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Chloride Induced Corrosion in Cracked Reinforced Strain Hardening Cement-Based Composite (R/SHCC)

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ABSTRACT

This paper reports on corrosion of reinforcing steel bars in strain hardening cement-based composites (SHCC) under chloride attack. As part of a continued research project on durability of SHCC, the main focus here is on flexurally induced cracks in R/SHCC specimens with different cover depths. After unloading, these specimens are subjected to chloride attack, while monitoring steel corrosion. Two types of SHCC are studied, namely a fine grained SHCC, prepared with sand of maximum particle size 0.3 mm and a coarse SHCC prepared with a coarse sand of maximum particle size 1.7 mm. The crack patterns are characterized, in order to relate the crack width distribution, cover depth and corrosion. Crack widths of 50 μm are found to allow chloride penetration to rebar within hours, but several weeks of accelerated chloride exposure do not cause corrosion in the steel bars.

Keywords: corrosion, multiple cracks, strain hardening cement-based composite (SHCC).

INTRODUCTION

Structural concrete is usually specified by the slump value and compressive strength. Flexural strength, tensile strength and E-modulus are often calculated from the compressive strength by empirical relations in codes. However, this is not sufficient to guarantee the durability of concrete (Richard and Cheyrezy, 1994, Saravanan et al., 2010). The compressive strength of the concrete is a bulk property, while its durability is affected by the properties of the surface and near-surface region of the concrete in structures. Also, ingress of gas, liquid and water is controlled by the overall percolation of the pore network, while strength is usually controlled by the pore size in a concrete matrix. Durability is mostly influenced by concrete properties such as diffusivity, permeability and sorptivity. Thus, in order to design durability of concrete, it is required that properties other than compressive strength must be stipulated.

In concrete structures, one of the major durability problems is the corrosion of reinforcing steel, which reduces the expected life-span of reinforced concrete. Resistance to steel corrosion is considered to be dependant on the water-cement ratio, compatibility of ingredients, cover thickness, and density of the cover. Depassivation of the film surrounding the steel by ingress of deleterious gases, liquids and ions leads to the corrosion. Corrosion increases the volume of the embedded steel, causing pressure between the steel and the cover concrete. This imposes strain in the cover concrete which may exceed the tensile strength, leading to cracking and eventual spalling away from the steel. This scenario is very common for conventional concrete structures, necessitating expensive repairs or replacements. The formation of large crack widths in conventional reinforced concrete (RC) is one of the major problems from a structural durability point of view, as it is well known that large cracks allow fast penetration of gases and liquids into the concrete. Cracks in RC structures are unavoidable because of the low tensile strength and low elastic deformability of concrete. SHCC have been introduced over the last two decades and have been found to control crack widths. The finely cracked SHCC has been shown to reduce ingress rates, which hold potential for a delay in corrosion initiation and reduced corrosion rate in reinforced SHCC (R/SHCC) structures and thus an extended service life (Miyazato and Hiraishi, 2005, Sahmaran et al., 2008, Kobayashi et al., 2010). The results reported in this paper are part of a larger research project on SHCC at Stellenbosch University over the last number of years and the information given here is limited to the 56 day accelerated corrosion status observed in both fine sand (FS) and coarse sand (CS) pre-cracked R/SHCC beams.

DURABILITY OF SHCC STRUCTURES

In addition to energy dissipation by multiple crack formation, SHCC has the potential of high durability due to the small crack widths. The typical elastic strain limit of normal concrete (NC) ranges between 0.015% and 0.02%. This is similar for SHCC, but its mechanical resistance as well as crack control to fine widths can be maintained to strain levels of up to 3% or more (Sahmaran et al. 2008, Paul and van Zijl 2013b). These are particular useful characteristics for structural repair, but also new structural elements. In its short existence, SHCC has been applied as an overlay and repair material (bridge decks, tunnels, retaining walls) and as seismic damping structural elements in tall buildings. Full scale use of SHCC in structures has not yet been done and the reasons behind it may be the high initial cost of SHCC and also that limited information on SHCC performance is available as it is a relatively new material.

The limitation of crack widths to below 100 μm is generally believed to limit water permeability in concrete. However, cracks do act as pathways for fast penetration under conditions of capillary absorption (Zhang et al., 2010). In contrast, cracked SHCC has been shown to protect steel reinforcing bars against corrosion in patch-repaired reinforced concrete beams subjected to cyclic spraying with a salt solution (Kobayashi et al. 2010). It appears that, despite penetration of water and possibly chlorides and oxygen, the corrosion rate is low in SHCC. Miyazato & Hiraishi (2005) reported that no or little macro-cell corrosion could be observed in finely cracked R/SHCC, as opposed to significant macro-cell corrosion in R/mortar specimens with a wide crack. Characterisation of crack patterns in R/SHCC, and linking it to ingress rates and actual corrosive deterioration are required.

EXPERIMENTAL DESIGN

The research work is divided into four different parts namely direct tensile response, flexural response, corrosion and lastly chloride penetration in SHCC. This section maps the way in which the above-mentioned research objectives were executed.

Mix design of SHCC

To achieve the ductile behaviour of SHCC it is necessary to have a good mix design. So far in this research, more than three trial mixes of SHCC made with FS and CS have been made and tested and one set of their results has been shown in Paul and van Zijl (2013a & b). The SHCC mix is sensitive to changes in ingredient materials as they are received in different batches from industry. The mix used in this paper is a bit different than the previous mix, but nevertheless this specific mix of SHCC also shows ductile behaviour and multiple cracking. The mix design together with the slump flow value is shown in Table 1.

Table 1. Materials used in this study

	Materials (kg/m^3)								
Type	C	FA	S	W	F	SP	VA	AE	Flow (mm)
FS-SHCC	390	670	550	390	26	31.2	5.85	3.94	200-220
CS-SHCC	390	670	550	390	26	31.2	5.85	3.94	200-220

Note: C= cement, FA= Fly ash, S= fine or coarse sand, W=water, F= fibre, SP= super plasticizer, VA= viscous agent, AE= air entraining agent.

Test setup for mechanical behaviour

To characterise the mechanical behaviour of SHCC, the compressive strength, direct tensile and three-point bending (flexural) tests were performed. For determining the compressive strength and flexural response, 100 mm cubes and 100 x 100 x 500 mm beam respectively were used. The same specimens used in the flexural test were then used in the corrosion test. In direct tensile tests, small dumb-bells of 30 mm width, 16 mm thickness and with a gauge length of 80 mm were used. The cubes were tested at the age of 28 days while beams and dumbbell specimens were tested at 14 days. Two days after mixing, all specimens were removed from the mould and kept in a water tank (temperature 21 ± 2^{o}C) for 7 days. After 7 days all the specimens were removed from the water tank and stored at ambient laboratory temperature until the testing date. Details can be found in Paul and van Zijl (2013a,b).

Test setup for durability performance of SHCC

In the assessment of the durability performance of SHCC, testing of corrosion of steel and chloride penetration into cracked SHCC was performed. Typically the corrosion process starts in the uncracked specimen after a long time and it is fully depended on the type of concrete, cover depth, surrounding environment, etc. However in the limited time of a research project accelerated testing is required. In this study the cracked specimens were subjected to chloride-induced corrosion. The specimens were cracked in flexure up to a vertical deflection level of 3.5 mm just to form several cracks in the specimens. Single and double steel bar reinforcements were used in both FS-SHCC and CS-SHCC at three different cover depths (15 mm, 25 mm and 35 mm).

After applying the flexural loads in SHCC beam specimens, multiple cracks were found in the bottom face of specimens. This face was subsequently kept in contact with NaCl solution (3.5% wt of water) and corrosion was measured using a half cell potential (HCP) apparatus with a copper / copper sulphate (CSE) was used as a reference electrode. Only one directional penetration through a 200 mm length of cracked face was allowed by sealing four sides of the specimen with an epoxy coating together with a 150 mm length from both sides of the total cracked face, the details of which are shown in Figure 1. This paper is limited to corrosion measuring using the HCP method which does not give the corrosion rate (I_{corr}) in the specimens but rather predicts the corrosion probability and gives the corrosion potential (E_{corr}). However, in future work different corrosion measuring techniques will be used from which the corrosion rate might be determined.

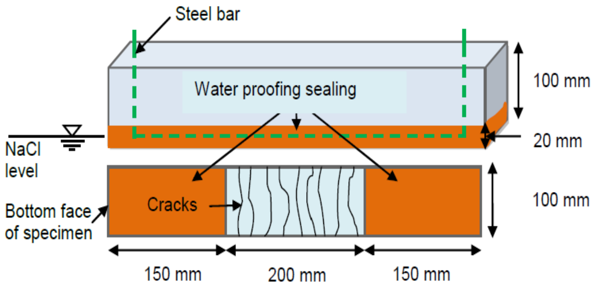


Figure 1. Corrosion measuring setup for the cracked specimen

The corrosion potential was measured in a ambient laboratory conditions (temperature 21 ± 2°C and relative humidity 55 to 65%). The bottom face of all specimens was kept in contact with NaCl solution for 21 days after which cyclic drying and wetting exposure was carried out to accelerate the corrosion potential rate. In the first week of the corrosion test, data were recorded every day and thereafter it was recorded randomly (at least one reading from each drying and wetting exposure). The exposure condition used here is not a standard method recommended by the codes because currently there is no specification for the SHCC test method. As a result researchers follow different exposure conditions for the SHCC corrosion test (Miyazato and Hiraishi, 2005, Sahmaran et al., 2008, Kobayashi et al., 2010) but none of them are compared to real life structural exposures. The exposure condition used here also needs to be compared to real life structural exposure conditions and perhaps the corrosion modelling needs to develop a link between experimental and real exposure conditions.

In order to study the chloride penetration through the cracked SHCC specimen, unreinforced SHCC specimens (sizes are the same as those of the reinforced SHCC specimens) were made and pre-cracked at 28 days in three point flexure. In this case, loads were applied up to a deflection level of 1.0 mm. After the flexural test, specimens of 150 mm long were cut from the middle portion of the 500 mm long beam. Similar to the corrosion specimens, four sides were sealed with epoxy coating and penetration was allowed through the cracked face with a length of 150 mm. A total of four specimens from both the FS-SHCC and CS-SHCC specimens were used and penetration was observed after 1, 3 and 6 hours. After these periods, specimens were cut again along the 150 mm length into two parts (50 mm width and 150 mm length). The details of specimen preparation can be found in Paul and van Zijl (2013b). The same NaCl solution was used here and penetration was measured by applying 0.1N $AgNO_3$ solution. The reaction of NaCl with $AgNO_3$ is of a different colour to that of the concrete surface. Typically when $AgNO_3$ is applied on the concrete surface, the presence of NaCl shows as a white gray colour but otherwise it becomes brown (Otsuki et al 1993). A similar trend was also found here and details of the results are shown next.

EXPERIMENTAL OUTCOME

The results from this research work are limited to the mechanical and durability performance of SHCC made from FS and CS. For the comparison of the results, FS-SHCC is considered to be the reference which is then compared with the results obtained from CS-SHCC. The details of the experimental outcomes are discussed below.

Compressive and tensile strength and strain of SHCC

A total of 6 cubes from each type of SHCC were tested for determining the compressive strength. The average 28 days compressive strengths were 24.22 MPa (coefficient of variation (CoV) 1.43%) and 23.1 MPa (3.84%) respectively for FS-SHCC and CS-SHCC.

In order to confirm strain hardening behaviour of both FS-SHCC and CS-SHCC, direct tensile tests were performed on a total of 4 dumbbell specimens of each SHCC type at 14 days and their results are shown in Figure 2. The average ultimate tensile strength of 2.76 MPa (CoV 7.54%) was obtained for CS-SHCC and of 2.67 MPa (4.85%) for FS-SHCC. The first crack strengths were 2.46 MPa (7.03%) and 2.04 MPa (3.91%) respectively for CS-SHCC and FS-SHCC. An average ultimate tensile strain of 2.93% (CoV 16.39%) was obtained for FS-SHCC and 1.52% (CoV 103.7%) for CS-SHCC. Higher first crack and ultimate strengths lead to a lower ultimate strain of CS-SHCC. In previous work an ultimate strain of more than 3% was obtained for CS-SHCC at 14 days age (Paul and van Zijl 2013b).

Flexural deflection and cracks in SHCC

A total of 18 beams of each type of SHCC were tested for corrosion. All of them were pre-cracked in flexure. For each cover depth of concrete, two types of reinforcement (single and double bar) were used. For each cover depth, a total of 6 beams, 3 with a single bar and 3 with two bars were tested for both types of SHCC. Their flexural test results are shown in Figure 3a,b for one specimen of each cover depth. C15, C25 and C35 denote different cover depths and B1 and B2 denote a single bar and double bar specimens.

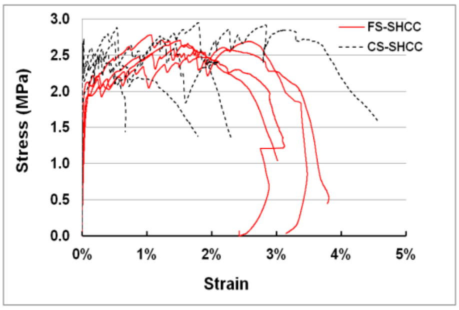


Figure 2. SHCC stress strain response in a tensile test

Table 2 shows the SHCC beam results in three point flexural testing up to a deflection level of 3.5 mm. A maximum load of 52.73 kN was obtained for the CS-SHCC specimen with C15 and B2. In case of B1, a maximum load of 44.43 kN was obtained for the FS-SHCC specimen with C15. In most cases, the largest loads were found in the specimens made from FS-SHCC. It is worth mentioning that for the specimens with B2, two cycles of loads were applied in both types of SHCC. The reason is that in the first cycle (up to a deflection level of 2.9 mm) the crack widths were very fine (less than 50 μm) which is thought to be below the threshold level of chloride penetration. Therefore in the second cycle of loading, all the specimens were loaded again up to a deflection level of 3.5 mm and this time crack widths were found to be larger than 50 μm. After the first cycle of loading, a permanent deflection of about 0.5 mm was observed in all specimens and after the second cycle of loading the total deflection in all specimens was about 4 mm. However, this is not shown in the Figures 3a & b, as only one final cycle is shown there. In the case of B1, only one cycle of loading was applied in both types of SHCC.

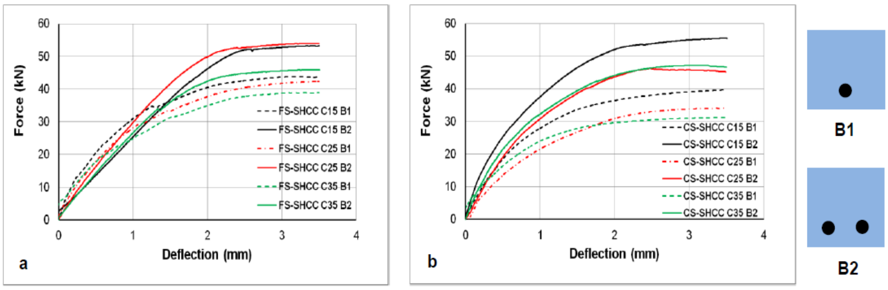


Figure 3. FS-SHCC (a) and CS-SHCC (b) beams response in three-point flexural up to 3.5 mm deflection level

Table 2. Flexural beam response up to the vertical deflection level of 3.5 mm

Type	SHCC cover (mm)	No of steel bars	Ave. Load (kN)	CoV (%)
FS-SHCC	15	1	44.43	1.67
		2	50.18	8.65
	25	1	41.14	4.13
		2	48.67	10.07
	35	1	39.12	0.92
		2	45.27	8.42
CS-SHCC	15	1	38.11	4.67
		2	52.73	4.66
	25	1	33.28	2.33
		2	47.11	1.86
	35	1	29.89	3.85
		2	41.23	1.67

For the unreinforced SHCC specimens an average maximum load of 17.06 kN was required to cause a 1 mm vertical deflection in the FS-SHCC specimens while in the case of CS-SHCC specimens the load was 14.73 kN for the same level of deflection.

The total number of cracks and crack widths were also examined in the specimens from the flexural test results. As mentioned earlier, only a 200 mm length in the bottom face of the specimens was considered for the path of chloride penetration for the corrosion test and so in Figures 4a & b, the number of cracks and crack widths are shown from that specific region only. In both SHCC specimens, maximum cracks were found in those specimens with B1. The crack widths were measured with the aid of a Leica MZ 7.5 microscope and a line width template. Most crack widths were in the range of 50 to 100 µm, but a few crack widths were found to be in the range of 100 to 200 µm. In the case of B2, most cracks were perpendicular to the steel bar and no parallel cracks were found while in the case of B1, apart from multiple perpendicular cracks, some cracks parallel to the steel bar were also observed in most specimens. In unreinforced SHCC, a maximum of 5 cracks of between 50 to 100 µm wide were found in both FS-SHCC and CS-SHCC specimens.

Corrosion in SHCC

The corrosion mechanism in SHCC is believed to be different from that of NC and therefore the typical corrosion potential range for NC in the assessment of the corrosion status by ASTM C876 appears not to be applicable for SHCC and may need to be calibrated. As per ASTM recommendation, an E_{corr} value of less than -350 µV indicates a high possibility of steel corrosion. Ahmed and Mihashi (2010) also found the similar trend (much lower than -350 µV) of E_{corr} using HCP in SHCC. In this research work a total of 36 FS and CS-SHCC specimens were used and in most cases reading of less than -350 µV were recorded until 49 days. After this period again a trend of lower potential values were observed in all specimens. No major difference was found between FS-SHCC and CS-SHCC corrosion potential value. However, in both types of sand, a cover depth of 15 mm SHCC shows a lower corrosion potential than the others.

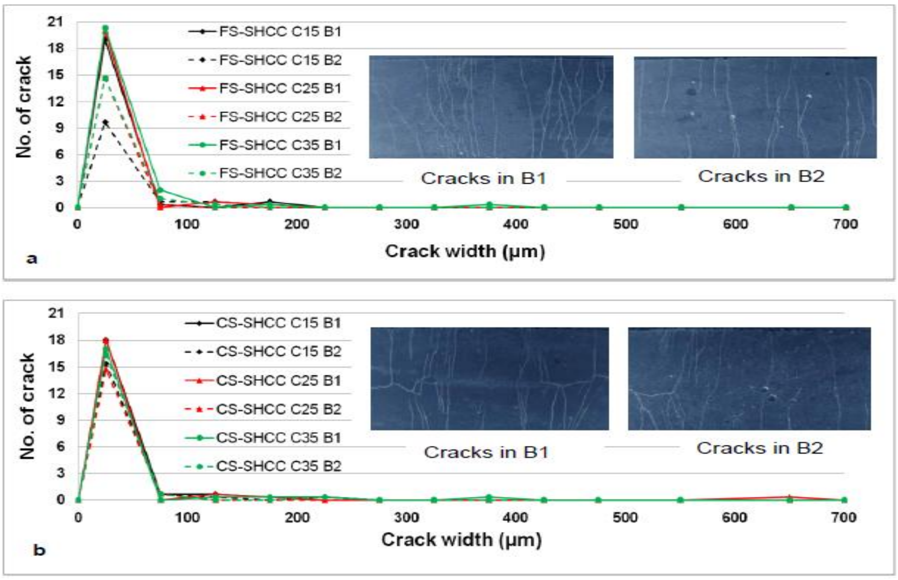


Figure 4. Flexural cracks in FS-SHCC (a) and CS-SHCC (b) beams

During the the first 21 days of continuous wetting exposure, the potential rate reduced with time and after this period when drying exposure started, a sudden increase in the potential rate was observed in all specimens. In each drying exposure the higher potential rate was observed and it can be explained by lower conductivity in this exposure than during the wetting exposure. Also HCP value depends on so many factors like temperature, humidity, moisture content in the specimens etc. (ASTM C876) which may be another cause of higher values in drying exposure than the wetting exposure. After the drying exposure a lower potential rate was again found during the next wetting exposure. During the total of 56 days of corrosion testing reported here the potential rates of 6 drying exposures (4 days) and 6 wetting exposures (one 21 days and four for 3 days) were monitored and their results are shown in Figures 5 and 6. Note that the results in these figures are the average results taken from the middle of 3 specimens for each cover depth. So, each data point is the average of 3 and 6 readings for B1 and B2 respectively.

As the HCP method indicates corrosion probability only, a total of three specimens from both SHCC were destructed for visual observation of the corrosion status of reinforcing steel bars (see Figure 7). After 17 days, one CS-SHCC C15 B1 specimen was destructed but no active corrosion found at this stage. Again after 52 days two specimens (FS-SHCC C35 B1 and FS-SHCC C15 B2) were destructed and the corrosion status was monitored visually. In all specimens, corrosion activities were higher where there the most cracks were but no pitting or reduction of steel was found at this level. From Figure 7 it is clear that the higher discolouration in the steel bars was observed at 52 days than 17 days. Of course visually it is not possible to quantify the exact corrosion status in steel bars. In future work, the linear polarization method will also be used to monitor the corrosion in SHCC with mortar as reference.

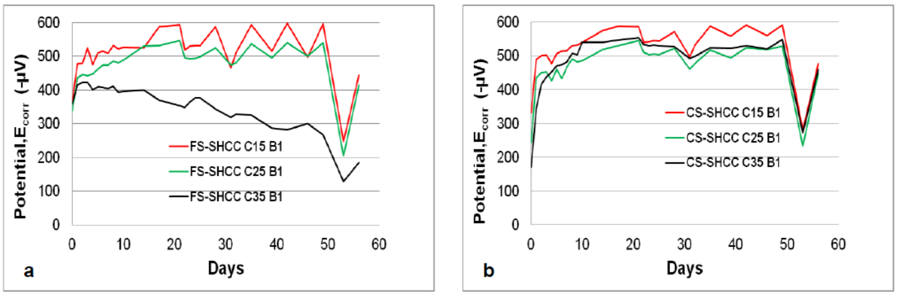
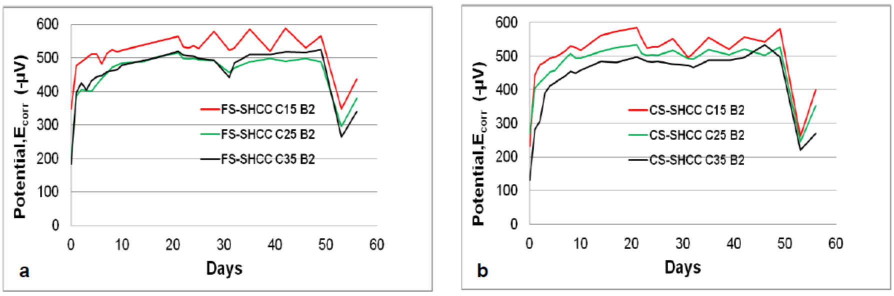
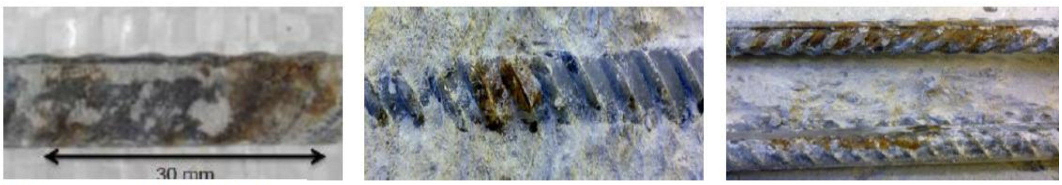


Figure 5. Corrosion potential of SHCC with 1 bar (B1) for different cover depth (C15, C25, C35) (a) FS-SHCC (b) CS-SHCC





17 days (CS-SHCC C15 B1) 52 days (FS-SHCC C35 B1) 52 days (FS-SHCC C15 B2)

Figure 6. Corrosion potential of SHCC with 2 bar (B2) for different cover depth (C15, C25, C35) (a) FS-SHCC (b) CS-SHCC

Figure 7. Corrosion inspection in SHCC by destruction at different days

Chloride penetration in SHCC

Figure 8 shows the chloride penetration through the cracked SHCC specimens at different time periods. The highest penetration depth was found in the specimens after 6 hours of exposure. It was also interesting to see that the penetration was widening, i.e expanding horizontally along the crack height with time. A crack width range of between 50 to 100 μm provides a quick path for chloride penetration into the SHCC. In future work, the determination of two-dimensional chloride profiles in the SHCC is envisaged.

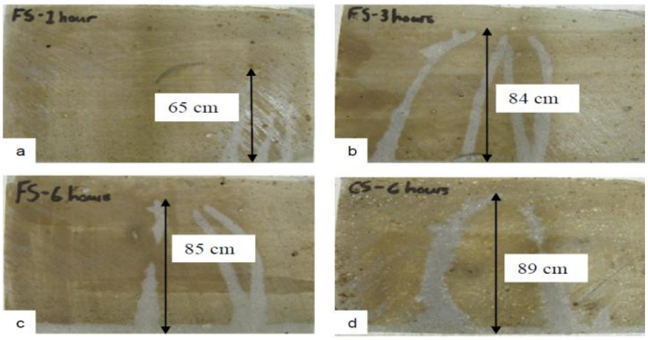


Figure 8. Chloride penetration in SHCC at different time periods

CONCLUSIONS AND RECOMMENDATIONS

This paper presents preliminary results of chloride penetration and chloride-induced corrosion in FS-SHCC and CS-SHCC specimens. In terms of the crack patterns in tension and flexure, the following conclusions can be drawn:

- No significant difference in the number of cracks and crack widths was observed for these two types of SHCC under uniaxial tension, although a significantly lower ultimate tensile strain was found for CS-SHCC. However, the ultimate strain level in CS-SHCC of roughly 1.5% may still be sufficient in many structural applications.
- Under the same deflection level, a similar number of cracks and crack widths can be found in both CS and FS-SHCC beams.

Crack widths ranging between 50 to 100 μm are sufficient to allow chloride to penetrate deep into R/SHCC within hours. Subsequently, the chloride gradually ingresses from the cracks into the uncracked matrix.

In terms of the corrosion probability testing through half cell potential readings over a period of 56 days of chloride exposure in wetting and drying cycles of both FS-R/SHCC and CS-R/SHCC, the following conclusions can be made:

- The half cell potential readings indicate a higher probability of corrosion for smaller cover depth.
- The ASTM recommendation regarding the corrosion potential status for NC may not be suitable for SHCC, and should be calibrated for SHCC.
- After 56 days of exposure, no significant corrosion was detected. Discolouration of the steel in the region of cracks was evident, but no pitting or reduction in bar diameter were found.

Until now the HCP results are not showing the real corrosion status in SHCC and long term experimental and field study may be required to understand the corrosion mechanism in SHCC.

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