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Resistance of cracked concrete to chloride attack

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ABSTRACT

Generally, it is assumed that cracks influence the durability of concrete negatively by accelerating the penetration of aggressive elements. One of those elements are chlorides, present in marine environments. In this research, the effect of the crack width on chloride penetration is investigated by means of non-steady state migration tests based on NT Build 492. Therefore, ordinary Portland cement-, high sulphate resistant- and slag concrete are used. The cracks are formed by means of steel plates with widths of 0.1, 0.2 and 0.3 mm. After the migration test, migration coefficients and penetration depths are obtained. It seems that a crack width of 0.1 mm already has an influence on chloride penetration. Nevertheless, the biggest changes are measured for a crack width of 0.3 mm. The migration coefficient at the 0.3 mm crack tip increases 47 % compared to the migration coefficient at the exposed surface, regardless the concrete type.

Keywords. Concrete, cracks, chlorides, migration, chloride penetration depth

INTRODUCTION

A lot of damage is reported for constructions in marine environments (e.g. bridge pillars, piers, wharfs, foundations, …). Marine environments are very aggressive, since a large fraction of sea water consists of chlorides. A commonly used material for such structures is reinforced concrete. However, chlorides affect the durability of concrete by initiating corrosion of the reinforcement steel. It is important to notice that corrosion will only be initiated by the free chlorides and not by the fraction that is chemically bound to the cement hydrates or physically adsorbed at the pore walls.

It is assumed that cracks negatively influence concrete's durability by increasing the chloride penetration. So, chlorides will be able to initiate corrosion faster as well. However, only limited literature is found concerning chloride penetration in cracked concrete.

To form cracks, destructive methods as well as non-destructive methods are used. Cracks obtained by a destructive test method (e.g. Brazilian splitting test, 3-point bending test or 4-point bending test) simulate actual cracks in concrete structures, since the crack pattern, tortuosity and connectivity, etc. is similar. In contrast, cracks obtained by the non-destructive method are similar to notches. According to (Song, 2012), the destructive methods have some disadvantages: repeatability, accuracy and reliability. It is very difficult to obtain the same cracks with a destructive method, besides, it is hard to obtain the required crack width or length due to recovery of the crack during unloading. Although the non-destructive method cannot provide real cracks, it is an effective way to study and analyze the effects of cracks on transport properties of concrete. (Audenaert et al., 2007) and (Song, 2012) established a notch method to produce an artificial crack in concrete by means of thin steel plates inside the specimen. Varied crack parameters such as crack width, depth and length are obtained by choosing varied sizes of sheets to insert into the concrete.

Regardless of the cracking methods, a lot of other parameters influence the chloride penetration in cracked concrete. The most important parameters are the crack width and the crack depth. Besides, also the concrete mix, the aggressiveness of the environment, etc. play an important role.

Conform to the standard (NBN EN 1992-1-1, 2010), the allowable crack width in marine environments is in the range of 0.3 to 0.4 mm. However, (Win et al., 2004) found that the chloride penetration depth at the crack tip of a 0.1 mm wide crack is already higher than the chloride penetration from the surface. According to (Ismail et al., 2008) crack widths of 0.2 mm and wider are no limiting factor for the chloride diffusion perpendicular to the crack wall. Besides, (Djerbi et al., 2008) and (Audenaert et al., 2009) found a bilinear relation between chloride migration and the crack width: For crack widths between 0 and 0.1mm, the penetration depth increased with increasing crack width and for crack widths between 0.1mm and 0.2mm, this increase is less clear and the migration is rather stable.

In this research, the influence of crack widths on the resistance of concrete against chlorides is investigated. To do so, rapid chloride migration tests are performed and cracks are made in a non-destructive way.

MATERIALS AND METHODS

Concrete mixtures

Four different concrete mixtures were prepared (Table 1): two Portland cement mixtures and two Blast-Furnace Slag (BFS) mixtures. The Ordinary Portland Cement mixture (OPC) was seen as the reference mixture. The other Portland cement mixture contained High Sulphate Resistant cement (HSR). The BFS concrete mixtures contained high amounts of slag. The cement replacement levels amounted to 50 % (S50) and 70 % (S70), respectively. The total binder content (cement + slag) was maintained at 350 kg/m³ and the water-to-binder factor (W/B) at 0.45. This is in accordance with (EN 206-1, 2000), when the concrete is applied in an ES3 environment. Superplasticizer (SP) was added to the mixture to obtain a slump between 160 mm – 210 mm, namely consistency class S4 (EN 12350-2, 2009).

Since HSR cement was used because of its low C_3A-content, the C_3A-content was calculated by using the Bogue equations. According to (EN 197-1, 2000), the C_3A-content for HSR cement is limited to 3 %. In current research, the C_3A-content for HSR amounts to 2.50 % and corresponds to the standard. For OPC the C_3A-content is 7.92 %.

Table 1. Concrete mixtures

	OPC	HSR	S50	S70
Sand 0/4 (kg/m³)	781	781	781	781
Aggregate 2/8 (kg/m³)	619	619	619	619
Aggregate 8/16 (kg/m³)	480	480	480	480
CEM I 52.5 N (kg/m³)	350	-	175	105
CEM I 52.5 N HSR (kg/m³)	-	350	-	-
Slag (kg/m³)	-	-	175	245
Water (kg/m³)	157.5	157.5	157.5	157.5
W/B (-)	0.45	0.45	0.45	0.45
S/B (%)	0	0	50	70
SP (ml/kg B)	4	4	4.5	4.5
Strength class	C45/55	C45/55	C35/45	C30/37

Curing and sample preparation

For the uncracked concrete, cubes with a 150 mm side were casted and cured at 20 °C and a relative humidity higher than 95 %. After demoulding, the specimens were stored again under the same conditions until the age of 14 days. At the age of 14 days, a core with a diameter of 100 mm was drilled out of each cube. This core was cut in three specimens with a thickness of 50 mm. These specimens were stored again until the age of testing. The cracked concrete samples were made in another way, described in the following part.

Crack formation

A non-destructive method was used to generate cracks in concrete conform the method described by (Song, 2012). The thin plates used in this research had a preset width of 0.1 mm, 0.2 mm and 0.3 mm. The crack depth was kept constant at 15 mm for crack widths 0.1 mm and 0.2 mm, and at 20 mm for crack widths of 0.3 mm. The crack length was 60 mm.

The steel plates are placed on an iron rod which is fixed by using a magnetic base. The plates are positioned into the center of the molds at the intended depth. The molds are cylindrical with a height of 50 mm and a diameter of 100 mm. The concrete is poured into the molds and vibrated while the plates stay in the concrete. After a couple of hours, the steel plates are removed carefully before the concrete totally hardens. After three weeks storage, the outermost layer is cut off to obtain a flat measuring surface. The set-up to prepare the artificial cracks and the resulting concrete samples are shown in Figure 1 and Figure 2.

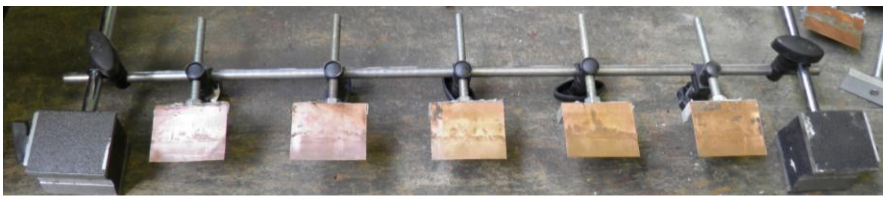

Figure 1. Preparation of the steel plates.

Figure 2. Positioning of the steel plates (left), Casting of the samples (middle), Prepared concrete sample (right).

Rapid chloride migration test

The resistance to chloride penetration of cracked and uncracked concrete was evaluated by using the rapid chloride migration test as described in (NT Build 492, 1999). This method enables the calculation of a non-steady state chloride migration coefficient.

At least three samples are tested per concrete type and crack width. The penetration depth after spraying silver nitrate on the split specimen was measured from the centre to both edges at intervals of 10 mm with an accuracy of 1 mm for the uncracked concrete. For the cracked concrete, the penetration depth was measured in a zone of 20 mm around the crack tip, at intervals of 5 mm, as schematically shown in Figure 3. The penetration depth in this zone was measured starting from the crack tip.

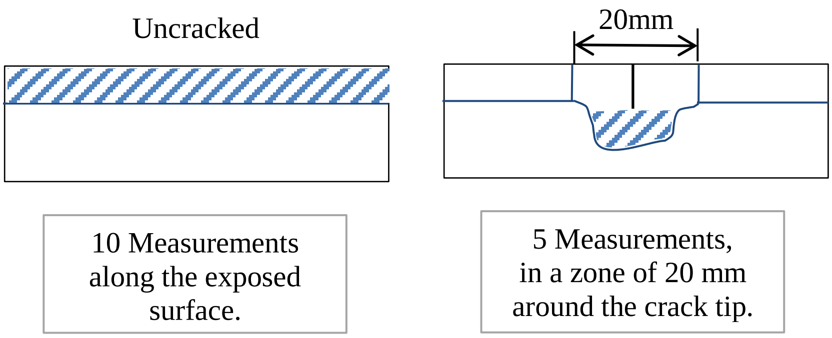

Figure 3. Colour change boundary measurement:
Uncracked: from the surface (left) - Cracked: around the crack tip (right)

The obtained chloride ingress boundary provides the necessary input parameters to calculate the non-steady state migration coefficient by using Equation 1 (NT Build 492, 1999):

$$D_{RCM} = \frac{0.0239\,(273+T)\,L}{(U-2)\,t}\left(x_d - 0.0238\sqrt{\frac{(273+T)\,L x_d}{U-2}}\right) \qquad (1)$$

where D_{RCM}, U, T, L, x_d and t represent the non-steady state migration coefficient (10^{-12} m²/s), the absolute value of the applied voltage (V), the average value of the initial and final temperatures in the anolyte solution (°C), the thickness of the specimen (mm), the penetration depth (mm) and the test duration (h). According to (Maes et al., 2012) this simplified formula can also be used to calculate the migration coefficient in slag concrete.

Secondly, penetration depths were compared. In order to be able to compare the chloride penetration depths on itself, the rapid chloride migration test was executed with constant set-up parameters. However, according to the method described in (NT Build 492, 1999), the applied voltage during the test differs for every measurement. The applied voltage should be based on the initial current through the specimen at a voltage of 30 V. Because of this, it is not possible to compare the colour change boundaries. Therefore, in the second part of this research, some tests were performed at a constant voltage of 30 V and a constant duration of 8 hours. Afterwards the chloride penetration is measured by means of the colorimetric method.

RESULTS

Chloride migration coefficient

Figure 4 shows the non-steady state migration coefficients and the standard errors for the four different concrete mixtures tested at 28 days.

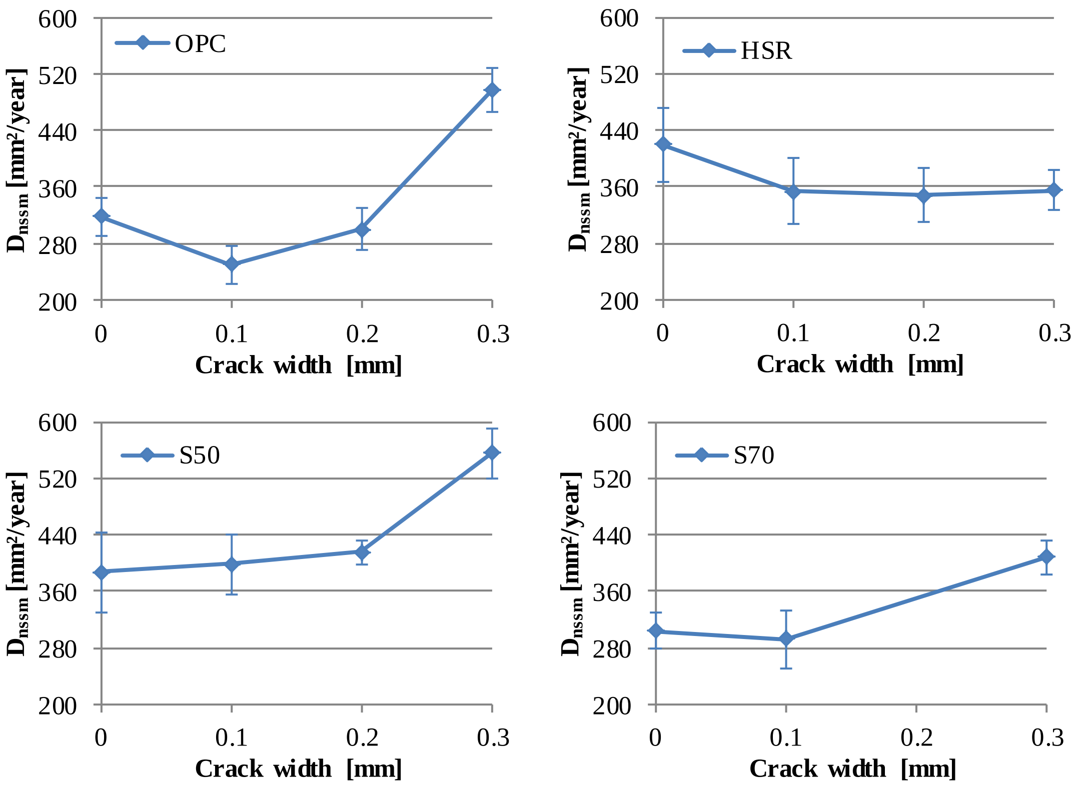

Figure 4. Non-steady state migration coefficients in function of the crack width for OPC, HSR, S50 and S70 concrete at the age of 28 days.

According to the graphs shown in Figure 4, the migration coefficient is influenced by increasing crack width for all concrete types except the HSR concrete. For HSR concrete, the migration coefficients at the crack tips, regardless the crack width, seem to be slightly lower than the migration coefficient for uncracked concrete. However, according to a one-way ANOVA with a Student-Newman-Keuls test (level of significance = 0.05) these coefficients are not significantly different from each other. In contrast, the migration coefficient, measured at the crack tip for the other concrete types increases when the crack width is equal to 0.3 mm. By means of a one-way ANOVA with a Student-Newman-Keuls test (level of significance = 0.05) it is shown that the migration coefficient at the crack tip of a 0.3 mm crack is significantly higher than the migration coefficient for uncracked concrete measured from the exposed surface. Crack widths of 0.1 and 0.2 mm do not influence the migration coefficient significantly.

Figure 5 shows the non-steady state migration coefficients and the standard errors for the four different concrete mixtures tested at 56 days. It should be remarked that for S50 concrete with cracks widths of 0.2 and 0.3 mm, only one specimen was tested. For S70, no results are available.

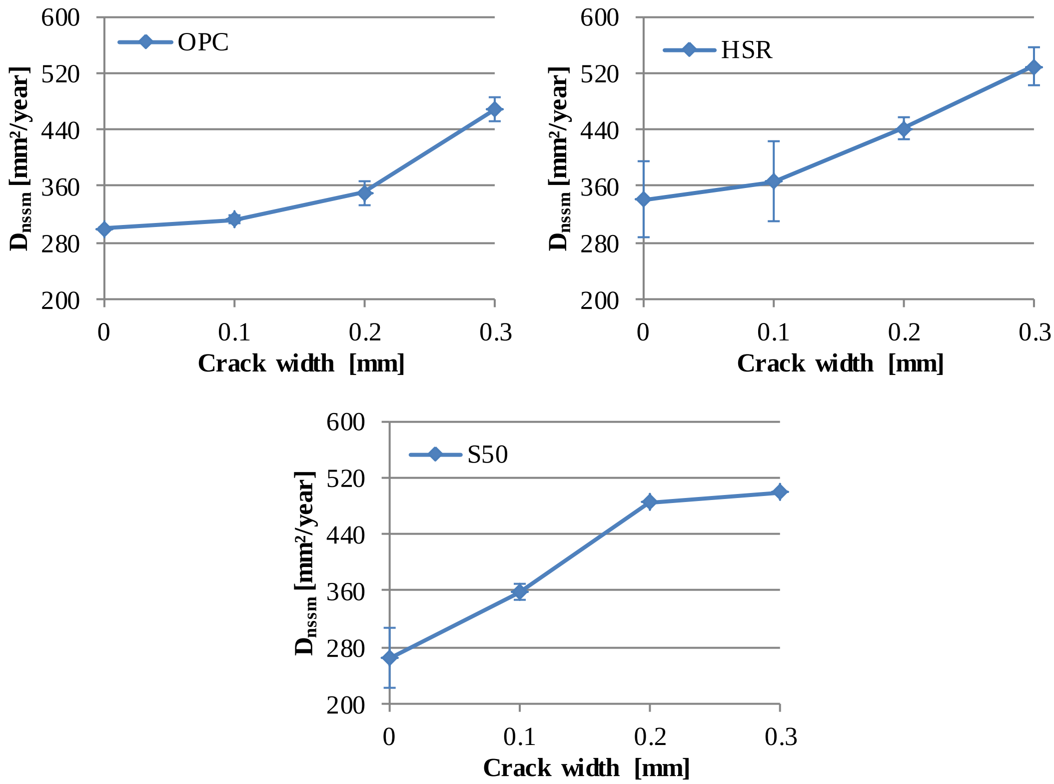

Figure 5. Non-steady state migration coefficients in function of the crack width for OPC, HSR and S50 concrete at the age of 56 days.

The graphs in Figure 5 show that the chloride migration coefficient of concrete is influenced by increasing the crack width for all concrete types at the age of 56 days. The migration coefficient, measured at the crack tip increases when the crack width increases from 0 to 0.3 mm. At 28 days still a small decrease was measured between 0 mm and 0.1 mm for OPC concrete. However, this was not significant. To compare the results per concrete type, a one-way ANOVA with a Student-Newman-Keuls test (level of significance = 0.05) is performed (except for S50, since only one specimen was tested). In accordance with this test, it is clear that the migration coefficient at the crack tip from a 0.3 mm crack in OPC and HSR concrete is significantly higher than the migration coefficient at the tip of a 0.1 and 0.2 mm crack and than for uncracked concrete, measured from the exposed surface. Also the migration coefficient at the crack tip from a 0.2 mm crack in OPC concrete is significantly higher than the migration coefficient for uncracked concrete. Besides, crack widths of 0.1 mm do not influence the migration coefficient significantly.

Relative differences between the migration coefficients of cracked concrete, measured form the crack tip, and the migration coefficient at the exposed surface of uncracked concrete are calculated. These results are tabulated in Table 3. The differences are calculated for OPC and HSR concrete tested at 28 days and 56 days and for S50 and S70 tested at 28 days.

Table 3. Relative difference between the migration coefficient at the tip of a crack and the migration coefficient at the surface for uncracked concrete ($D_{nssm, s}$).

Concrete type	% increase compared to $D_{nssm, s}$		
	0.1 mm	0.2 mm	0.3 mm
		28 days	
OPC	-21	-6	47
HSR	-16	-17	-15
S50	3	7	43
S70	-4	-	35
		56 days	
OPC	4	17	56
HSR	7	29	55

Except for the results of HSR concrete at 28 days, the migration coefficient increase at the crack tip of a 20 mm deep and 0.3 mm wide crack compared to the migration coefficient at the surface of uncracked concrete is in the range of 35 % to 56 %, regardless the concrete type and age. The average increase amounts 47 %.

Based on these results a relation can be found between the change in migration coefficient at the crack tip and the crack width, as shown in Figure 6. The values of HSR concrete at 28 days are not taken into account. This relation is independent of the concrete type.

The relation can be described as follows:

$$D_{nssm,cr} / D_{nssm,s} = 9.3\, w^2 - 1.2\, w + 1 \qquad (0.1 \text{ mm} \leq w \leq 0.3 \text{ mm})$$

where $D_{nssm,cr}$ is the migration coefficient measured in the zone around the crack tip, $D_{nssm,s}$ is the migration coefficient measured at the exposed surface for uncracked concrete and w is the crack width [mm]. The obtained relation is only valid for cracks in the range of 0.1 mm to 0.3 mm. It is assumed that wider cracks will not influence the migration coefficient at the crack tip in the same way. As can be found in literature, from a certain width, the influence of the crack width on the chloride migration will become stable.

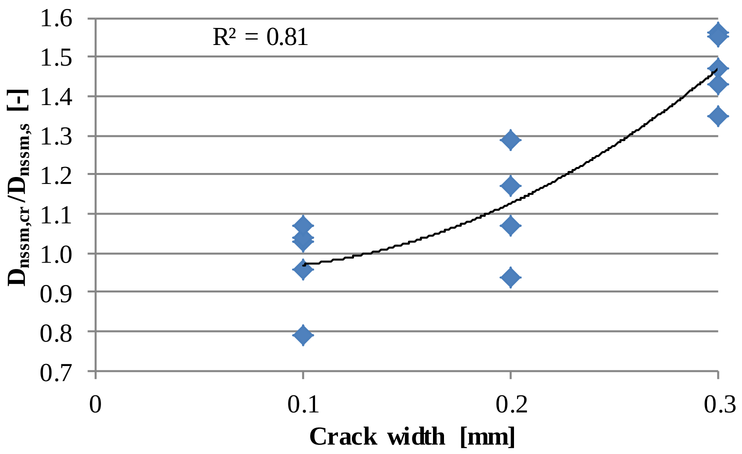

Figure 6. Relation between the non-steady state migration coefficient (D_{nssm}) ratio and the crack width.

So, these findings are not completely in accordance with the findings of (Djerbi et al., 2008) and (Audenaert et al., 2009) who described a bilinear relation between chloride migration and crack width. According to them, the migration coefficient increases significantly until a crack width of 0.1 mm and becomes stable for cracks wider than 0.1 mm. It should be noted that they did not measure from the crack tip but always from the exposed surface. However, when the migration coefficients in this research are calculated from the surface, it seems that they stay increasing for cracks wider than 0.1 mm.

Chloride penetration depth

Next to the calculation of the non-steady state migration coefficients conform to (NT Build 492, 1999), penetration depths are measured for OPC concrete at 28 days after performing the rapid chloride migration test with constant set-up parameters. Firstly, the chloride penetration depth is measured from the surface for uncracked concrete. Secondly, the chloride penetration depth is measured from the crack tip as well as perpendicular to the crack (measured from the crack tip). Table 4 gives the results of these measurements.

Table 4. Chloride penetration depth measured at the crack tip and perpendicular to the crack.

Crack width	Chloride penetration	
	Depth from the crack tip [mm]	Perpendicular to the crack [mm]
Uncracked	12.07 ± 0.44	
0.1 mm	14.02 ± 0.36	8.66 ± 0.71
0.3 mm	13.85 ± 0.43	9.32 ± 0.38

To compare the obtained results, a one-way ANOVA with a Student-Newman-Keuls test (level of significance = 0.05) is performed. In accordance with this test, chloride penetration from the crack tip of the 0.1 and 0.3 mm wide cracks is significantly higher than the chloride penetration from the surface. Besides, no significant difference is found between chloride penetration at the crack tip of the 0.1 mm crack and the 0.3 mm crack. On the other hand, the chloride penetration perpendicular to the crack (measured at the crack tip) is significantly lower than the chloride penetration from the surface for 0.1 mm wide cracks as well as for 0.3 mm wide cracks. No significant difference is found between the 0.1 and 0.3 mm crack.

These results partially confirm the former results. It is clear that the chloride penetration at the crack tip is higher than from the exposed surface. However, no difference was measured between the crack widths. These findings are comparable to the findings of (Djerbi et al., 2008) and (Audenaert et al., 2009), namely an increasing chloride penetration when crack width increases until 0.1 mm and a stable chloride penetration when the crack width increases from 0.1 mm to 0.3 mm. These results also confirm the findings of (Win et al., 2004) who found that the penetration depth at the crack tip is higher than from the exposed surface. Besides Win et al. say that the chloride penetration remains constant with increasing crack width of 0.1 mm until 0.5 mm.

Concerning the penetration perpendicular to the crack, (Win et al., 2004) and (Audenaert et al., 2009) say that the chloride penetration is higher around the crack than from the surface. However, this phenomenon is not clear from these results since the opposite is found. (Ismail et al., 2008) measured the chloride concentrations perpendicular to the crack and they did not measure higher values than from the surface.

Probably the higher chloride penetration at the crack tip is due to the combined effect of horizontal and vertical chloride penetration.

Besides, in marine environments, autogenous healing of cracks will occur since cracked concrete is able to heal cracks by means of ongoing hydration (Van Tittelboom et al., 2012). The healing capacity by ongoing hydration is even improved when slag is used as cement replacement. This phenomenon will influence the chloride penetration in cracked concrete.

CONCLUSIONS

- Chloride migration coefficients at the tip of 0.1 mm and 0.2 mm wide cracks do not increase compared to the chloride migration from the exposed surface. Besides, crack widths of 0.3 mm result in significant higher chloride migration coefficients.

- The average migration coefficient increase at the tip of a 0.3 mm wide crack, compared to the migration coefficient at the surface of uncracked concrete, amounts to 47%.

- The relation between the migration coefficient (D_{nssm}) ratio and the crack width (w) can be described as follows:

  $$D_{nssm,cr} / D_{nssm,s} = 9.3\, w^2 - 1.2\, w + 1 \qquad (0.1 \text{ mm} \leq w \leq 0.3 \text{ mm})$$

- The penetration depths at the crack tips of 0.1 mm and 0.3 mm wide cracks, measured after rapid chloride migration tests with constant set-up parameters for every measurement, are significantly higher than those measured from the surface of uncracked concrete. The penetration depths at the crack tips of 0.1 mm and 0.3 mm wide cracks do not differ.

In general, chloride penetration at crack tips, for crack widths between 0.1 mm and 0.3 mm, is underestimated if the chloride penetration is assumed to be equal to the chloride penetration at the surface. It should also be noted that extra research is needed focusing on the influence of autogenous healing and its effect on chloride penetration in cracked concrete. So, a clear relation between these parameters is necessary in order to predict the influence of cracked concrete on the service life of concrete constructions as exactly as possible.

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