

# CHLORIDE PENETRATION AND CARBONATION IN SELF-COMPACTING CONCRETE

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## Abstract

In this research program, both the steady-state and the non-steady-state migration test are used to determine the chloride diffusion coefficient  $D$  ( $\text{m}^2/\text{s}$ ) of 8 different self-compacting concrete mixes and 1 reference, traditionally vibrated, concrete mix. Simultaneously, the carbonation behaviour of those mixes was investigated. Here fore, the carbonation depth was measured at regular points in time according to NT BUILD 357 (1989) and a carbonation constant  $A$  ( $\text{mm}/\sqrt{\text{year}}$ ) was deduced.

Concerning the chloride diffusion coefficient, test results revealed that the determination of the steady-state migration coefficient according to NT BUILD 355 (1997) is far from easy and question marks could be placed beside the corresponding diffusion coefficient but an explanation for the observed anomalies could not be found yet. The non-steady-state diffusion coefficient according to NT BUILD 492 (1999) was used in order to rank the different concrete mixtures.

The carbonation constant could best be measured using an inflated  $\text{CO}_2$ -concentration, resulting in a more linear behaviour of the carbonation depth in function of time. Besides, using this carbonation constant, results reveal that the concrete mixtures could be ranked in the same way as they were by the non-steady-state chloride diffusion coefficient.

**Keywords:** chloride penetration, carbonation, self-compacting concrete.

## 1. INTRODUCTION

The service life of reinforced concrete structures exposed to chlorides (seawater, deicing salts, ...) strongly depends on the quality of the concrete cover determining the chloride penetration velocity. The means by which chloride ions can penetrate concrete are capillary absorption, hydrostatic pressure and diffusion. Of these three transport mechanisms that can bring chlorides into the concrete to the level of the rebar, the principal method is that of

diffusion. It is rare for a significant hydraulic head to be exerted on the structure, and the effect of absorption is typically limited to a shallow cover region (20-40 mm). In the bulk of the concrete, the pores remain saturated and chloride ion movement is controlled by concentration gradients.

Since diffusion is a very slow process, some test methods using an electrical field to increase the diffusion rate were developed. In this research program, both the steady-state migration test (NT BUILD 355) and the non-steady-state migration test (NT BUILD 492) are used to determine the chloride diffusion coefficient  $D$  of 8 different self-compacting concrete mixes and 1 reference, traditionally vibrated concrete mix.

The corrosion of steel reinforcement, caused by moisture penetrating to the depth of the steel where it reacts in the presence of oxygen is aggravated by carbonation. This is due to the action of acidic environmental conditions resulting in a lowering of the alkalinity of the concrete from an initial pH of around 13.5 to around 8.5 over some years. Since carbonation of concrete is dominated by a diffusion regime, it can be considered as a Fickian problem. In that way, the carbonation constant  $A$  ( $\text{mm}/\sqrt{\text{year}}$ ), which depends on the diffusion resistance of the material, of the different mixes was also determined.

It has to be mentioned that the executed experiments are done in the framework of a Research Programme of the Research Foundation – Flanders, “*Transport properties of potentially aggressive media in SCC and the relation with durability*”, in cooperation with the Magnel Laboratory (Ghent University) and the Royal Military Academy (Brussels).

## 2. MIX COMPOSITIONS

In the experimental part of the research project, 9 different mix compositions are tested. They consist of 1 traditionally vibrated concrete mixture (TC1) and 8 self-compacting concrete mixtures (SCC1 = reference SCC-mixture). They were all fabricated at the Magnel Laboratory. An overview of the different mixtures is given in tables 1 and 2. Besides the composition, also the ‘fresh state’ properties and the compressive strength at 28 days of 150 mm cubes are represented. All self-compacting mixtures showed a self-levelling behaviour in the U-box test. Note that W/C, C/P, W/P are respectively the water/cement, the cement/powder, and the water/powder ratio *by mass*.

Table 1: Mix composition of TC1

<b>TC1</b>		
River Gravel 4/14	[kg/m <sup>3</sup> ]	1225
River Sand 0/5	[kg/m <sup>3</sup> ]	640
CEM I 42.5 R	[kg/m <sup>3</sup> ]	360
Water	[l/m <sup>3</sup> ]	165
W/C	[-]	0.46
C/P	[-]	1
W/P	[-]	0.46
Slump	[mm]	14
Flow	[-]	1.89
Air	[%]	1.1
$f_{c,cube}$	[N/mm <sup>2</sup> ]	47.6

Table 2: Mix compositions of the SCC mixtures

		SCC1	SCC3	SCC5	SCC9	SCC14	SCC15	SCC16	SCC17
River Gravel 4/14	[kg/m <sup>3</sup> ]	698	698	698	698	707	683	-	675
Limestone 4/14	[kg/m <sup>3</sup> ]	-	-	-	-	-	-	698	-
Sand 0/5	[kg/m <sup>3</sup> ]	853	853	853	853	865	835	853	825
CEM I 42.5 R	[kg/m <sup>3</sup> ]	360	-	300	360	360	360	360	360
CEM III/A 42.5 LA	[kg/m <sup>3</sup> ]	-	360	-	-	-	-	-	-
Limestone filler	[kg/m <sup>3</sup> ]	240	240	300	-	240	240	240	240
Fly Ash	[kg/m <sup>3</sup> ]	-	-	-	240	-	-	-	-
Water	[l/m <sup>3</sup> ]	165	165	165	165	144	198	165	216
Glenium 51 (35%)	[l/m <sup>3</sup> ]	-	2.75	2.5	2.4	4.0	2.0	3.6	2.0
W/C	[-]	0.46	0.46	0.55	0.46	0.4	0.55	0.46	0.6
C/P	[-]	0.6	0.6	0.5	0.6	0.6	0.6	0.6	0.6
W/P	[-]	0.28	0.28	0.28	0.28	0.24	0.33	0.28	0.36
Slumpflow	[mm]	875	905	785	800	775	920	875	805
V-funnel	[s]	7.9	7.0	8.1	16.6	20.0	3.5	8.0	2.8
Air	[%]	1.7	0.7	2.3	3.5	2.5	1.1	1.1	0.4
f <sub>c,cube</sub>	[N/mm <sup>2</sup> ]	57.1	69.2	49.0	63.5	68.4	46.7	73.3	39.9

### 3. CHLORIDE PENETRATION

#### 3.1 Steady-state conditions: The underlying theory of NT BUILD 355 (1997)

The underlying theory of NT BUILD 355 (1997) is based on the fact that the movement of chloride ions in a solution under an electrical field is governed by the Nernst-Planck equation [1]:

$$-J_{Cl} = D_{Cl} \frac{\partial C_{Cl}(x)}{\partial x} + \frac{z_{Cl}F}{RT} D_{Cl} C_{Cl} \frac{\partial E(x)}{\partial x} + C_{Cl} v_{Cl}(x) \quad (1)$$

where  $J_{Cl}$  is the chloride flux [kg/m·s],  $D_{Cl}$  is the chloride diffusion coefficient [m<sup>2</sup>/s],  $C_{Cl}(x)$  is the chloride concentration as a function of location  $x$  [kg/m<sup>3</sup>],  $z_{Cl}$  is the chloride valence (= -1 equivalent/mol),  $F$  is Faraday's number (= 9.648·10<sup>4</sup> C/equivalent),  $R$  is the universal gas constant (= 8.314 J/mol·K),  $T$  is the absolute temperature [K],  $E(x)$  is the applied electrical potential as a function of location  $x$  [V] and  $v_{Cl}(x)$  is the chloride convection velocity [m/s].

So, equation 1 shows that the chloride flux can be described by three separate phenomena [1]: flux = “pure diffusion” + “electrical migration” + “convection”. As we will see further on, the test setup of NT BUILD 355 implies neglecting “pure diffusion” and “convection” compared to the effect of electrical migration: the applied voltage is sufficiently strong (at least 10-15 V, [1]) and there are no pressure or moisture gradients.

In that way, equation 1 becomes:

$$J_{Cl} = D_{ssm} \frac{z_{Cl}FC_{Cl}}{RT} \frac{\Delta E}{\Delta x} \quad (2)$$

Where  $D_{ssm}$  is the steady-state migration coefficient,  $\Delta E$  is the voltage drop across the concrete specimen and  $\Delta x$  is the thickness of the concrete specimen (see also figure 1).

Equation 2 allows the solution for  $D_{\text{ssm}}$  once the chloride flux  $J_{\text{Cl}}$  is determined (see equation 3). Note that the following assumptions are implicitly made [1]:

- the voltage drop across the concrete specimen is linear ( $\partial E/\partial x \equiv \Delta E/\Delta x$ );
- the chloride concentration in the “upstream cell” is constant;
- the steady state conditions have been reached ( $J_{\text{Cl}} \equiv \text{constant}$ ); and
- the heating of the solution and the concrete specimen is negligible.

The chloride flux  $J_{\text{Cl}}$  can be determined by the slope of the linear increase in chloride concentration in compartment 2 (see also section 3.2),  $\Delta C_{\text{Cl},2}$  [ $\text{kg}/\text{m}^3$ ], with increasing time  $\Delta t$  [s] (as also shown in figure 2):

$$J_{\text{Cl}} = \frac{\Delta C_{\text{Cl},2} V_2}{A_c \Delta t} \quad (3)$$

where  $V_2$  is the volume of compartment 2 [ $\text{m}^3$ ] and  $A_c$  is the concrete disc area [ $\text{m}^2$ ].

### 3.2 Steady-state conditions: test setup according to NT BUILD 355

For each concrete mix, four test specimens ( $\phi = 100$  mm,  $h = 50$  mm) were taken from two cylindrical concrete samples ( $\phi = 100$  mm,  $h = 200$  mm). The test specimens were stored in a climate room at  $20 \pm 2$  °C and  $> 90$  % R.H. until a concrete age of 89 days when they were vacuum saturated in a saturated  $\text{Ca}(\text{OH})_2$ -solution at  $20 \pm 2$  °C and kept in the same  $\text{Ca}(\text{OH})_2$ -solution until testing. Testing started at 90 days by mounting the specimen in a two-compartment cell (figure 1) and filling both compartments with the  $\text{Ca}(\text{OH})_2$ -solution, leaving the cell undisturbed for 24 hours in order to check for possible leakage (note that no leakage was observed in any of the experiments). After 24 hours, both compartments were emptied. The negative side of the cell (compartment 1) was filled with 250 ml of a 5 % NaCl solution, where the positive side of the cell (compartment 2) was filled with 240 ml of a 0.3 N NaOH solution. Precautions by means of sealant plugs were taken to prevent evaporation during the test period. Each day, 150 ml of compartment 1 was replaced by the same volume of the initial 5 % NaCl solution to maintain its concentration approximately constant (within 95 % of the initial value) during the entire experiment.

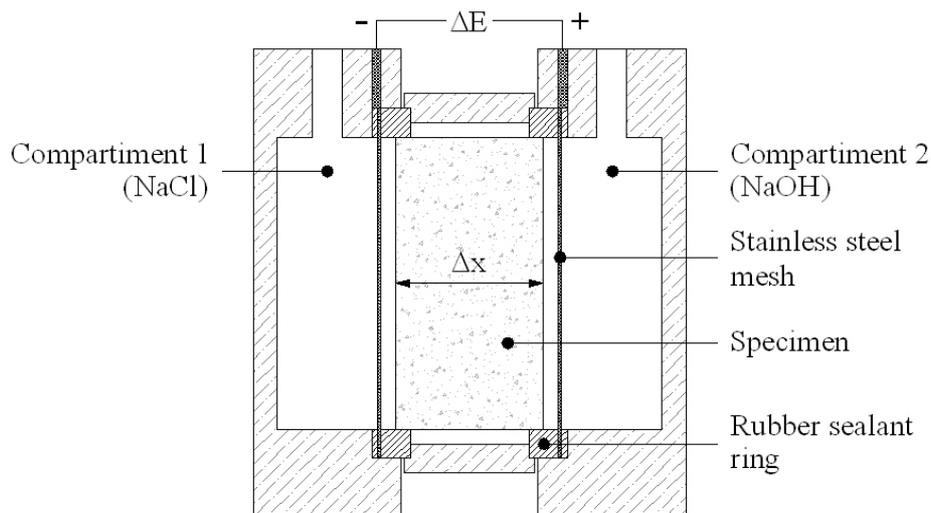


Figure 1: Schematic view of test cell and mounted specimen

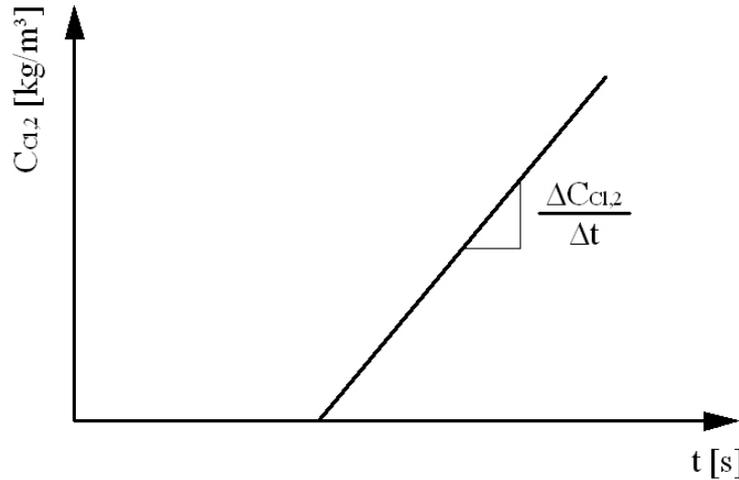


Figure 2: Schematic variation of the chloride concentration with time in compartment 2

The stainless steel mesh electrodes were connected with a power supply in such a way that the voltage across the concrete specimen  $\Delta E$  was fixed at 10 V. A first, exploratory, test revealed that the use of  $\Delta E = 18$  V gave rise to corrosion of the stainless steel meshes.

The chloride concentration in compartment 2,  $C_{Cl,2}$  [kg/m<sup>3</sup>], was determined by means of a Cl<sup>-</sup> ion-selective electrode instead of the more expensive and time-consuming chemical analyses. This Cl<sup>-</sup> ion-selective electrode was calibrated every other day by measuring 7 standard solutions with known chloride concentrations (10<sup>-5</sup>, 10<sup>-4</sup>, 10<sup>-3</sup>, 10<sup>-2</sup>, 10<sup>-1</sup>, 0.5 and 1 mol Cl<sup>-</sup>/l). It was observed that 5 consecutive readings, with a mutual divergence of 0.1 mV at most, lead to a stable measurement. In this way, further calculations could be excluded from coincidental unstable readings from the Cl<sup>-</sup> ion-selective electrode.

### 3.3 Steady-state conditions: test results

An overview of the test results is given in table 3.

Table 3: Overview of the measured steady-state migration coefficients  $D_{ssm}$  [10<sup>-12</sup> m<sup>2</sup>/s]

TC1	r <sup>2</sup>	D <sub>ssm</sub>	SCC1	r <sup>2</sup>	D <sub>ssm</sub>	SCC15	r <sup>2</sup>	D <sub>ssm</sub>
1	0.80	0.78	1	0.75	0.53	1	0.25	0.0029
2	0.87	3.47	2	0.88	0.27	2	-	-
3	0.74	0.83	3	0.93	0.33	3	0.19	0.0019
4	0.87	1.79	4	0.35	0.0007	4	0.20	0.0020

Some remarks should be made immediately:

- For TC1 and SCC1, 3 weeks were needed to have sufficient data for further analysis. For SCC15, even after 3 months only a negligible amount of chloride ions were found in compartment 2 (although it was expected that the  $D_{ssm}$  must be higher for SCC15 compared with SCC1, due to its higher W/C ratio).
- It was impossible to determine the steady-state migration coefficient for SCC15-2 due to a complete corrosion of the stainless steel mesh almost immediately after the “breakthrough” of the chloride ions in the 0.3 N NaOH compartment (before a linear increase in chloride concentration in compartment 2 could be observed).

- The measured steady-state migration coefficients given in *italic* (SCC1-4 and SCC15-1/3/4) are of non importance and are excluded in further discussions.
- Besides, according to NT BUILD 355, the linear correlation coefficient  $r^2$  used to calculate  $\Delta C_{Cl,2}/\Delta t$  should be at least 0.9. This correlation could only be found for SCC1-3. Note that, consistent with the low steady-state migration coefficients, the correlation coefficients for SCC1-4 and SCC15-1/3/4 are also very small.

Seen the difficulties which arose during those experiments (corrosion of the stainless steel mesh at relative low power supply, test duration, ...) and their interpretation (significance of the finally obtained values for  $D_{ssm}$ ?), questions arise about the suitability of this test method. An explanation for the observed anomalies could not be found yet.

Besides, it is known from [3] that the determination of steady-state chloride diffusion coefficients can only rank concrete qualities or recipes. It can not be used directly in service life calculations for structures where chlorides in practice are transported in a non-steady-state manner. Also, [3] stated that a far more simple measurement of electrical resistance will rank the same concretes in the same way.

### 3.4 Non-steady-state conditions as an alternative?

In the framework of the above mentioned Research Programme, non-steady-state migration experiments were also executed at the Magnel Laboratory. The experiments were executed according to NT BUILD 492 (1999), also known as the CTH test, proposed by [2].

The test proceeds as usual for an electrical migration test, except that the chloride concentration of the “downstream solution” is not monitored. Instead, the samples are removed after 24 h and split axially. The chloride penetration depth is then determined in one half of the freshly split specimen (according to NT BUILD 492, appendix 1) using a 0.1 N  $AgNO_3$  solution as a colorimetric indicator: the chloride ions bind with the silver ions to produce the white  $AgCl$ -precipitation. The chloride penetration depth is used to determine the chloride diffusion coefficient  $D_{nssm}$ .

The test specimens ( $\phi = 100$  mm,  $h = 50$  mm) were drilled out 150 mm concrete cubes at 21 days. Until that moment, cubes were stored in a climate room at  $20 \pm 2$  °C and  $> 90$  % R.H. and after drilling, the cores were placed back in the climate room. At a concrete age of 27 and 89 days, the cores were vacuum saturated in a saturated  $Ca(OH)_2$ -solution and kept in the same  $Ca(OH)_2$ -solution for another  $18 \pm 2$  h. Afterwards, the test specimens were setup as shown in figure 3 and a power supply of 25 ~ 40 V was applied. Note that the chloride concentration in the “upstream solution” is twice as high as in the above mentioned test setup according to NT BUILD 355.

The non-steady-state migration coefficient  $D_{nssm}$  [ $m^2/s$ ] can be calculated from:

$$D_{nssm} = \frac{RT}{|z_{Cl}|F} \frac{\Delta x}{\Delta E} \frac{x_a - \alpha \sqrt{x_a}}{t} \quad (4)$$

$$\text{with } \alpha = 2 \sqrt{\frac{RT}{|z_{Cl}|F} \frac{\Delta x}{\Delta E}} \cdot \text{erf}^{-1} \left( 1 - \frac{2c_d}{c_0} \right) \equiv 2.5624 \sqrt{\frac{RT}{|z_{Cl}|F} \frac{\Delta x}{\Delta E}} \quad (5)$$

where R is the universal gas constant (= 8.314 J/mol·K), T is the average of the initial and final absolute temperatures in the 0.3 N NaOH solution [K],  $|z_{Cl}|$  is the absolute value of the chloride valence (= 1 equivalent/mol), F is Faraday’s number (=  $9.648 \cdot 10^4$  C/equivalent),  $\Delta x$

is the thickness of the concrete specimen,  $\Delta E$  is the voltage drop across the concrete specimen,  $x_a$  is the average penetration depth,  $\text{erf}^{-1}$  is the inverse error function and  $c_d$  and  $c_0$  are the chloride concentrations, respectively at which the colour changes ( $= 0.07 \text{ N}$ ) and in the 10 % NaCl solution ( $= 2 \text{ N}$ ). Note that  $\text{erf}^{-1}(1 - 2 \cdot 0.07/2) = \text{erf}^{-1}(0.93) = 1.2812$  (found in math tables or as a function in common computer spreadsheets).

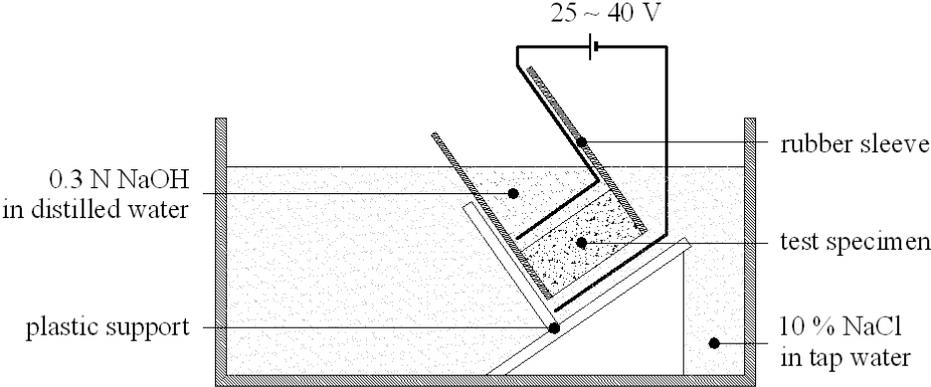


Figure 3: Schematic view of the CTH test setup

An overview of the test results is given in figure 4.

Note that the determination of the non-steady-state migration coefficient is most relevant for submerged concrete structures but less relevant for simulating the service life of concrete structures in tidal or ‘splashing’ zones (due to, e.g., the additional importance of capillary suction and evaporation effects in the chloride transport and environmental variables such as damage caused by erosion and/or frost attack).

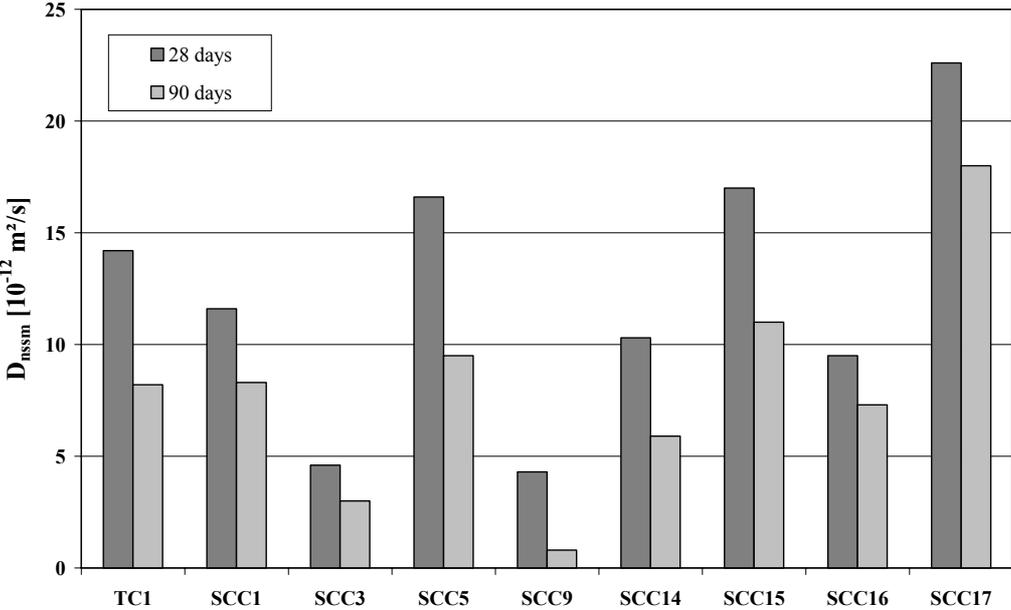


Figure 4: Overview of the measured non-steady-state migration coefficients  $D_{nssm}$

### 3.5 Discussion of test results

Before discussing the test results of the non-steady-state migration coefficient, it has to be pointed out that  $D_{ssm}$  and  $D_{nssm}$  can not just mutually be compared. The following theoretical relationship between the diffusion coefficients obtained from the CTH and the steady-state migration test is proposed by [4]:

$$D_{ssm} = D_{nssm} (\varepsilon + K_b W_{gel}) \quad (6)$$

where  $\varepsilon$  is the concrete porosity,  $K_b$  is the chloride binding factor and  $W_{gel}$  is the gel content (note that it is the hydrate gel which plays the roles of both adsorbent and reactant in chloride binding [4]). In the framework of this research project,  $\varepsilon$ ,  $K_b$  and  $W_{gel}$  were not determined. Besides, seen all the questions which arose during the determination of  $D_{ssm}$  (see § 3.3), the calculated relationship between  $D_{ssm}$  and  $D_{nssm}$  can be questioned too (in this case).

Let us now use  $D_{nssm}$  for ranking the different mixtures (SCC1 is always used as reference mix), keeping in mind that not all the binding will have occurred, which would also affect the results:

- We see that the W/C ratio is of great importance on the non-steady-state migration coefficient: mixtures SCC5, SCC15 (both W/C = 0.55) and SCC17 (W/C = 0.6) showed a higher  $D_{nssm}$ , both at 28 d and 90 d, while SCC14 (W/C = 0.4) showed a lower  $D_{nssm}$ , both at 28 d and 90 d.
- TC1 showed a higher  $D_{nssm}$  at 28 d but a comparable coefficient at 90 d, compared with SCC1.
- If CEM I 42.5 R is replaced by CEM III/A 42.5 LA (SCC3) a lower  $D_{nssm}$  is expected (due to its lower  $C_3A$ -content) and was observed (both at 28 d and 90 d).
- If fly ash (SCC9) instead of limestone filler is used as mineral additive,  $D_{nssm}$  is lowered considerably at 28 d and even more pronounced at 90 d.
- If crushed limestone (SCC16) instead of river gravel is used as coarse aggregate,  $D_{nssm}$  is lowered at 28 d but the difference is less pronounced at 90 d.

Keep in mind that a higher compressive strength can be seen as an indication of a more dense microstructure and so of a lower  $D_{nssm}$ .

Although no acceptance criteria are stated for the chloride diffusion coefficient in any Belgian concrete standard, the following guidelines for  $D_{nssm}$ , measured with the CTH method at 28 days after casting, can be used [5]:

- $D < 2 \cdot 10^{-12} \text{ m}^2/\text{s}$  → Very good resistance against chloride ingress
- $D < 8 \cdot 10^{-12} \text{ m}^2/\text{s}$  → Good resistance against chloride ingress
- $D < 16 \cdot 10^{-12} \text{ m}^2/\text{s}$  → Moderate resistance against chloride ingress
- $D > 16 \cdot 10^{-12} \text{ m}^2/\text{s}$  → Not suitable for aggressive environments

According to these guidelines, SCC3 and SCC9 show good resistance against chloride ingress, where TC1, SCC1, SCC14 and SCC16 show moderate resistance. SCC5, SCC15 and SCC17 exceed  $16 \cdot 10^{-12} \text{ m}^2/\text{s}$  and therefore are characterized as being not suitable for use in aggressive environments.

## 4. CARBONATION

### 4.1 Test setup

The carbonation depth was measured according to NT BUILD 357 (1989). For each concrete mix, 8 prisms (100·100·500 mm<sup>3</sup>) were fabricated. They were stored in a climate room at 20 ± 2 °C and > 90 % R.H. until a concrete age of 28 days. Afterwards, they were in a climate room at 20 ± 2 °C and 60 ± 5 % R.H. for another 14 days.

One long side was left unsealed. Other sides were sealed with a two-layer protective coating. In this way, only the unsealed side of the concrete specimen was exposed to the carbonation atmosphere. 4 prisms were placed in a carbonation chamber (CC) at 20 ± 2 °C, 60 ± 5 % R.H. and a CO<sub>2</sub>-concentration of 10 vol-%. The other 4 prisms were placed unsheltered outside (O). Of each climate condition, 3 prisms were tested and 1 was kept in reserve. At regular points in time (1, 14 and 28 days and 1/4, 1/2 and 1 year), the carbonation depth is measured by applying a phenolphthalein colour indicator on the saw-cut of a freshly cut-off fragment (± 50 mm) of the specimens. Sound concrete is coloured purple/red by the indicator while the colour of the carbonated concrete remains unchanged. The colour switchover takes place in the pH-range 8.3–10.0. The saw-cuts of the remaining prisms are re-cured. The carbonation fronts are evaluated by the following scheme [6]:

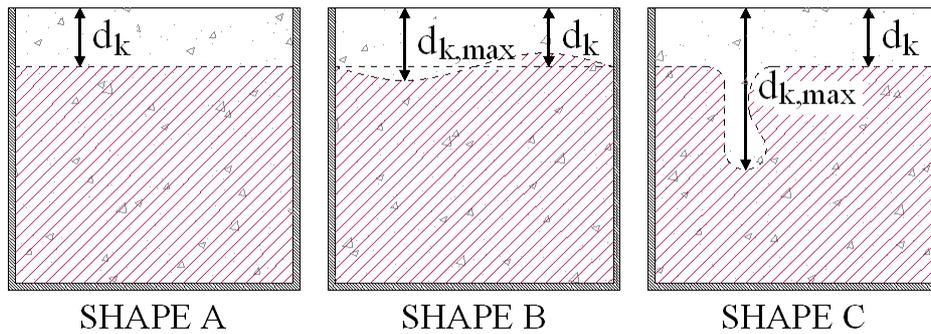


Figure 5: Defining the carbonation depth according to shape A, B and C

### 4.2 Test results

Normally, the test report should contain all data (shape,  $d_k$  and  $d_{k,max}$ ) for each measurement. In this case, it would lead to a considerable, hardly workable, table. Therefore, we decided to reduce the test results to the mean carbonation depth ( $d_{k,mean}$ ) as a function of time for each mixture. In this way, the carbonation constant A (mm/√year) could be deduced from linear analysis. The results are given in table 4. Concerning the specimens placed unsheltered outside (O), only the results after 1 year of carbonation exposure are represented. According to [7], the carbonation constant in natural conditions  $A_0$ , can be calculated as follows ( $[CO_2]_{CC} = 10 \% \equiv 183.1 \text{ g/m}^3$ ,  $[CO_2]_O \approx 0.8 \text{ g/m}^3$ ):

$$A_0 = A_{CC} \sqrt{\frac{0.8}{183.1}} \quad (7)$$

The calculated  $A_0$  is also given in table 4, allowing a comparison with the corresponding carbonation depth after 1 year. The following remark should immediately be made: due to the fact that the carbonation depth is determined every time on another saw-cut, keeping in mind

the inhomogeneous character of concrete, it is not surprising to find that the carbonation depth is not always increasing with time. In this way, the use of  $A_0$  as calculated above may be of more significance for concrete practice (see difference between  $A_0$  and carbonation depth after 1 year of “natural” exposure).

Let us now use  $A_{CC}$  for ranking the different mixtures (SCC1 is always used as reference mix):

- According to  $D_{nssm}$  (see above), the W/C ratio is also of great importance on the carbonation constant  $A_{CC}$ : mixtures SCC5, SCC15 (both W/C = 0.55) and SCC17 (W/C = 0.6) showed a higher  $A_{CC}$ , while SCC14 (W/C = 0.4) showed a lower  $A_{CC}$ .
- TC1 showed a higher  $A_{CC}$ .
- If CEM I 42.5 R is replaced by CEM III/A 42.5 LA (SCC3) a higher  $A_{CC}$  observed.
- If crushed limestone (SCC16) instead of river gravel is used as coarse aggregate, a somewhat lower but comparable  $A_{CC}$  is found.

Note that also here a higher compressive strength can be seen as an indication of a more dense microstructure and so a lower  $A_{CC}$  (similar to  $D_{nssm}$ ).

Table 4: Overview of carbonation depths  $d_{k,mean}$  and the carbonation constant  $A$  (mm/ $\sqrt{\text{year}}$ )

	TC1	SCC1	SCC3	SCC5	SCC14	SCC15	SCC16	SCC17
1 day (CC)	1.65	1.85	0.25	0.20	2.00	2.60	0.53	0.67
14 days (CC)	0.80	4.50	4.03	3.59	3.75	5.75	2.81	5.95
28 days (CC)	4.33	3.22	4.83	4.83	4.92	8.33	3.40	8.33
¼ year (CC)	8.33	7.00	8.50	4.67	7.33	13.92	4.00	11.83
½ year (CC)	15.75	12.08	15.00	15.17	8.33	18.25	10.50	20.33
1 year (CC)	12.75	12.00	19.83	18.67	10.67	23.17	14.00	25.50
1 year (O)	1.47	1.33	0.00	0.60	1.50	2.27	0.80	0.73
<b><math>A_{CC}</math> [mm/<math>\sqrt{\text{year}}</math>]</b>	<b>15.74</b>	<b>13.84</b>	<b>19.66</b>	<b>18.03</b>	<b>11.99</b>	<b>24.89</b>	<b>13.32</b>	<b>26.33</b>
$r^2$ [-]	0.80	0.86	0.98	0.90	0.82	0.97	0.93	0.98
<b><math>A_0</math> [mm/<math>\sqrt{\text{year}}</math>]</b>	1.04	0.91	1.30	1.19	0.79	1.65	0.88	1.74

## 5. CONCLUSIONS

In this research program, both the steady-state and the non-steady-state migration test were used to determine the chloride diffusion coefficient of different self-compacting concrete mixes. Also one traditionally vibrated concrete mix (TC1) was tested.

The results revealed that the determination of the steady-state migration coefficient according to NT BUILD 355 (1997) is far from easy and question marks could be placed besides the corresponding chloride diffusion coefficient. The main problems that occurred were corrosion of the stainless steel mesh at relative low power supply and the test duration (still no chloride ion “breakthrough” after 3 months) but no reason nor even a possible explanation for the observed anomalies could be found yet. Besides those steady-state migration tests, also non-steady-state migration tests according to NT BUILD 492 (1999) were executed. The corresponding chloride diffusion coefficient could also be used to rank the different concrete mixtures (even acceptance criteria were found in literature). In this way, the different self-compacting concrete mixes and the reference traditionally vibrated concrete mix could mutually be compared.

Using the carbonation constant, determined by use of an inflated CO<sub>2</sub>-concentration of 10 vol-%, the concrete mixtures were ranked in the same way as they were by the non-steady-state chloride diffusion coefficient.

In general, test results revealed the great importance of the W/C ratio and cement type on both the chloride diffusion coefficient and the carbonation depth. Keeping this in mind, it is possible to design self-compacting concrete mixtures which are more durable concerning chloride penetration and carbonation.

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