are spread at the vicinity of primary cracks [14, 19, 43]. As the strain accumulates in the secondary cracks, the width of the primary crack at the reinforcement is reduced. Therefore, as per Figure 2, it can be concluded that the predictions of semi-analytical models in [5, 6] are similar to the surface crack width.

3.1 Crack width calculation methods in existing codes

When examining the models proposed by codes, it can be observed that some codes mention limitations for parameters like concrete cover thickness. EC2 and MC2010 mention their limitations as 70 mm and 75 mm respectively. Therefore, it was necessary to examine how these two models are developed and the reasons for mentioning such limitations. Table 1 shows both EC2 and MC2010 models and their significances.

Table 1. Crack width calculation models and the significances in EC2 and MC2010.

Model	Equations	Remarks
EC2 [5] and MC2010 [6]	w_k = Maximum crack spacing × Mean strain difference of rebar and concrete	Semi-analytical models
	Maximum crack spacing EC2 Crack Spacing Model $s_{r,\,max} = k_3 c + k_1 k_2 k_4 \varphi / \rho_{p,eff}$ $\rho_{s,ef}$ Effective steel ratio c Cover k_1 Factor for bond properties k_2 Factor for distribution of strain k_3 Recommended 3.4 k_4 Recommended 0.425 MC2010 Crack Spacing Model $l_{s,\,max} = k\,c + (1/4)\,(f_{ctm}/\,\tau_{bms})\,(\varphi_s / \rho_{s,ef})$ $s_{s,\,max} = 2.[k\,c + (1/4)\,(f_{ctm}/\,\tau_{bms})\,(\varphi_s / \rho_{s,ef})]$ k Empirical parameter on cover c Cover τ_{bms} Mean bond strength (steel-concrete) φ_s Bar diameter $\rho_{s,ef}$ Effective steel ratio f_{ctm} The tensile strength of concrete	**Assumptions** From the different bond stress models between reinforcement and concrete (linear, non-linear) between a crack and a no-slip location [28, 29, 44, 45], a constant mean bond stress has been assumed [46]. **Significance** EC2 uses a 'k_2' factor to take into account the variation in strain distributions (flexural or axial tension) [47], and MC2010 considers that only the 'effective concrete area' can represent the effect [48]. **Limitations for cover** EC2 and MC2010 limits of 70 mm and 75 mm, respectively.
	Mean strain difference of rebar and concrete EC2 Mean strain differences Crack Formation Stage $\xi_{sm} - \xi_{cm} \geq 0.6\,(\sigma_s / E_s)$ Stabilized cracking stage, $\xi_{sm} - \xi_{cm} = \dfrac{\sigma_s - k_t\,(f_{ct,eff}/\,\rho_{p,eff})\,(1+\alpha_e\,\rho_{p,eff})}{E_s}$ σ_s Stress of steel at the cracked section k_t Factor on the loading duration $\rho_{p,eff} = (A_s + \xi_1^2 A_p') / A_{c,eff}$ A_s Reinforcement area A_p' Area of post tension tendons $A_{c,eff}$ Effective area of concrete in tension $\alpha_e = E_s/E_c$ MC2010 Mean strain differences Crack Formation Stage $\varepsilon_{sm} - \varepsilon_{cm} = \sigma_{sr} / E_s * (1-\beta) - \eta_r\,\varepsilon_{sh}$ Stabilized cracking stage, $\varepsilon_{sm} - \varepsilon_{cm} = (\sigma_s - \beta\,.\sigma_{sr}) / E_s - \eta_r\,\varepsilon_{sh}$ σ_{sr} Max. steel stress at the crack formation stage σ_s Stress of steel at the cracked section β Factor on the duration of load η_r Coefficient for shrinkage strain ε_{sh} Shrinkage strain	Concrete cannot further increase its strain when the total number of cracks have formed (as the available length to develop stress in concrete is fixed). Therefore, when the strain of the steel is further increased (when reaches to the stabilized cracking stage), the concrete strain remains unchanged. This causes there to be different formulas for the mean strain difference between reinforcement and concrete in both cracking stages. **Significance** Except for the effect of shrinkage considered in the MC2010 model, both EC2 and MC2010 use the same equation in the stabilized cracking stage.

From Table 1, it is clear that both EC2 and MC2010 have mentioned limitations for the concrete cover thickness. Further, according to the literature on recent experiments, many cases can be identified in which the experimental values deviate from the EC2 and MC2010 predictions [34, 36, 49-54]. Therefore, many improvements have been proposed for these two models, some of which are listed in Table 2.

Table 2. Suggested improvements for the EC2 and MC2010 crack-width calculation models.

Literature	Improving Parameter	Suggestion	Remarks
Caldentey (2017) [48]	Mean strain difference	Include the shrinkage strain effect with a restraint factor (R_{ax}). $w_k = s_{r,max} \cdot (\varepsilon_{sm} - \varepsilon_{cm} - R_{ax}\, \eta_r\, \varepsilon_{sh})$ where 'R_{ax}' can be 1, when a member is completely restrained at the edges (e.g. wall is restrained by previously cast foundation) and 'R_{ax}' can be 0, when restrained at the ends (e.g. RC tie subjected to axial tension).	The authors are investigating on the effect of 'casting position' [55] from the experimental results of [56].
Debernardi and Taliano (2016) [43] Taliano (2017) [31]	Crack spacing	From the local equilibrium of the stabilized cracking stage and the derivation of average bond stress, $\tau_{bms} = (f_{ct}.A_c)/ (n_s.\pi.\varphi_s.L_s)$, where n_s is the number of tensile bars. $s_{r,max} = 2.\ L_s = 2.\ (1/4)\ (f_{ctm}/ \tau_{bms})\ (\varphi_s / \rho_{s,ef})$. The author suggests a table of values for the 'τ_{bms}/ f_{ctm}' from the suggested 'general equation' developed by Balazs (1993) [35]. It is important to identify that the 'concrete cover' has no influence on the crack spacing, in this method.	Experimental comparison is made up to 45 mm of cover. The suggested method gives more conservative values than the experimental values.
	Mean strain difference	In order to represent the reduction of tension stiffening, due to internal cracks, the 'k_t' coefficient is considered to be 0.45 (which is 0.6 for the short-term load suggested by EC2). $\varepsilon_{sm} - \varepsilon_{cm} = \dfrac{\sigma_s - 0.45\ (f_{ct,eff}/ \rho_{p,eff})\ (1+ \alpha_e\ \rho_{p,eff})}{E_s}$	
Rospars and Chauvel (2014)[50] Bisch (2017) [51]	Crack spacing	After a statistical analysis of the results of 131 tests from own experiments (CEOS project France) and previous literature, an equation has been identified that gives better agreement with experimental results. $s_{r,\ max} = 1.7\ [1.37\ c + 0.117.(\varphi_s / \rho_{s,ef})]$	Covers of the experimental specimens are 50 mm and 70 mm.

According to the suggested improvements mentioned in Table 2, it can be identified that none of the improved models has compared the data with specimens with above 70 mm of cover thickness. Therefore, the applicability of the aforementioned improved models needs to be verified for concrete covers larger than 70 mm. Furthermore, many researchers have identified that the 'crack spacing' model is the governing part in crack-width calculation models, and improving that part is vital [57-59]. Tammo and Thelandersson [57] proved that changing the concrete properties makes no difference to the surface crack width or internal crack widths, if the crack spacing values are the same. Further, many researchers have experimentally [26, 42, 49, 60] and analytically [27, 30] identified that the 'bond-slip theory' has a low influence or no influence on crack spacing and, therefore, on the crack width. The literatures [61, 62], which are from the same authors of this manuscript has further discussed on this scenario. From a detailed literature survey, it had identified that the 'slip' in axial tensile specimens are negligible [61]. This emphasizes to reconsider the effect of the term '$\varphi_s / \rho_{s,ef}$' term which comes from the bond-slip theory. However, when the suggested improvements listed in Table 2 are considered, it seems that all of them have considered the effect of '$\varphi_s / \rho_{s,ef}$' term. Considering these findings, it can be concluded that the existing/suggested crack spacing models should be improved, by taking into account the parameters that have a significant influence on crack spacing.

Table 3. Crack width calculation models in JSCE, ACI and BS codes and the significances.

Model	Equations	Remarks
JSCE code[4]	$w¿1.1\, k_1 k_2 k_3 \{4c+0.7(c_s-\varnothing)\}[\frac{\sigma_{se}}{E_s}+\varepsilon'_{csd}]$ $k_2=\frac{15}{(f'¿¿c+20)+0.7¿}$ $k_3=\frac{5(n+2)}{7n+8}$ w crack width, c concrete cover k_1 constant on the surface of rebar (1.0 for deformed and 1.3 for plain bars), k_2 constant on the concrete quality on crack width, f_c design compressive strength of concrete, n number of layers of tensile rebar k_3 constant to take account of the multiple layers of tensile bars, c_s distance of the tensile rebar (from center to center), $\varnothing$ the diameter of the tensile rebar, σ_{se} tensile stress increment of the rebar E_s Young's modulus of steel and ε_{csd} compressive strain from shrinkage and creep of concrete.	This model is based on a semi-analytical approach. Crack spacing model (without strain components) is based on the concrete cover and the distance between tensile bars. Bar spacing has been proved a factor for crack spacing in [63]. The experimental findings in [64] prove that smooth bars cause large crack spacing. While both EC2 and MC2010 predict increasing crack width with concrete strength, JSCE code predicts the opposite. However, this behaviour matches with the results in [52, 65, 66]. **Limitations for concrete cover thickness:** No limitations have been mentioned for concrete cover thickness.
ACI code [7, 67]	$w=2.2\beta\varepsilon_s\sqrt[3]{d_c A}$ w most probable maximum crack width at the extreme tensile fiber (inches), β ratio of the distance between the neutral axis and tension face to the distance between neutral axis and centroid of reinforcing steel, ε_s strain in reinforcement due to the applied load, d_c thickness of cover from extreme tension fiber to the closest bar (inches), A area of concrete symmetric with reinforcing steel divided by the number of bars (square inches).	The empirically based equation was developed in [37] with the results of six different bending experiments. The ACI Committee 224 [67] modified the aforementioned model by using the strain, instead of the stress in the reinforcement. **Limitations for concrete cover thickness:** No limitations have been mentioned for concrete cover. However, the results of specimens with up to 84 mm of concrete cover were used to develop the model.
BS code [8]	w = $\frac{3Ce}{1+2(\frac{C-C0}{d-dn})}$, where $e=\left(est-\frac{2.5bd*10^{-6}}{Ast}\right)\frac{d-dn}{d1-dn}$ c distance from the point considered to the nearest bar, d overall depth of the member, d_n neutral axis depth calculated on the assumption that concrete has no tensile strength, d_1 effective depth of a member, B breadth of the member, A_{st} area of the tensile steel, e_{st} strain in the steel, assuming concrete has no tensile strength.	The empirically based equation was developed based on the experiments conducted in [38]. The derived equation in [38] has been simplified in [68] to be used in the BS code. Results showed that crack width is linearly proportional for concrete covers below 40 mm, and the pattern differs when the cover increases. **Limitations for concrete cover thickness:** No limitations have been mentioned for concrete cover. However, the results of specimens with up to 89 mm concrete cover were used to develop the model.

Table 3 describes the important information on the crack width calculation models in JSCE, ACI and BS code. From the models proposed by codes in Table 1 and Table 3, various crack-width governing parameters can be identified. A detailed list of such parameters and their involvement in crack width can be identified from the literature [69], which is from the authors of this paper. Table 4 shows the included crack-width governing parameters in the mentioned code models. Although the aforementioned models have been developed based on different approaches, the concrete cover thickness parameter is included in every model.

Table 4. Crack width governing parameters included in models proposed by codes.

Parameter	EC2	MC 2010	JSCE	ACI	BS
Concrete cover	x	x	x	x	x*
φ / $\rho_{p,eff}$ (diameter to effective steel ratio)	x	x			
Tensile strength of concrete		x			x
Concrete compressive strength			x		
Steel stress/strain	x	x	x	x	
Bond properties between steel and concrete	x	x			
Rebar diameter	x	x	x		
Effective concrete area	x	x		x	
Reinforcement area	x	x			x
Rebar surface geometry			x		
Maximum to minimum crack spacing	x	x			
Loading condition (axial tension or bending)	x				
Member size					x
Neutral axis position				x	x
Duration of loads (short- or long-term)	x	x			
Shrinkage/creep of concrete		x	x		
Young’s modulus of steel	x	x	x		
Young’s modulus of concrete	x	x			
Rebar layers			x		
Distance between tensile rebar			x		
Number of rebars				x	
Note ‘x’ denotes that the mentioned parameter is included in the crack width calculation model. * Distance between the crack width measuring location at the concrete surface and the nearest reinforcement surface.					

From Table 4, it can be identified that the EC2, MC2010 and JSCE code models, which are based on a semi-analytical approach, consider a higher number of parameters than the empirically-based ACI and BS code models. Further, it is clear that, although the mentioned models have been developed based on different approaches, the concrete cover thickness parameter is included in every model. The calculated crack width from these models causes the crack width to increase with the increase in concrete cover. The models suggested to calculate the crack width in EC2 and MC2010 specifically mentioned their applicable limitations for concrete cover thickness. The models in the JSCE, ACI and BS codes do not mention such limitations. The main reason could be that the commonly used concrete cover thickness in the period of developing the code might be not as large as the current requirement. It is important to note that the empirically-based crack width calculation models developed by ACI and BS codes have considered test specimens with concrete cover thicknesses of 84 mm and 89 mm respectively. However, as mentioned earlier, there is a demand for large concrete cover thickness for those RC structures expected to be built in environments of adverse exposure [11]. As the first step to check the predictability of the crack widths of specimens with large concrete covers, model comparison is conducted with the recent experimental results.

4. Comparison of the crack width calculation model predictions with the specimens with large concrete covers.

There is a limited number of previous studies that have observed the cracking behavior of RC specimens with large concrete cover thicknesses. Among them, axial tensile experiments were selected from [36, 54, 70, 71], because they represent the tensile region of a bending specimen [14]. From the study conducted by Tan et al [36, 54], the cracking behavior of two specimens with 40-mm and 90-mm cover thickness have been considered. From the study by Dawood and Marzouk [70, 71], four specimens have been considered in two sets. The details of the selected specimens are listed in Table 5. It is important to notice that the specimens in Tan et al (2018) are loaded from the concrete, while the specimens from Dawood and Marzouk (2011) are loaded from the reinforcement. The crack widths mentioned in Tan et al (2018) are the 95 percent fractile of measured crack width at the concrete surface above tensile reinforcement. As specified by Dawood and Marzouk (2011), for the crack width at the concrete surface above tensile reinforcement, the mentioned values in the literature have been multiplied by a factor of 0.7.

Table 5. Details of the selected specimens.

Study	Specimen width×height× length (m× m×m)	No	Concrete strength (Mpa)	R/f ratio (%)	Concrete cover (mm)	Number of bars in a specimen	Bar diameter (mm)	Crack width at 2/3. fy * (mm)
Tan et al (2018)	0.4 × 0.4 × 2.0	1	74.3	1.60	40	8	20	0.22
		2	74.3	1.60	90	8	20	0.34
Dawood and Marzouk (2011) Set 1	0.9 × 0.26 × 0.9	3	75	1.20	37.5	6	25	0.16
		4	75	1.20	62.5	6	25	0.20
Dawood and Marzouk (2011) Set 2	0.9 × 0.38 × 0.9	5	65	1.20	45	6	30	0.18
		6	65	1.20	75	6	30	0.23
Note * f_y is the yield strength of reinforcement								

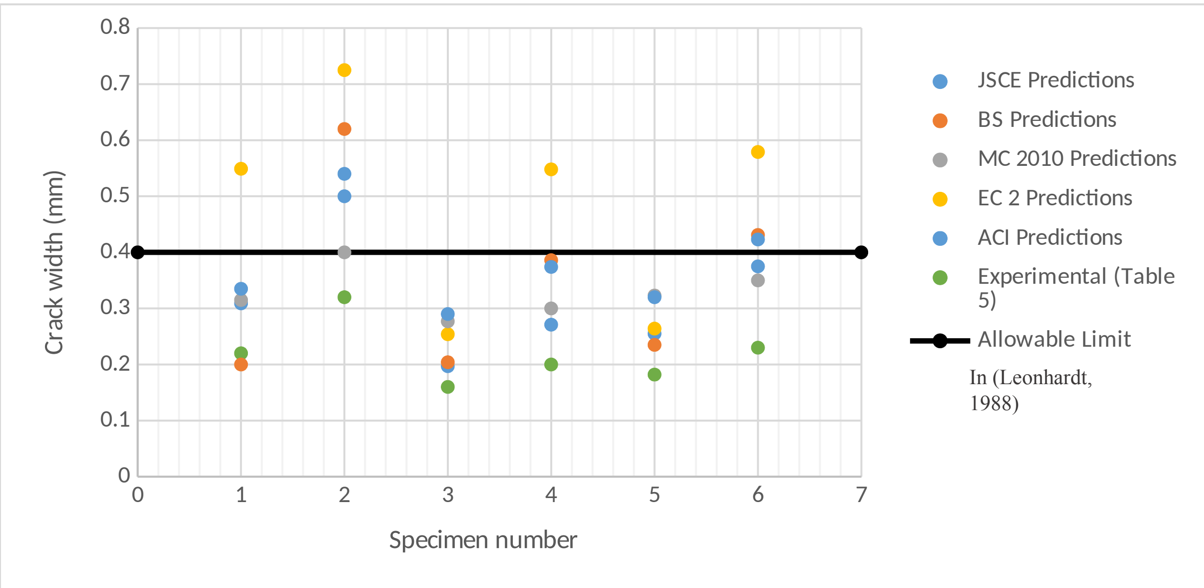


Figure 3. Experimental and code-predicted crack widths of the selected specimens in Table 5 at the service load (2/3. fy).

The experimental crack width of the mentioned specimens and their predicted crack widths from the codes at the service stress are given in Figure 3. As specified in the ACI code [7], the serviceability limit of steel stress is considered to be 2/3 × f_y, where f_y is the yield strength of reinforcement. Leonhard [1, 17] has mentioned, that for the structures made with quality concrete and with necessary concrete cover the allowable crack width limit can be 0.4 mm. This limit is considered as a reference for this study in section 4. However, the current limitations on the allowable crack width are discussed in section 5.0. Specimens 1, 3 and 5 have concrete covers of 40 mm, 37.5 mm and 45 mm, respectively. When considering Figure 3, it is clear that, for these specimens, both the experimental and the code-predicted crack widths lie below the specified allowable crack width limit (except for the EC2 prediction of specimen 1). Specimens 2, 4 and 6 have concrete covers of 90 mm, 62.5 mm and 75 mm, respectively. The experimental crack widths of these specimens lie below the mentioned allowable limit. However, as in Figure 3, almost all the code predictions for specimen 2, the EC2 prediction for specimen 4, and the EC2, JSCE and BS code predictions for specimen 6 have predicted crack widths above the allowable limit. According to the aforementioned code predictions, the specimens would require additional tensile reinforcement to limit their crack widths to below the allowable limit. However, as the experimental crack widths lie below the allowable limit, the actual specimens do not require additional reinforcement to limit the crack width.

From this study, a clear difference can be witnessed in the predictions of crack widths of specimens with relatively small concrete covers and those with relatively large concrete covers. This parametric study emphasizes the requirement to improve existing crack width calculation models, in order to predict the crack widths of specimens with large cover thickness.

5. Allowable crack widths in the existing codes

From the previous discussion, it has been proved that the crack width increases with the concrete cover thickness (Section 3 and 4). When considering the allowable crack width limits in the discussed codes, with the exception of the JSCE code, every other code's allowable limit does not increase with the concrete cover thickness. The allowable crack width limits of an RC structure (in the absence of a water tightness requirement) have been decided for durability and aesthetic acceptance. For liquid storage RC structures, special attention is required on tightness, and the values of crack width limits are defined separately.

It can be observed that the prescribed allowable crack width limits in the codes have been changed from time to time. For example, Model Code 1978 [72] and MC 90 [73] recommend 0.1-mm and 0.3-mm crack widths, respectively, for severe exposure classes. Further, the allowable limits in each code differ from each other. For structures exposed to adverse environmental conditions, EC2, MC2010 and BS codes recommend limiting the crack widths to 0.3 mm (Table 7.1 N in EC2, Cl. 7.6.4.1.4 in MC2010 and Cl. 3.2.4 in BS codes). Moreover, for severe exposure conditions, the ACI 318 code recommends limiting the crack width to 0.33 mm (Cl. 10.6.4), and the ACI 224 report recommends limiting it to 0.15 mm (Table 4.1 in ACI 224R). However, the Norwegian National Annex [74] follows slightly different criteria than EC2. It has introduced a k_c coefficient ($k_c = c_{nom} / c_{min,dur} \leq 1.3$) and allows the EC2-specified crack width limit to be multiplied by the k_c coefficient. The allowable crack width limit of the JSCE code is shown in Table 6.

Table 6. The limit value of crack width as per JSCE standards (Table 8.3.2 in JSCE standard).

	Environmental condition		
	Normal	Corrosive	Severely corrosive
Deformed bars and plain bars	0.005c	0.004c	0.0035c

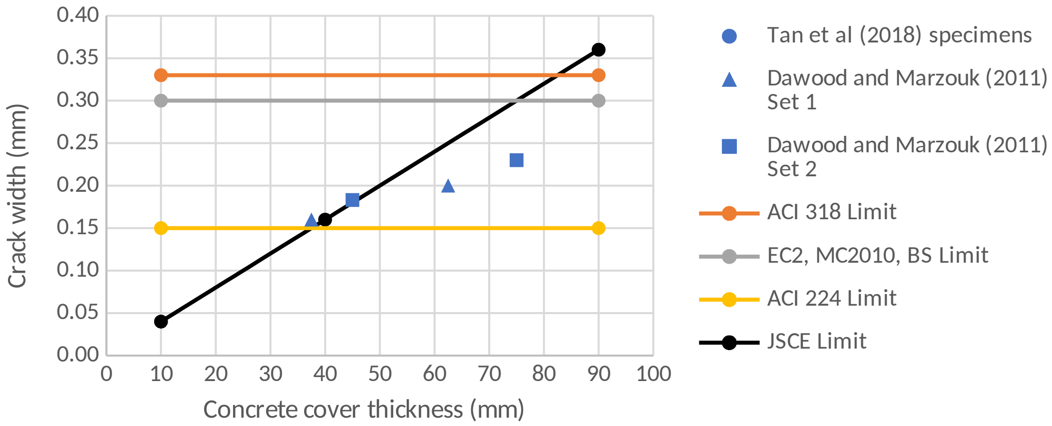


Figure 4. Experimental crack widths of the specimens from Table 5 and allowable crack widths of different codes in adverse environmental conditions.

To compare the applicability of allowable crack width limits of specimens with large cover thicknesses, the discussed experimental results in the parametric study (Table 5) have been considered. According to Figure 4, none of the specimens has a crack width under the limit of ACI 224 guidelines. The EC2, MC2010 and BS codes have a similar limit for the allowable crack width (0.3 mm) for specimens in adverse environmental conditions. ACI 318 specifies a crack width limit of 0.33 mm for specimens in adverse environmental conditions. With the exception of the specimen with a cover of 90 mm (specimen 2 in Table 5), every other specimen is under the limit of the EC2, MC2010, BS and ACI 318 guidelines. Among the listed codes, the only code which changes the allowable limit with the concrete cover is the JSCE code. According to the allowable limit in the JSCE code, except for the specimen with the 40-mm cover (specimen 1 in Table 5), every other specimen crack width lies on or within its allowable limit.

Therefore, in order to identify the most suitable allowable crack width limit, it is important to investigate the reasons for the aforementioned differences in each code. Further, in order to identify the effect of concrete cover thickness on the allowable crack width, a literature survey has been carried out. The focus is to identify how the existing limits are placed and to check whether the increased crack width of specimens with the increase in concrete cover has an effect on the durability, aesthetic aspect and liquid tightness of an RC structure

5.1 Crack width limitation considering the durability

There is consensus that cracks appearing in reinforced concrete structures lead to penetrate CO_2, chloride and other corrosive agents to the steel and can initiate reinforcement corrosion [3, 75]. This reinforcement corrosion could lead to a reduction in the amount of steel in the reinforcement and the corrosive products expanding in volume. When the amount of steel is reduced, the expected performance of the structure decreases, and, when the corrosion products increase in volume, they cause cracking and spalling of the concrete. To reduce the adverse effect of cracking, the current practice is to limit the width of the crack. Further, increasing the concrete cover is one of the main measures that has been identified to enhance the durability of an RC structure. However, as per the previous discussion, the crack width also increases with the simultaneous increase in concrete cover. This reveals that the discussed actions considered to increase the durability contradict one another. Therefore, in order to identify how the existing allowable crack width limits are decided, based on the durability, a literature survey has been carried out.

5.1.1 Previous studies on crack width and rebar corrosion

In the available literature, various types of experiments can be identified, which have studied the effect of crack width on rebar corrosion. However, when considering the results of some of these experiments, the effect of crack width on reinforcement corrosion is quite complicated. Depending on the experimental duration and the outcome of the results of the available experiments, the authors have divided them into four categories: 1. the 'crack width' does not have a 'relatively short-term' effect on corrosion; 2. the 'crack width' has a 'relatively short-term' effect on corrosion; 3. the 'crack width' does not have a 'long-term' effect on corrosion; and 4. the 'crack width' has a 'long-term' effect on corrosion, as given in Table 7. Experiments conducted for up to 10 years are categorized as 'relatively short-term' experiments; those which have continued for longer or experiments conducted for more than 10 years are considered 'long-term experiments'.

Table 7. The details and the outcome of previous experiments on crack width and corrosion.

Category [1]	Experiment	Experiment Details					Results	Remarks
		Specimen sizes in mm	Cover (mm)	Crack width (mm)	Exposure	Period (years)		
1.	Makita et al (1980) [76]	Length 750		0.05- 0.3	Seawater	2.7	Corrosion had no relationship to crack width.	Specimens were unloaded during exposure.
	Lin (1980) [77]	914× 76× 152		0.1 0.15 0.18	Seawater	2 - 10	Crack width does not influence the amount of corrosion.	Specimens were loaded during exposure.
	Tremper (1947) [78]	200× 200× 63	28.6	0.127 0.254 0.508	Coastal exposure	10	Corrosion only in cracked locations.	Specimens were unloaded during exposure.
	Francois and Arliguie (1991) [79]	3000× 150× 280		< 0.5	NaCl & CO_2 prone	10	No relationship to crack width.	
	Berke et al (1993) [80]	762× 152× 152	38	0.2 (mean)	NaCl solution	1.3	Corrosion in cracked and uncracked locations.	Specimens were unloaded during exposure.
	Kahhaleh (1995) [81]		50	around0. 33	NaCl solution	1.1	Corrosion had no relationship to crack width.	Both loads held and were unloaded during exposure.
	Chen et al. (2020) [82]	1100× 180× 100	30	0.1- 0.4	NaCl solution	3	Cracks induce corrosion, but no correlation with the crack width.	Beams with FRC has a lower corrosion level than plain concrete.
2.	Ohta (1991) (i) [83]	1000× 150× 150	20 40	0-0.1 0.1-0.2 0.2-0.3	Coastal	10	20-mm cover, every cracked location (0-0.3 mm) is similarly corroded. 40-mm cover, corrosion and crack width are related.	40-mm cover, 0-0.1-mm cracks show the minimum corrosion.
	Schiessl (1976) (i) [84, 85]	1950× 250× 150	25 35	0.075- 0.55	Mixed	4	Corrosion and crack width are related.	
	Carevic and Ignjatovic (2019) [86]	500× 100× 100	25	0.05 0.1 0.15 0.2 0.3	2% CO_2 with 65% humidity	0.1	The carbonation depth was 3 times lower in uncracked specimens.	Corrosion is three times higher in 0.3-mm cracked locations.
	Schiessl and Raupach (1997) [87]	700× 97× 150	15	0.1 0.2 0.3 0.5	Saltwater	2	Corrosion increases with increasing crack width.	Concrete cover and w/c ratio are more dominant for corrosion than

								cracks.
	Swamy (1990) [88]	Length 760	50 70	0.11-0.25	Marine		Crack width above 0.15 mm shows corrosion.	
	Misra and Uomoto (1991) [89]	2100× 100× 200	10		Marine	1	Crack width above 0.5 mm shows severe corrosion.	
	Vennesland and Gjoro (1981) [90]	500× 100× 100		0.1- 2.0	Seawater	0.3	Crack width above 0.5 mm shows severe corrosion.	
	Miyagawa (1980) [91]	1000× 50× 50	20	< 0.3	NaCl solution		Crack width above 0.2 mm shows corrosion.	
	Li et al. (2017) [92]	400× 100× 100	40	0 – 0.5	NaCl solution	1.8	Crack width correlates with the corrosion amount.	Plain bars were used.
	Houston et al (1972) [93]		25 50 75		NaCl solution	2.8	50 and 75-mm covers, observed corrosion above 0.13-mm crack widths.	25-mm cover, corrosion in both cracked and uncracked.
3.	Ohta (1991) (ii) [83]	1000× 150× 150	20 40	0-0.1 0.1-0.2 0.2-0.3	Coastal	20	Corrosion and crack width are not related.	Every cracked location is similarly corroded.
	Schiessl (1976) (ii) [84, 85]	1950× 250× 150	25 35	0.075-0.55	Mixed	10		
4.	O'Niel (1980) [94]		19 50	0 - 0.4 Above 0.4	Tidal wave with freeze and thaw	25	Corrosion observed in cracks above 0.4 mm. 11/82 specimens survived to be tested after 25 years.	Survived specimens were of 19-mm cover and with air-entrained concrete.

Notes * Length × width × height of test specimens

Category[1]

1. Crack width has no effect on corrosion (relatively short-term), 2. Crack width has an effect on corrosion (relatively short-term), 3. Crack width has no effect on corrosion (long-term), 4. Crack width has an effect on corrosion (long-term)

Prior to the comparison of the conclusions of the different experiments in Table 7, it is important to mention that the concrete quality, cover, exposure condition, method of corrosion measurement, method of crack generation and so on differ in each piece of the mentioned research. When considering the experiments of category 1, the conclusion is that the cracks cause the initiation of corrosion, regardless of the crack width. The authors have observed a similar amount of corrosion in locations with different crack widths. However, it is important to identify that most of the experiments categorized in category 1 had released the load during exposure. Therefore, even where the surface crack width remains open, there is a possibility of closing the internal crack. This could be a reason why a similar amount of corrosion is generated at cracks with different surface crack widths. The experiment in category 4 concludes that the crack width has an effect on long-term corrosion. However, the specimens tested in the experiment used air-entrained concrete, and only 11 specimens out of 82 were able to be tested, due to excessive damage. It is quite impossible to explain the damage to this number of mentioned specimens within 25 years (service life), with the conventional method of corrosion. This creates a conflict as regards using these results in ordinary concrete structures.

By observing categories 2 and 3, it can be concluded that the cracks initiate corrosion and, at this stage, the 'crack width' plays a vital role. However, when the testing time increases, the crack width does not influence corrosion. This could be the main reason why MC 1978 prescribes limiting the crack width in severe conditions to 0.1 mm and releases it in MC 90 and MC2010 to 0.3 mm. It can be assumed that MC 1978 had considered the short-term tests, and this limitation was changed after considering the results of long-term experiments. In order to identify the reasons for such results, it is vital to understand the corrosion mechanism in RC structures. According to [95], it is clear that the protective layer around the rebar tends to be damaged when the carbonation or chloride layer reaches the rebar of uncracked sections.

When there are cracks in concrete, the time required to penetrate the carbonation or chloride layer to the rebar is drastically reduced, and corrosion can be initiated in the early stages [3]. Figure 5 shows the penetration depth of impurities (carbonation, chloride, etc.) at a cracked and an un-cracked location. As per Figure 5, t_1 and t_2 indicate the corrosion initiation time in cracked and un-cracked sections, respectively. Therefore, even the cracks cause the corrosion to be initiated proportionately to the crack width; as time goes on, there is no difference in the amount of corrosion in locations with smaller crack widths and larger crack widths or in un-cracked locations. This theoretical explanation matches the corrosion mechanism observed in the category 2 and category 3 experiments shown in Table 7.

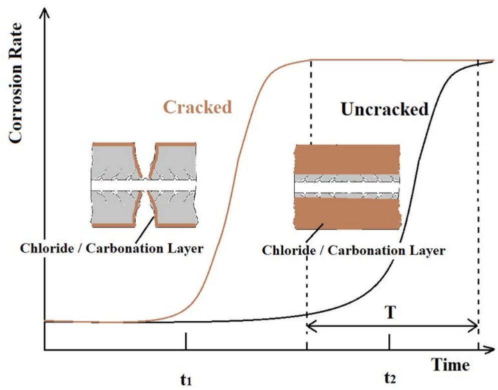


Figure 5. The time difference in corrosion initiation at a cracked and an un-cracked location.

5.1.2 Deciding the allowable crack width limits on durability

According to the experimental results and explanations, it is a complicated task to decide an allowable crack width limit, considering the durability. However, Schiessl's experiment mentioned in the report in [84] has been considered by many researchers in the field; it tried to elaborate criteria for the limiting value of crack width. In the mentioned study, the level of corrosion in the reinforcement is categorized, based on the measured corrosion height ('t_m' - based on the prepared 'rust calibration scale' by the author in [84]), as 'passive corrosion' ($t_m < 0.01$ mm) or 'active corrosion' ($t_m > 0.01$ mm). For specimens exposed for four years, active corrosion could be observed from crack widths of 0.125 mm onwards. The admissible crack width for a 25-mm cover in a 'corrosive environment' is 0.1 mm in the JSCE code and 0.15 mm in the ACI 224 code. The limit value of the JSCE and ACI 224 codes ensures that there are no active cracks in the aforementioned experimental results. However, if the limit value in EC2, MC2010 and BS codes, which is 0.3 mm, is considered, it could be observed that a considerable number of active cracks are present in the specimens.

5.1.3 Allowable crack width limits considering the durability of specimens with different concrete covers

The study conducted by Schiessl mentioned in the report in [84] tried to emphasize the possibility of increasing the limit of allowable crack width, with the increase in concrete cover. For specimens exposed for 10 years, active corrosion could be observed, even at un-cracked locations. However, Schiessl identified that, when the concrete cover is 25 mm, 66% of cracks are active in corrosion when the crack width is 0.3 mm. When the concrete cover is increased to 35 mm, only 50% of cracks are shown to be active in corrosion for a 0.3-mm crack width. Based on the results of this long-term experiment, it can be stated that the increasing concrete cover has the potential to increase the limit of allowable crack width.

5.2 Crack width limits considering the aesthetic aspect

Each code of practice has specified the allowable crack width limits, based on the structure's exposure class. When deciding this allowable limit for structure's built-in environmental conditions, where there is no risk of corrosion, the limits are given in consideration of the aesthetic acceptance of the structure. Most of the time, although RC structures are designed and constructed by experts in the field, they are used by ordinary citizens, who do not have any expertise or knowledge in the field. Therefore, users should always feel that it is safe to use these RC structures. It is obvious that unsatisfactory appearance, due to cracks, causes safety alarms and lowers the acceptance of a structure [96]. However, the aesthetic acceptance of cracking is one of the research areas which has attracted the least attention [97]. Leonhardt [1] stated that, if the structure has a necessary amount of cover with good quality concrete, a crack width of 0.4 mm is not harmful to its durability (corresponding with the outcome of [94]), but, in order to avoid unnecessary concern among casual observers, the crack width should be limited to 0.2 mm. However, it is not possible to state a fixed value for all types of RC structures and for every type of user, as the viewer's attitude can have a greater influence than what is actually observed [98]. On the other hand, it is not possible to limit the widths of controllable cracks to a very fine level, as this would increase the cost of the structure. To justify the statement that the user's attitude is of greater influence than the actual effect of cracks, the study performed by Padilla and Robles [99] gives good agreement. The study was based on cracks in a low-cost housing scheme and clearly emphasized how the different sizes of crack widths affected tenants, landlords or engineers. Figure 6 illustrates different observers' attitudes towards a crack and the actual effect of a crack on a structure.

5.2.1 Allowable crack width limits of structures with different cover thicknesses

The concrete cover thickness is decided on to protect the reinforcement against corrosion, for the safe transmission of bond forces and for adequate fire resistance [5]. Therefore, even for structures that are not threatened by corrosion, large covers can be decided on, due to the safe transmission of bond forces and for adequate fire resistance. For example, according to EC2, concrete covers can be large as 56 mm for cases with bundled bars. The surface crack width increases with the increase in concrete cover thickness. The limit of visibility of cracks is expressed by 'crack width' [1, 96], and a proper guide should be available to the client to decide the allowable crack width of controllable cracks. The study conducted by Campbell-Allen, mentioned in the report in [96], identified that the minimum crack width of a structure is a function of viewing distance, a structure's prestige and the nature of the surface (the visibility of cracks changes when they are wet or filled with impurities). The authors proposed nine categories of structures, depending on their prestige, and graphically interpreted the acceptable crack widths, depending on the distance of the viewer. The most highly prestigious buildings, such as monumental buildings, have a scale of 9, while little-used storage buildings are categorized into the lowest prestige level of 1. The proposed criterion is mentioned in Figure 7.

The outcomes of the aforementioned study [96] can be extended for every type of structure and used to estimate the allowable maximum crack width in respect of the aesthetic aspect. Every structure (or part of the structure) can be categorized into different prestige levels, depending on its usage (purpose of the structure and number of users). For example, monumental towers, pedestrian bridges, etc. can be categorized as 'higher prestige level' and structures, while dams, offshore structures, highway bridges and offshore structures can be categorized as structures with a 'lower prestige level'. Then, the client can identify the scale of the structure from 1 to 9 and the average viewing distance, depending on the actual usage, to measure the allowable crack width limit as per aesthetic satisfaction. It can be concluded that, for structures categorized at the higher prestige level, the increasing concrete cover thickness causes a comparatively higher amount of tensile reinforcement to be required, to limit the crack width.

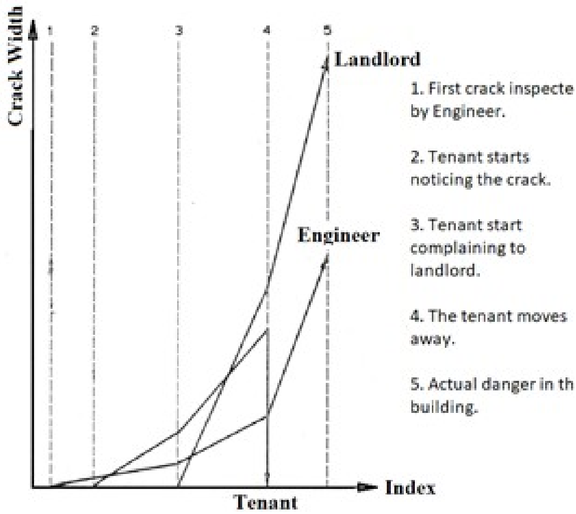


Figure 6. Different observers' attitude to a crack (adapted from Padilla and Robles 1971).

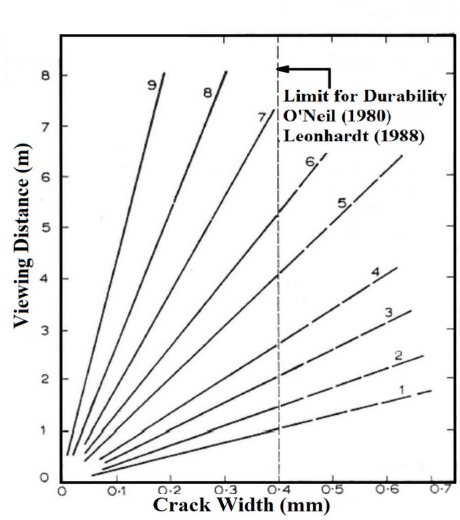


Figure 7. Aesthetically acceptable crack width (adapted from Campbell-Allen 1979).

5.3 Crack width limits for the liquid-retaining structures

RC structures designed to store liquids require additional concern regarding cracks. It is experimentally proven in much of the literature that through cracks can cause leakages or increase the permeability of a structure [100-102]. Therefore, when designing liquid- or gas-retaining structures, serviceability checks, such as limiting the crack widths, tend to dominate the design [103]. However, many previous researchers have identified that cracks with a limited width in concrete have an autogenous self-healing ability [104]. Self-healing of a structure can occur, due to precipitation of calcium carbonate, continued hydration of concrete, stagnation of debris and so on [103]. The British standard for liquid-retaining structures [105] recommends a crack width of 0.2 mm for structures without a high-pressure flow. Further, the *Reinforced Concrete Designer's Handbook* (11^{th} edition) [106] states that a 0.2-mm wide crack would autogenously heal in 21 days, and it would take only seven days to heal a 0.1-mm wide crack. However, Lohmeyer, Meichsner, EN 1992-3 [107] and so on state that the self-healing of a crack is a function of water head (water head to the width of the tank), and Figure 8 represents the criteria.

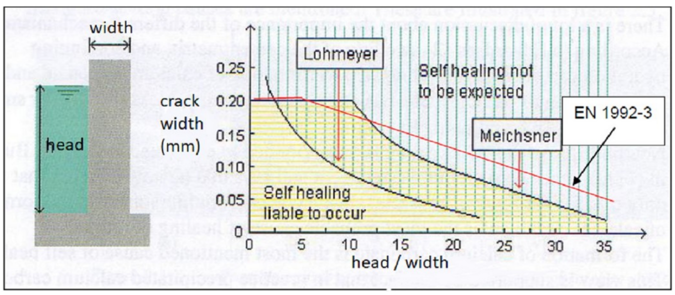


Figure 8. Self-healing of cracks according to different models (adapted from [103]).

The retaining structures can be used to store normal liquids such as water and, at the same time, they can be used to store poisonous substances or those that react with moisture, oil storage and so on. Therefore, depending on the type of requirement, Euro Code 3 [107] categorized tightness classes from 0 to 3. Class 0 is for structures where the leakage is irrelevant, while class 3 covers structures where complete leakage and staining are to be avoided. Class 1 accepts a certain amount of leakage (allows some through cracks) and follows the autogenous healing criteria mentioned in Figure 8. Classes 2 and 3 do not allow for through cracks and, therefore, limit the minimum thickness of the compression zone, to ensure no through cracks.

From the previous discussion, it has experimentally (Section 4) and theoretically (Section 3) been proved that increasing the concrete cover thickness causes the surface crack width to increase. Therefore, increasing the concrete cover can cause the crack width to exceed the prescribed limitations for autogenous healing. If the liquid-retaining structure is designed to the current guidelines, additional tensile reinforcement is needed to limit the crack width. However, it has been identified that the autogenous healing methods can be improved by using different additives [108, 109]. For example, [108] identified that, by using an 'ion chelator' admixture, cracks of up to 0.4-mm width can be healed. At the same time, modern waterproofing techniques used at the construction stage are becoming advanced [110, 111]. In order to consider that the combination of modern waterproofing techniques and admixtures can release the allowable crack width limits of the retaining structures, further confirmation studies are required.

6. Identified future research

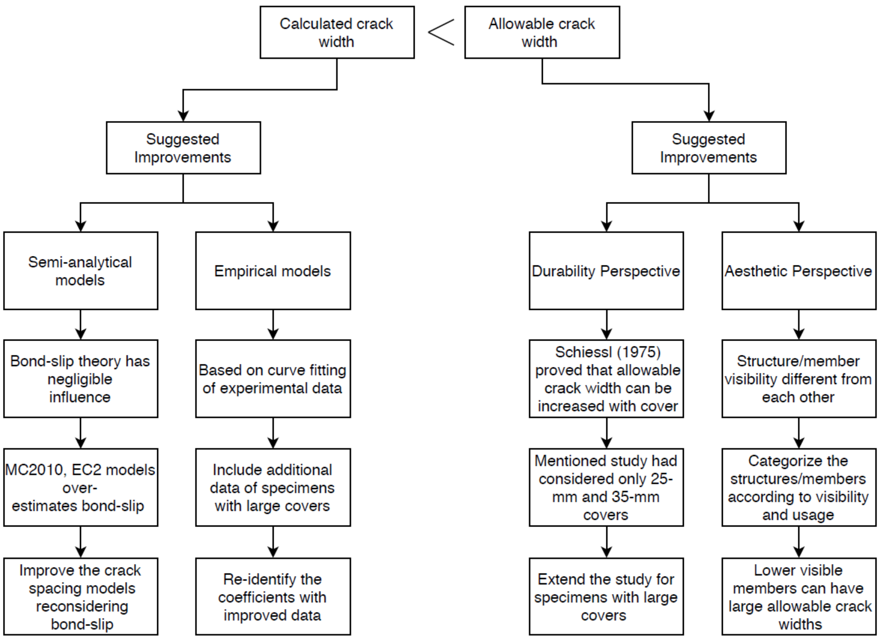


Figure 9. Identified improvements for the existing crack controlling criteria to effectively control the cracks of structures with large concrete covers

Identified future researchers for the existing crack controlling criteria to effectively control the cracks of structures with large concrete covers, are briefed in the flowchart given in Figure 9. Both EC2 and MC2010 crack width calculation models have specified limitations for concrete cover thickness. Further, a model comparison in the section 4 highlighted that both models could not predict the crack widths of specimens with a large cover thickness. The authors have identified that these models have overestimated the effect of bond-slip theory. A study is continuing by the authors of this paper, based on these facts, to identify a better crack width calculation model. In order to improve the empirically based crack width calculation models in ACI and BS codes, to predict crack widths in large cover thicknesses, additional crack width data from experiments with large covers can be considered. Then, the existing coefficients can be adjusted, to match with the data from specimens with large concrete covers.

Allowable crack width limits are decided based on durability and aesthetic acceptance of the RC structure. Among many previous studies, the study reported in [84] has proved with long-term results that the concrete cover plays a critical role in durability. At the same time, with the results of RC specimens with 25-mm and 35-mm cover thicknesses, it has proved that the allowable crack width limit can be increased with the increase in cover. Therefore, this study can be extended to check the corrosion of specimens with large concrete cover thicknesses. Further, it is a fact that crack width increases with the concrete cover, and that is due to the increase in crack spacing. This means that, although the crack width increases, the number of cracks also reduces (as the crack spacing becomes larger with the cover). This effect also has to be considered, when deciding an allowable limit for structures with large concrete covers. The final aim could be to introduce a method to ensure an effective allowable limit for structures with large covers.

When there is no risk of corrosion, the allowable limits are based on aesthetic appearance. The authors suggest extending the study conducted in [96] for every type and part of the structure. A study can be carried out to categorize the structures or parts of structures (members) into different prestige levels and decide the allowable crack width limit accordingly. For example, if a bridge with no risk of corrosion is considered, a lower crack width limit can be placed on the parts that pedestrians use, while a higher crack width limit can be set for members like girders and piers, where pedestrians do not make any contact.

7. Summary

Widely used crack width controlling methods have been considered in this study. All the discussed methods control the adverse effect from cracks, by limiting the calculated crack width to a prescribed allowable crack width. Crack width calculation models have been clarified, starting with the cracking phenomenon. However, when considering the applicability of such models, for structures with large covers, it was identified that some models have limitations for the concrete cover thickness. The focus was then on how these models are developed; then, after conducting a model comparison, it was identified that improvements are needed, to predict the crack widths of specimens with large covers. The necessary improvements and research gaps on how to improve these models have been clearly mentioned.

When focusing on the allowable crack width limits of the discussed models, they seem to differ from each other. Further, these limitations have been identified as changing from time to time. In order to identify the reasons and to check whether these limitations are applicable for large covers, a literature study was carried out. When considering the crack width limit from the perspective of durability, the authors categorized the existing studies into four categories and examined the reasons separately. It was identified that even the crack width plays a critical role in rebar corrosion at the early stage; when the time increases, this behavior changes. When considering a long-term study, it identified that this 'allowable crack width' can be released, when the concrete cover increases. Further, considering the crack width limit from the aesthetic aspect, the authors suggest categorizing structures or parts of structures into different prestige levels and deciding on the limiting values accordingly. The authors suggest reconsidering the autogenous healing criteria of water-retaining structures with the new findings. Finally, this study influences and highlights the necessary improvements in the existing crack controlling methods, to effectively control the cracks of structures with large concrete covers.

References

[1] F. Leonhardt, Cracks and crack control in concrete structures, PCI Journal 33(4) (1988) 124-145.
[2] H.M. Makhlouf, F.A. Malhas, The effect of thick concrete cover on the maximum flexural crack width under service load, Structural Journal 93(3) (1996) 257-265.
[3] A. Beeby, Concrete in the Oceans Technical Report No. 1, Cement and Concrete Association, Slough, UK (1978).
[4] J.S.o.C. Engineering, Standard Specifications for Concrete Structures-2007 ''Design'', JSCE Guidelines for Concrete, 2007.
[5] E.C.f. Standardization, Eurocode 2: Design of concrete structures - Part 1–1: General rules and rules for buildings 2004.
[6] I.F.f.S.C. (fib), fib Model Code for concrete structures, Structural Concrete, 2010.
[7] A.C. 318, Building Code Requirements for Structural Concrete:(ACI 318-95); and Commentary (ACI 318R-95), American Concrete Institute, 1995.
[8] L.U. British Standards Institute, 8110: Part 1, Structural use of concrete–code of practice for design and construction, 1985.
[9] G. Markeset, M. Kioumarsi, Need for further development in service life modelling of concrete structures in chloride environment, Procedia engineering 171 (2017) 549-556.
[10] J. Connal, M. Berndt, Sustainable bridges: 300 year design life for Second Gateway Bridge, 7th Austroads Bridge Conference, Auckland, New Zealand, 2009.
[11] S. Vegvesen, Håndbok N400 Bruprosjektering, Oslo: Vegdirektoratet, 2009.
[12] T. Danner, M.R. Geiker, Long-term Influence of Concrete Surface and Crack Orientation on Self-healing and Ingress in Cracks–Field Observations, Nordic Concrete Research 58(1) (2018) 1-16.
[13] M. Basteskår, M. Engen, T. Kanstad, K.T. Fosså, A review of literature and code requirements for the crack width limitations for design of concrete structures in serviceability limit states, Structural Concrete 20(2) (2019) 678-688.
[14] P.G. Debernardi, M. Guiglia, M. Taliano, Effect of secondary cracks for cracking analysis of reinforced concrete tie, ACI Materials Journal 110(2) (2013) 207.
[15] B.B. Broms, L.A. Lutz, Effects of arrangement of reinforcement on crack width and spacing of reinforced concrete members, ACI Journal Proceedings, 1965, pp. 1395-1410.
[16] J. Illston, R. Stevens, Internal Cracking, Concrete (London) 6(7) (1972).
[17] F. Leonhardt, Crack control in concrete structures, IABSE surveys, No, International Association for Bridges and Structural Engineering (1977) 26.
[18] M. Nielsen, Beton 1–del 3, 2. udgave (Concrete Structures 1–part 3,), Department of Civil Engineering, Technical University of Denmark, Lyngby, 2005.
[19] Y. Goto, Cracks formed in concrete around deformed tension bars, Journal Proceedings, 1971, pp. 244-251.
[20] M. de Saint-Venant, Mémoire sur la torsion des prismes: avec des considérations sur leur flexion ainsi que sur l'équilibre intérieur des solides élastiques en général: et des formules pratiques pour le calcul de leur résistance à divers efforts s' exerçant simultanément, Imprimerie nationale1856.
[21] A. Beeby, Crack control provisions in the new eurocode for the design of concrete structures, ACI Special Publication 204 (2001) 57-84.
[22] R. Park, T. Paulay, Reinforced concrete structures, John Wiley & Sons1975.
[23] R. Tan, M.A. Hendriks, T. Kanstad, Evaluation of current crack width calculation methods according to eurocode 2 and fib Model Code 2010, High Tech Concrete: Where Technology and Engineering Meet, Springer2018, pp. 1610-1618.
[24] G.D. Base, J.B. Read, A. Beeby, H. Taylor, An investigation of the crack control characteristics of various types of bar in reinforced concrete beams, Research Report No. 18, Cement and Concrete Association Wexham Springs, Slough England, Cement and Concrete Association, London, 1966.
[25] J.F. Borges, Cracking and deformability of reinforced concrete beams, Laboratório Nacional de Engenharia Civil1965.
[26] G.D. Base, J.B. Read, A. Beeby, H. Taylor, An investigation of the crack control characteristics of various types of bar in reinforced concrete beams, Cement and Concrete Association Wexham Springs, Slough England1966.

[27] A.W. Beeby, The influence of the parameter ϕ/ρ eff on crack widths, Structural Concrete 5(2) (2004) 71-83.
[28] R. Saliger, High grade steel in reinforced concrete, Second Congress of IABSE, IABSE Publications

Berlin-Munich, 1936.
[29] D. Watstein, D. Parsons, Width and spacing of tensile cracks in axially reinforced concrete cylinders, Journal of Research, National Bureau of Standards 31(1) (1943) 1-24.
[30] A. Beeby, R. Scott, Cracking and deformation of axially reinforced members subjected to pure tension, Magazine of concrete research. 57(10) (2005) 611-621.
[31] M. Taliano, Cracking analysis of concrete tie reinforced with two diameter bars accounting for the effect of secondary cracks, Engineering Structures 144 (2017) 107-119.
[32] J.G. Rots, J. Blaauwendraad, Crack models for concrete, discrete or smeared? Fixed, multi-directional or rotating?, Heron 34 (1989).
[33] C. Unanwa, M. Mahan, Statistical analysis of concrete compressive strengths for California highway bridges, Journal of performance of constructed facilities 28(1) (2012) 157-167.
[34] C.N. Naotunna, M.K.S. S.M. Samindi, K.T. Fosså, Comparison of the experimental crack spacing behaviour with the theorotical predictions, The Nordic Concrete Federation, Oslo, Norway, 2019.
[35] G.L. Balazs, Cracking analysis based on slip and bond stresses, Materials Journal 90(4) (1993) 340-348.
[36] R. Tan, K. Eileraas, O. Opkvitne, G. Žirgulis, M.A. Hendriks, M. Geiker, D.E. Brekke, T. Kanstad, Experimental and theoretical investigation of crack width calculation methods for RC ties, Structural Concrete (2018).
[37] P. Gergely, L.A. Lutz, Maximum crack width in reinforced concrete flexural members, Special Publication 20 (1968) 87-117.
[38] A.W. Beeby, An investigation of cracking in slabs spanning one way, Cement and concrete association, 1970.
[39] S.I. Husain, P.M. Ferguson, Flexural crack width at the bars in reinforced concrete beams, The University of Texas at Austin, Center for Highway research, 1968.
[40] K. Tammo, S. Thelandersson, Crack opening near reinforcement bars in concrete structures, Structural Concrete 7(4) (2006) 137-143.
[41] A. Borosnyói, I. Snóbli, Crack width variation within the concrete cover of reinforced concrete members, Építõanyag 62(3) (2010) 70-74.
[42] A. Pérez Caldentey, H. Corres Peiretti, J. Peset Iribarren, A. Giraldo Soto, Cracking of RC members revisited: influence of cover, ϕ/ρs, ef and stirrup spacing–an experimental and theoretical study, Structural Concrete 14(1) (2013) 69-78.
[43] P.G. Debernardi, M. Taliano, An improvement to Eurocode 2 and fib Model Code 2010 methods for calculating crack width in RC structures, Structural Concrete 17(3) (2016) 365-376.
[44] G. Kaklauskas, R. Ramanauskas, R. Jakubovskis, Mean crack spacing modelling for RC tension elements, Engineering Structures 150 (2017) 843-851.
[45] D.H. Olsen, M.P. Nielsen, Ny teori til bestemmelse af revneafstande og revnevidder i betonkonstruktioner, Afdelingen for bærende konstruktioner, Danmarks tekniske højskole1990.
[46] P. Marti, M. Alvarez, W. Kaufmann, V. Sigrist, Tension chord model for structural concrete, Structural Engineering International 8(4) (1998) 287-298.
[47] C.H. McLeod, C. Viljoen, Quantification of crack prediction models in reinforced concrete under flexural loading, Structural Concrete 20(6) (2019) 2096-2108.
[48] A.P. Caldentey, Proposal of new crack width formulas in the Eurocode 2, background, experiments, etc, Crack width calculation methods for large concrete structures. , Nordic Concrete Federation, Oslo, 2017, p. 17.
[49] A. Rimkus, V. Gribniak, Experimental investigation of cracking and deformations of concrete ties reinforced with multiple bars, Construction and Building Materials 148 (2017) 49-61.
[50] C. Rospars, D. Chauvel, CEOS. fr experimental programme and reference specimen tests results, European Journal of Environmental and Civil Engineering 18(7) (2014) 738-753.

[51] P. Bisch, The CEOS. fr research project-Behaviour and assessment of massive structures: cracking and shrinkage, Crack width calculation methods for large concrete structures. Workshop proceedings No.12 Oslo: Nordic Concrete Federation (2017) 11.
[52] G.-Y. Lee, W. Kim, Cracking and tension stiffening behavior of high-strength concrete tension members subjected to axial load, Advances in Structural Engineering 12(2) (2009) 127-137.
[53] J.R. Deluce, Cracking behaviour of steel fibre reinforced concrete containing conventional steel reinforcement, University of Toronto, University of Toronto, 2011.
[54] R. Tan, M.A. Hendriks, M. Geiker, T. Kanstad, Analytical Calculation Model for Predicting Cracking Behavior of Reinforced Concrete Ties, Journal of Structural Engineering 146(2) (2020) 04019206.
[55] A.P. Caldentey, R. García, Cracking of RC structures: Differences between tension and flexure, in: N.C. Federation (Ed.) Design and construction of sustainable concrete structures: causes, calculation and consequences of cracks, Nordic Concrete Federation, Oslo Norway, 2019, p. 3.
[56] R. García, A.P. Caldentey, CRACKING OF RC: Tension vs. Flexure, Technical University of Madrid (UPM), Madrid, 2018.
[57] K. Tammo, S. Thelandersson, Crack behavior near reinforcing bars in concrete structures, ACI Structural Journal 106(3) (2009) 259.
[58] J.-J. Wang, M.-X. Tao, X. Nie, Fracture energy-based model for average crack spacing of reinforced concrete considering size effect and concrete strength variation, Construction and Building Materials 148 (2017) 398-410.
[59] H.J. Oh, Y.K. Cho, S.-M. Kim, Experimental evaluation of crack width movement of continuously reinforced concrete pavement under environmental load, Construction and Building Materials 137 (2017) 85-95.
[60] J.-K. Kim, Y.-D. Park, Shear Strength of Reinforced Higy Strength Concrete Beams without Web Reinforcement, Magazine of Concrete Research 46(166) (1994) 7-16.
[61] C.N. Naotunna, M.K.S. S.M. Samindi, K.T. Fosså, Identification of the Influence of Concrete Cover Thickness and $\varnothing/\rho$ Parameter on Crack Spacing (forthcoming). XV International Conference on Durability of Building Materials and Components, Barcelona, 2020.
[62] C.N. Naotunna, M.K.S. S.M. Samindi, K.T. Fosså, Experimental and theoretical behavior of crack spacing of specimens subjected to axial tension and bending (forthcoming). Structural Concrete 1-18 (2020).
[63] Z.P. Bažant, B.H. Oh, Spacing of cracks in reinforced concrete, Journal of structural Engineering 109(9) (1983) 2066-2085.
[64] J. Radnić, L. Markota, Experimental verification of engineering procedures for calculation of crack width in concrete elements, International journal for engineering modelling 16 (2003) 63-69.
[65] C. Ouyang, S.P. Shah, Fracture energy approach for predicting cracking of reinforced concrete tensile members, Structural Journal 91(1) (1994) 69-78.
[66] S.P. Shah, S.E. Swartz, C. Ouyang, Fracture mechanics of concrete: applications of fracture mechanics to concrete, rock and other quasi-brittle materials, John Wiley & Sons1995.
[67] A.C. Institute, A.C. 224, Control of Cracking in Concrete Structures-ACI 224R-01, American Concrete Institute-ACI2001.
[68] A. Beeby, The prediction and control of flexural cracking in reinforced concrete members, ACI Special Publication 30 (1971) 55-76.
[69] C.N. Naotunna, M.K.S. S.M. Samindi, K.T. Fosså, Comparison of the behavior of crack width-governing parameters with existing models, Proceedings of the International Conference on Sustainable Materials, Systems and Structures (SMSS2019) 2019, pp. 124-131.
[70] N. Dawood, H.J.C.J.o.C.E. Marzouk, Crack width model for thick reinforced concrete plates subjected to in-plane forces, 38(11) (2011) 1262-1273.
[71] N. Dawood, H.J.E.s. Marzouk, Experimental evaluation of the tension stiffening behavior of HSC thick panels, 33(5) (2011) 1687-1697.
[72] M. Code, CEB-FIP Model code for concrete structures, Comité Euro-International du Béton, Paris, France, 1978.

[73] M. CEB-FIP, 90. Design of concrete structures. CEB-FIP-Model-Code 1990, British Standard Institution, London (1993).
[74] S. Norge, NS-EN 1992-1-1: 2004+ NA: 2008, Eurocode 2, 2008.
[75] K. Hornbostel, M. Geiker, Influence of cracking on reinforcement corrosion, Crack width calculation methods for large concrete structures, Nordic Concrete Federation, Oslo, 2017, p. 53.
[76] M. Makita, Y. Mori, K. Katawaki, Marine corrosion behavior of reinforced concrete exposed at tokyo bay, ACI Special Publication 65 (1980) 271-290.
[77] C. Lin, Bond Deterioration Dueto Corrosion of Reinforcing Steel, ACI Special Publication 65 (1980) 255-270.
[78] B. Tremper, The corrosion of reinforcing steel in cracked concrete, ACI Journal Proceedings, 1947, pp. 18 (10): 1137-1144.
[79] R. Francois, G. Arliguie, Reinforced concrete: correlation between cracking and corrosion, Special Publication 126 (1991) 1221-1238.
[80] N.S. Berke, M. Dallaire, M. Hicks, R. Hoopes, Corrosion of steel in cracked concrete, Corrosion 49(11) (1993) 934-943.
[81] K.Z. Kahhaleh, Corrosion performance of epoxy-coated reinforcement, The University of Texas at Austin 1995.
[82] E. Chen, C.G. Berrocal, I. Löfgren, K. Lundgren, Correlation between concrete cracks and corrosion characteristics of steel reinforcement in pre-cracked plain and fibre-reinforced concrete beams, Materials and Structures 53(2) (2020) 1-22.
[83] T. Ohta, Corrosion of reinforcing steel in concrete exposed to sea air, Special Publication 126 (1991) 459-478.
[84] P. Schiessl, Admissible crack width in reinforced concrete structures. Contribution II, 3-17, Preliminary reports Vol. II, Inter-association Colloquium on the Behaviour of in Service of Concrete Structures, 1975.
[85] P. Schießl, Zur Frage der zulässigen Rissbreite und der erforderlichen Betondeckung im Stahlbetonbau unter besonderer Berücksichtigung der Karbonatisierung des Betons, Technische Universität München, 1976.
[86] V. Carević, I. Ignjatović, Influence of loading cracks on the carbonation resistance of RC elements, Construction and Building Materials 227 (2019) 116583.
[87] P. Schießl, M. Raupach, Laboratory studies and calculations on the influence of crack width on chloride-induced corrosion of steel in concrete, Materials Journal 94(1) (1997) 56-61.
[88] R. Swamy, Durability of rebars in concrete., The GM Idorn International Symposium on Durability of Concrete, sponsored by Committee 201 on Durability, was held at the 1990 Annual ACI Convention in Toronto, Ontario, Canada., 1992.
[89] S. Misra, T. Uomoto, Reinforcement corrosion under simultaneous diverse exposure conditions, ACI Special Publication 126 (1991) 423-442.
[90] O. Vennesland, O. Gjoro, Effect of cracks in submerged concrete sea structures on steel corrosion, Materials Performance 20(8) (1981) 49-51.
[91] K.O.T. Miyagawa, Chloride corrosion of reinforcing steel in cracked concrete, Special Publication 65 (1980) 237-254.
[92] W. Li, W. Liu, S. Wang, The effect of crack width on chloride-induced corrosion of steel in concrete, Advances in Materials Science and Engineering 2017 (2017).
[93] J.T. Houston, E. Atimtay, P.M. Ferguson, Corrosion of reinforcing steel embedded in structural concrete, Center for Highway Research, University of Texas at Austin1972.
[94] E.F. O'Neil, Study of reinforced concrete beams exposed to marine environment, ACI Special Publication 65 (1980) 113-132.
[95] P. Marcus, Corrosion mechanisms in theory and practice, CRC press2011.
[96] D. Campbell-Allen, The reduction of cracking in concrete, School of Civil Engineering, The university of Sydney, Sydney, 1979.
[97] E.-I.C.f. Concrete, R. Favre, CEB design manual on cracking and deformations, École Polytechnique Fédérale de Lausanne1985.

[98] J. Johnston, Mechanism of Vision—A Review, Sensory Evaluation of Appearance of Materials, ASTM International1973.
[99] J.D. Padilla, F. Robles, Human response to cracking in concrete slabs, ACI Special Publication 30 (1971) 43-54.
[100] M. Choinska, H. Al-Khazraji, N. Benkemoun, A. Khelidj, Influence of tensile cracking and of aggregate size on concrete permeability, MATEC Web of Conferences, EDP Sciences, 2018, p. 02001.
[101] C. Desmettre, J.-P. Charron, Novel water permeability device for reinforced concrete under load, Materials and structures 44(9) (2011) 1713.
[102] T. Van Mullem, E. Gruyaert, B. Debbaut, R. Caspeele, N. De Belie, Novel active crack width control technique to reduce the variation on water permeability results for self-healing concrete, Construction and Building Materials 203 (2019) 541-551.
[103] C.H. McLeod, Investigation into cracking in reinforced concrete water-retaining structures, Stellenbosch University, 2013.
[104] C. Edvardsen, Water permeability and autogenous healing of cracks in concrete, Materials Journal 96(4) (1999) 448-454.
[105] B.S. Institution, British Standard Code of Practice for Design of Concrete Structures for Retaining Aqueous Liquids: British Standard BS 8007, British Standards Institution, 1987.
[106] C.E. Reynolds, J.C. Steedman, A.J. Threlfall, Reinforced Concrete Designer's Handbook, CRC Press2007.
[107] E.C.f. Standardization, PN–EN 1992–3. Design of Concrete Structures—Part 3: Liquid Retaining and Containment Structures, Eurocode 2, 2006.
[108] R. Wang, J. Yu, S. Gu, P. He, X. Han, Q. Liu, Investigation of self-healing capability on surface and internal cracks of cement mortar with ion chelator, Construction and Building Materials 236 (2020) 117598.
[109] T. Qureshi, A. Kanellopoulos, A. Al-Tabbaa, Autogenous self-healing of cement with expansive minerals-II: Impact of age and the role of optimised expansive minerals in healing performance, Construction and Building Materials 194 (2019) 266-275.
[110] T.L. Traxler, Architectural waterproofing membrane and termite barrier, Google Patents, 2002.
[111] A. Scuero, Method and device for the waterproofing of joints and cracks in hydraulic works, concrete and masonry structures, Google Patents, 2003.