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Mechanically Induced Cracking Behaviour in Strain Hardening Cement Based Composites (SHCC) at different service load levels
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Scientific paper

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

Suvash Chandra Paul* and Gideon P. A. G. van Zijl

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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 reinforced SHCC (R/SHCC) specimens with different cover depths. After unloading from the flexural test, these specimens are subjected to chloride attack, while monitoring steel corrosion. The R/SHCC specimens comprised series manufactured with two different types of sand, reinforced with single and double tensile steel bars with three different cover depths, in order to relate the crack patterns, cover depth and rebar corrosion. Crack widths of below 50 μm are found to allow chloride penetration to the rebar within hours. Corrosion potential and corrosion rate measurements, following a Coulometric method, are reported. By removal of rebars from a number of specimens after more than a year of chloride exposure, corrosion damage is studied by visual observation, pitting depth measurements and tensile testing of the rebars. Chloride profiles are also determined through XRF in cracked and uncracked regions of the specimens. Relation of corrosion damage calculated from corrosion rate measurements, with observed corrosion damage is complicated by localised corrosion.

1. Introduction

In conventional reinforced concrete structures (RCS), one of the major durability problems is the corrosion of reinforcing steel, which reduces the expected life-span of RCS. Resistance to steel corrosion is considered to be dependant on the concrete water to cement ratio, compatibility of ingredients, cover thickness, and density of the cover. Depassivation of the passive 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 common for conventional concrete structures, necessitating expensive repairs or replacements. The process is accelerated by the formation of wide cracks allowing fast penetration of gases and liquids into the concrete. Cracks in RCS are unavoidable because of the low tensile strength and low elastic deformability of concrete. Strain hardening cement-based composites (SHCC) have been introduced over the last two decades and have been found to control crack widths (Li et al. 2001).

Corrosion in RCS occurs mainly in two ways, namely chloride-induced corrosion and carbonation corrosion. Corrosion itself a very slow process and requires time to observe severe damage in structures. As a result laboratory experimental studies use accelerated test methods. Even though the accelerated tests are not directly linked or calibrated to real exposures, they are beneficial for relative comparison of different materials at the same exposure conditions. Recent accelerated chloride-induced corrosion experimental results of reinforced mortar (R/mortar) and hybrid fibre reinforced cement-based composites (R/HFRCC) beams, pre-cracked in flexure to various maximum crack widths (0.12 - 0.36 mm in R/mortar, 0.12 - 0.51 mm in R/HFRCC), were reported to indicate a correlation between the initial crack width and the corrosion rate, steel mass loss, and residual rebar tensile strength (Mihashi et al. 2011). Unfortunately the crack width distribution, number of cracks and crack spacing were not reported. However, the finely cracked SHCC has been shown to reduce ingress rates, which fact holds potential for reducing corrosion rates 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 620 days of chloride induced accelerated corrosion observed in both fine sand (FS) and coarse sand (CS) pre-cracked R/SHCC and mortar beams.

2. Durability of SHCC Structures

The association of crack width limitation with reinforce-
ing steel bar corrosion resistance has been reinforced by observations that water permeability rate of cracked concrete becomes significantly higher once a threshold crack width level, roughly 0.10 – 0.15 mm is exceeded (Lepech and Li 2009). Recent research results indicate that, whilst effective chloride diffusion coefficients are lower in cracked R/SHCC than in R/mortar beams (Sahmaran et al. 2007), cracks act as pathways for fast ingress deep into mortar (Zhang et al. 2010; Wittmann et al. 2011; Altmann 2012).

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 and 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. Maalej et al. (2003) reported relatively insignificant corrosion in steel bar reinforced ductile fibre reinforced cementitious composite (R/DFRCC) beams containing multiple cracks, compared with the corrosion in cracked RC beams in accelerated chloride-induced corrosion tests. It has been postulated that the steel bar interface with the surrounding cement-based matrix plays a significant role in corrosion protection. Experimental evidence has been reported that a strong interface layer surrounding the steel increases the corrosion initiation period (Mohammed et al. 2013). It is also postulated that the increased tensile capacity brought about by the fibres in SHCC, limits the corrosion-induced volume expansion of the steel bar, whereby the rate of corrosion is subdued. Schiessl and Raupach (1997) reported that, for an increased crack spacing in RC from 10 to 20 cm, the rate of corrosion doubles, implying a lower corrosion rate for smaller crack spacing in this range. In RC, smaller crack spacing is usually achieved by using more, but smaller diameter steel bars. However, hereby an increased steel bar surface area is created. Upon exposure to chloride, the larger depassivated surface area of steel will accelerate the corrosion (Schiessl and Raupach 1997). This motivates the application of R/SHCC to control crack spacing by the inherent crack control of SHCC (van Zijl and Paul 2013).

In this paper, corrosion rate is measured in pre-cracked R/SHCC beam specimen with test parameters the steel bar reinforcement level (1 tensile steel bar versus 2 bars), cover to the steel (15, 25 and 35 mm) and fine aggregate type (a fine sand, versus a natural coarse sand). The intention is to illuminate the role of the above parameters, but also that of crack width and spacing.

3. Experimental Design

In the interest of SHCC cost reduction, a natural sand with particle size up to 1.7 mm which is classified as coarse sand (CS) in terms of aggregates commonly used in SHCC, is used in the experimental study and compared with fine sand (FS) SHCC with particle size up to 0.30 mm. The SHCC mixes, test parameters of cover and reinforcement level, as well as test procedures for pre-cracking, accelerated chloride exposure, XRF detection of chloride profiles, corrosion potential and rate measurements are described in the following sections.

3.1. Mix design of SHCC

The mix designs used in this research work are shown in Table 1. Viscosity modifying agent (VA) and superplasticiser (SP) were used to create the appropriate rheology for optimal fibre dispersal and avoiding segregation. An air entraining agent (AE) was used to form internal voids which may enhance multiple crack formation. Polyvinyl alcohol (PVA) fibres were used, with fibre length and diameter 12 mm and 0.04 mm respectively, nominal tensile strength of 1600 MPa and E-modulus of 40 GPa. High tensile steel reinforcing bars, with average yield stress of 510 MPa and nominal diameter 10 mm were used, denoted Y10 reinforcing bars. A high water to cement (w/c) ratio of 1.0 was used, which is at the upper limit in the w/c ratio range (0.3 to 1.0) that has been used for SHCC (van Zijl and Wittmann 2010). In normal reinforced concrete, such high w/c ratios are not allowed in prescriptive durability design rules, but here it is shown to maintain low corrosion rates in R/SHCC.

3.2. Test setup for mechanical behaviour

For mechanical characterisation of SHCC compressive tests (100 mm cubes, and 100 mm diameter by 200 long cylinders), direct tensile and three-point bending tests were performed on 100 x 100 x 500 mm beams. The same specimens used in the flexural test were subsequently used in chloride-induced corrosion tests. For direct tension, small dumbbell specimens with a central cross section of 30 mm wide and 16 mm thick over a gauge length of 80 mm were used. The cubes and cylinders were tested at the age of 28 days while beams and dumbbell specimens were tested at 14 days. Two days

<table>
<thead>
<tr>
<th>Table 1 Materials used in this study.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Materials proportions (kg/m³)</strong></td>
</tr>
<tr>
<td><strong>Type</strong></td>
</tr>
<tr>
<td>FS-SHCC</td>
</tr>
<tr>
<td>CS-SHCC</td>
</tr>
<tr>
<td>FS-Mortar</td>
</tr>
<tr>
<td>CS-Mortar</td>
</tr>
</tbody>
</table>
after mixing, casting and protection (covering) of the specimens in the laboratory, all specimens were removed from the moulds and kept in a curing tank with water temperature 21 ± 2°C for 7 days. After 7 days all the specimens were removed from the water tank and stored at ambient laboratory temperature (temperature 21 ± 2°C and relative humidity 60 ± 5%) until the testing date. These specimens are the same as those used for mechanical characterisation of SHCC, of which more details can be found in Paul and van Zijl (2013a, b).

3.3. Test setup for chloride penetration and corrosion

Pre-cracked specimens were subjected to chloride-induced corrosion. The reinforced specimens were cracked in three-point bending up to a vertical deflection level of 3.5 mm for R/SHCC and 1.5 mm for R/mortar to form several cracks in the specimens. The reasons for choosing different deflection levels for SHCC and mortar specimens are that (i) the deflection level at ultimate resistance of SHCC and mortar are different and (ii) for the same level of deflection in SHCC and mortar specimen, the crack width is much different. In this research the expected average crack widths in both SHCC and mortar was below 0.10 mm and the deflection levels of SHCC and mortar specimens were chosen such that crack width does not cross this limit. 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). Only 15 mm cover depth with single bar was used in FS and CS-Mortar specimens. Three specimens of each type were prepared, thus in total 2x2x3x3=36 R/SHCC and 2x3=6 R/mortar beams. Figure 1 shows the preparation for the beam specimens, including sealing of the sides with an epoxy coating to limit absorption to the cracked central 200 mm.

Multiple cracks were found in the bottom face of R/SHCC specimens, but only a few cracks in R/mortar specimens. The cracked face was subsequently kept in contact with NaCl solution (3.5% wt of water) in a cyclic drying (4 days) and wetting (3 days) exposure condition. Corrosion potential \( E_{corr} \) was measured using a copper / copper sulphate (CSE) half cell potential (HCP) apparatus. Corrosion rate \( V_{corr} \) in the steel bar was measured by a coulostatic technique in laboratory conditions (temperature 21 ± 2°C and relative humidity 60 ±5%). In laboratory testing, researchers followed 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 specimens, two approaches were followed.

(i) Additional, unreinforced SHCC beam specimens of the same size as the R/SHCC specimens were made and pre-cracked at 28 days in three point bending. In this case, loads were applied up to a deflection level of 1.0 mm. After the flexural testing, specimens of 150 mm long were cut from the middle portion (all cracks were limited to this portion) of the 500 mm long beam. Similar to the corrosion specimens, four sides were sealed with epoxy coating and capillary 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 in capillary suction 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 same NaCl solution was used here and penetration depth 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 grey colour but otherwise it becomes brown (Otsuki et al. 1993).

(ii) Chloride penetration profiles were drawn for R/SHCC and R/mortar specimens. In this case, a total of seven specimens were chosen after 560, 580 and 610 days of cyclic chloride exposure. Chloride (Cl) profiles were then obtained by drilling from the exposed crack surface in layers of 3 mm each, up to a depth of 45 mm. Powder samples were collected from the drilling (over the crack region) residue of the specimens, after which X-ray fluorescence (XRF) was used to determine the Cl content in each layer. A drill with diameter of 16 mm was used to ensure sufficient powder per 3 mm layer. This means that material directly adjacent to cracks was also included in the powder samples, which is unavoidable due to the required powder specimen volume and finite drill size. In addition, determination of crack lengths is complicated by the fineness of cracks in these specimens. It is believed that all cracks extended at least to beyond the reinforcing bar.

3.4. Corrosion measuring technique

A Coulostatic method was used, which is a type of polarization measuring technique where a small amount of a known current \( (I) \) is passed through the steel for a...
known amount of time ($\Delta t$) while the potential ($\Delta E$) decay is observed. Following from this, the polarization resistance ($R_p$) of the concrete can then be determined from the $\Delta I/\Delta E$ ratio. This measuring technique is a replication on the Randles circuit which discharges with an exponential function. Once $R_p$ is known, the corrosion current ($I_{corr}$) ($\mu$A/cm$^2$) and from that corrosion rate ($V_{corr}$) (mm/year) can also be determined. The polarization resistance $R_p$ is defined as the steady state resistance that the metal interface presents to a change in potential $\Delta E$ when the perturbation $\Delta i$ is small. Typically $\Delta i$ is selected so that the values for $\Delta E$ are no more than ± 20mV. Due to the extreme sensitivity of the equipment, disturbances during testing can have an effect on the results obtained. To overcome this problem, some researchers used a higher $\Delta E$ value (Gonzalez et al. 2001). So during corrosion testing, if no further current is applied to the steel bar, the potential decays can be represented exponentially with time as follows:

$$\eta_t = \eta_0 \exp \left( \frac{-t}{\tau_c} \right)$$

(1)

where, $\eta_t$ is the potential shift ($\Delta E$) at time $t$ and $\eta_0$ is the initial potential shift. Polarization resistance $R_p$ is then obtained from the time constant ($\tau_c$) and interfacial capacitance ($C$) as follows:

$$R_p = \frac{\tau_c}{C}$$

(2)

$$C = \frac{q_s}{A\eta_0}$$

(3)

$$q_s = \Delta i x \Delta t$$

(4)

$$I_{corr} = \frac{B}{R_p}$$

(5)

$$V_{corr} = 0.0116 I_{corr}$$

(6)

where, $q_s$ is the amount of charge, $\Delta i$ is the pulse duration. $\eta_0$ and $\tau_c$ values can be determined by drawing an exponential function is fitted to the perturbation (mV) vs time (sec) curve. $A$ is steel surface area ($\pi d_s L_s$) and $B$ is the Stern-Geary constant varying from 26 to 52 mV depending on the passive and active corrosion. In this research project the $B$ value was considered to be 26 mV. This corrosion rate measuring technique was broadly explained by the Glass (1995), Gonzalez et al. (2001), Andrade and Alonso (2004) and Otieno et al. (2010).

The Y10 steel reinforcing bar acted as the working electrode (WE) at 15, 25 and 35 mm cover depths respectively, and a 15 mm thick and 120 mm long steel plate was placed on top of each specimen as the counter electrode (CE), while a Ag/AgCl half-cell was used as the reference electrode as seen in Fig. 2a. Figure 2b shows the assumed uniformly corroded steel bar section where $d_c$ is the corroded depth. For the corrosion rate measurement, a Spider8 data logger was used while a current of 4 mA was applied to each of the specimens for a period of 5 mS by a laboratory built current pulse generator. However, with this particular test setup and the equipment being used, measurements of such small voltages can prove to be difficult. Large volumes of unshielded materials such as concrete and steel pick up high frequencies such as radio waves and noise which will have an impact on the result yielded for the test. This problem was overcome by taking a 10 to 20 point moving average and if necessary, the offset above zero was corrected by subtracting the average value of the data points before the pulse from all the data points in the record. The perturbation range throughout the time of taking readings fell between 200 and 1500 mV which are higher than typical ranges found in normal concrete. This higher perturbation found in SHCC materials could be explained by the vastly different matrix when compared to normal concrete. The differences in resistivity and permeability in SHCC materials might play a large role when the current is applied to the specimens. So far no other research has been found where this method has been applied to R/SHCC materials.

After a period of accelerated testing some of the specimens were broken to observe actual corrosion status in the steel bar. The outcomes like loss of rebar tensile resistance and pitting depths from this observation were then compared with the experimental outcomes. In this regard, eqs (7) and (8) were used to esti-

![Fig. 2 (a) Corrosion rate measuring procedure using the Coulometric technique (b) corroded bar section.](image-url)
mate uniformly corroded depth \(d_{c0}\) with time and change in rebar yield resistance due to cross section reduction \(\Delta F_y\):

\[
d_{c}(t) = d_{c0} + \int_0^t V_{cor} \, dt = d_{c0} + \sum_{i=1}^{n} \frac{1}{2} (V_{cor} + V_{cor,i}) (t_i - t_{i-1}) (7)
\]

\[
\Delta F_y(t) = \frac{\pi\sigma_y}{4} [d_{c} - 2d_{c0}]^2 (8)
\]

In these equations \(d_{c0}\) is the initial corroded depth, \(t_i\) corroding period, \(\rho_s\) is the steel density, \(L_s\) is the length of steel (here it was assumed 500 mm), \(d_s\) is the steel diameter and \(\sigma_y\) is the nominal yield stress of steel. Note that in this test series, the original rebar mass was not recorded, so the mass loss estimated from the corrosion rate is not compared with measured mass loss directly.

4. Experimental Outcome

4.1. Compressive and tensile strength and strain of SHCC

A total of 6 cubes and 4 cylinders from each type of SHCC was tested for determining the compressive strength \(f_{cu}\) and E-Mod of each SHCC and their average 28 days values are given in Table 2. Slump flow at fresh state of each mix also given in Table 2. Note that the different slump values for same SHCC can be explained for different batch of mixing.

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 typical responses are shown in Fig. 3 and Table 2. Almost similar average ultimate tensile strength was found in both SHCCs, and average first cracking strengths 2.04 MPa (7.54%) and 2.46 MPa (7.03%) respectively for FS-SHCC and CS-SHCC. Significantly higher average ultimate tensile strain \(\varepsilon_{\text{max}}\) was obtained for FS-SHCC than CS-SHCC. This lower strain capacity of CS-SHCC is attributed to large sand particles, where crack localisation is likely to occur already at lower strain level. Nevertheless, the ultimate strain of 1.52% of CS-SHCC is considered to be sufficient for many applications.

4.2. Flexural cracks in SHCC

As described in section 3.3, a total of 42 pre-cracked, R/FS and R/CS-SHCC and R/FS and R/CS-Mortar beams were tested for corrosion. For each cover depth, a total of 6 beams, 3 with a single bar and 3 with double bars were tested for both types of R/SHCC. 3 FS-Mortar and 2 CS-Mortar specimens with 15 mm cover and single bar reinforcement were also tested, with the third CS-Mortar beam lost due to pre-damage. The flexural responses of R/SHCC beams are shown in Figs. 4a,b for one specimen of each cover depth. Note that C15, C25 and C35 denote different cover depths and B1 and B2 denote a single bar and double bars in the specimens.

In the case of unreinforced beams a total of 6 beams were used for each type of SHCC. An average maximum load of 17.06 kN was required to cause a 1 mm vertical deflection in the FS-SHCC specimens while in

![Fig. 3 SHCC stress strain response in a tensile test.](image)

![Fig. 4 SHCC with B2 (a) and SHCC with B1 (b) beams response in three-point flexural up to 3.5 mm deflection level.](image)
the case of CS-SHCC specimens the load was 14.73 kN for the same level of deflection. These unreinforced beams were used for AgNO₃ testing of chloride penetration, reported in section 4.6.1.

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 Figs. 5a-e, the number of cracks and crack widths are shown from that specific region only. In both SHCC specimens, a larger number cracks were found in the B1 specimens than B2. The crack widths were measured with the aid of a high resolution camera together with 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 width between 50 and 100 µm wide were found in both FS-SHCC and CS-SHCC specimens. Shear cracks were observed in some of the B2 specimens. It was also observed that most cracks grew to a height of approximately 70 mm from the bottom of the specimens. Once the specimens were unloaded, crack widths were significantly reduced. Details about the crack widths and spacing in the specimens can be found in Paul and van Zijl (2013b), and a summary given in Table 3 in terms of average crack width (ACW), maximum crack width (MCW) and total crack width (TCW) for the respective specimens.

### Table 3 Details of cracks and Cl content in the reinforced specimens.

<table>
<thead>
<tr>
<th>Type</th>
<th>NOC</th>
<th>ACW (µm)</th>
<th>MCW (µm)</th>
<th>TCW (µm)</th>
<th>Cl (% wt of binder)</th>
</tr>
</thead>
<tbody>
<tr>
<td>FS-SHCC C15B1</td>
<td>18</td>
<td>24</td>
<td>50</td>
<td>452</td>
<td>0.50 - 0.30</td>
</tr>
<tr>
<td>FS-SHCC C25B1</td>
<td>14</td>
<td>36</td>
<td>80</td>
<td>503</td>
<td>0.50 - 0.40</td>
</tr>
<tr>
<td>CS-SHCC C25B1</td>
<td>11</td>
<td>38</td>
<td>120</td>
<td>300</td>
<td>0.47 - 0.34</td>
</tr>
<tr>
<td>CS-SHCC C35B1</td>
<td>5</td>
<td>22</td>
<td>50</td>
<td>112</td>
<td>0.30 - 0.24</td>
</tr>
<tr>
<td>FS-Mortar C15B1</td>
<td>5</td>
<td>139</td>
<td>250</td>
<td>695</td>
<td>0.30 - 0.24</td>
</tr>
</tbody>
</table>

Note: Number of cracks (NOC), average crack width (ACW), maximum crack width (MCW) and total crack width (TCW).

Fig. 5 Flexural cracks in R/SHCC and R/Mortar specimens.
4.3. Corrosion in R/SHCC

4.3.1. Corrosion potential ($E_{corr}$) in R/SHCC and R/Mortar specimens

The corrosion mechanism in cracked SHCC is believed to be different from that of NC and therefore the typical corrosion potential range for normal concrete 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 mV indicates a high possibility of steel corrosion. In the 41 R/SHCC and R/mortar specimens in the current study, HCP readings of less than -350 mV were recorded in most cases; see Figs. 6a-c. Within the first day, some readings were less than -350 mV. Ahmed and Mihashi (2010) also found a similar trend of $E_{corr}$ using HCP in SHCC. The HCP value is influenced by several factors, including moisture content in the specimen, presence of oxygen, and temperature.

A reason for the low HCP readings may of course be corrosion starting already at an early stage, due to quick ingress of water and chlorides in the cracks. It must be kept in mind that the HCP indicates corrosion potential and not necessarily an actual corrosion process. The presence of water and chloride in the vicinity of the steel influence the electro-chemical process, and therefore the HCP reading, even without an active corrosion process occurring. In addition to moisture, it is postulated that the presence of chloride at the steel surface, as illustrated in Fig. 7, influences the corrosion potential reading. These phenomena are considered next.

4.3.2. Corrosion rate ($V_{corr}$) in R/SHCC and R/Mortar specimens

Figures 8a-c show the corrosion rate ($V_{corr}$) in mm/yr in all R/SHCC and R/Mortar specimens. It appears from the figures that higher $V_{corr}$ values are prevalent in specimens with C15, with relatively similar values in C25 and C35 specimens. In terms of type of sand in SHCC, no clear difference is observed in the results. At this stage of testing, in most specimens with C15, corrosion stains were observed on the surface at cracks. Slightly lower $V_{corr}$ was observed in the specimens with double steel bars than with single bar. $V_{corr}$ values in mortar specimens also show a similar range as in R/SHCC, despite containing fewer cracks.

Wiggles in the corrosion rate reading is evident from Fig. 8. It is ascribed to sensitivity to variations in amongst other factors, temperature, humidity and moisture content of the specimens. However, upon integration according to eq. (7), the corrosion depth ($d_c$) values, shown in Figure 9, are relatively smooth, allowing improved interpretation of steel bar corrosion in the various specimens. Note that in this research work, corrosion rate readings were started after about 320 and 480 days from the start of accelerated corrosion testing in R/mortar and R/SHCC specimens respectively, once the coulstatic apparatus was developed. Also note that the corrosion at the start of corrosion rate measurement $d_{c0}$ was set to zero. From Fig. 9 it is evident that significantly higher corrosion rate occurs in the C15 specimens, in agreement with observations of corrosion stains on only these specimens.

Further analysis of corrosion data is depicted in Figs. 10 and 11. No clear correlation is found in Fig. 10 between uniform corrosion depth and total crack width in the specimens. Figure 11 appears to indicate a threshold cover depth of between 15 and 25 mm, beyond which uniform corrosion depth does not significantly affected by the cover depth for the specimens tested here. It should be noted that the analysis unfairly compares
R/SHCC specimen corrosion depth development with that in R/mortar specimens. For the former, corrosion rate measurements commenced after about 480 days of accelerated corrosion, while for the latter, it started already after about 320 days. Thus a larger $d_{c0}$ is expected for R/SHCC than for R/mortar.

4.4. Verification of corrosion activities in the R/SHCC specimens

To verify corrosion potential and corrosion rate measurements, three specimens were destructed at relatively early days (17 and 52 days respectively) for visual observation of the corrosion status of reinforcing steel bars. The first, a CS-SHCC C15B1 specimen, destructed after 17 days, and after 52 days two specimens (FS-SHCC C35B1 and FS-SHCC C15B2) were destructed and the rebar corrosion status inspected visually. In all three these specimens, the only indication of corrosion activities was discolouration, with no sign of pitting or steel volume reduction. The positions of discolouration coincided with concentrations of cracks. After longer accelerated corrosion, more rebars were removed from specimens for inspection, i.e. after 560 days two specimens (FS-SHCC C25B1 and CS-SHCC C25B1), after 580 days three specimens (FS-SHCC C15B1, CS-SHCC C35B1 and FS-Mortar C15B1) and after 610 days two specimens (FS-SHCC C25B2 and CS-SHCC C25B2). In these specimens, evidence of corrosion was found in the form of pitting.

Figure 12 shows a closer look at the corrosion in the R/FS-SHCC C15B1 steel bar. Figure 12a shows the bending cracks, drawn in pencil on the specimen for clarity. CT scans of this specimen is shown in Figs. 12B,
noting that a corrosion stain was observed exactly at the encircled crack position. Figure 12d shows the pitting corrosion in the same location.

Figure 13 shows the corroded steel bar after cleaning with the HCl acid. It is interesting to see that after cleaning the steel bar, what appears to be comprehensive, spread corrosion, turns out to be mere discolouration. Once cleaned, steel loss distributed over a significant rebar length in the case of the R/Mortar in Fig. 13a, but more localised in the case of R/SHCC specimens. This gives the perspective that the assumption of uniform diameter loss as expressed in eq. (7) may underestimate the loss in diameter in localised regions on rebars.

4.5. Pitting depth and loss of steel resistance due to corrosion

Pitting depths in the steel bars removed from beam specimens are summarised in Table 4 in terms of average and maximum pitting depth ($d_p$). The rebars were subsequently subjected to tensile testing in a Zwick Z250 Materials Testing Machine. As reference, four undamaged Y10 bars were tested, to determine the average, reference Y10 rebar yield resistance. The yield resistance of the corroded bars is tabulated as well as the % change relative to the reference, undamaged value.

Although mortar specimens show a lowest maximum measured ($d_p$) value, it must be kept in mind that the corrosion is spread over a larger area as seen in Fig. 13a. In all SHCC specimens more localized corrosion is observed. No clear correlation exists between the maximum pitting depth and loss in rebar yield resistance, while Fig. 14 indicates a reasonable correlation with average pitting depth.

From the corrosion depths calculated by eq. (7) in Fig. 9, the yield force was calculated from the assumption of retained 510 MPa yield stress, and the residual cross section. These values are compared with the actual measured yield forces in Fig. 15, clearly showing a

<table>
<thead>
<tr>
<th>Type</th>
<th>Ave $d_p$ (mm)</th>
<th>Max $d_p$ (mm)</th>
<th>$F_y$ (kN)</th>
<th>$\Delta F_y$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Un-Corr steel bar*</td>
<td>0</td>
<td>0</td>
<td>39.19</td>
<td>0</td>
</tr>
<tr>
<td>FS-SHCC C15B1</td>
<td>0.24</td>
<td>0.59</td>
<td>36.25</td>
<td>7.5</td>
</tr>
<tr>
<td>FS-SHCC C25B1</td>
<td>0.21</td>
<td>0.30</td>
<td>36.20</td>
<td>7.6</td>
</tr>
<tr>
<td>CS-SHCC C25B1</td>
<td>0.29</td>
<td>0.32</td>
<td>34.00</td>
<td>13.2</td>
</tr>
<tr>
<td>CS-SHCC C35B1</td>
<td>0.30</td>
<td>0.50</td>
<td>35.42</td>
<td>9.6</td>
</tr>
<tr>
<td>FS-Mortar C15B1</td>
<td>0.23</td>
<td>0.28</td>
<td>36.75</td>
<td>6.2</td>
</tr>
</tbody>
</table>

* A total of 4 un-corroded steel bars were tested as reference.
significant discrepancy due to the assumption of uniformly smeared damage as opposed to the localised corrosion. In addition to the localised vs smeared nature of corrosion damage, the negligence of corrosion depth development up to the time of commencement with corrosion rate measurements ($d_{c0}$ in eq. 7), i.e. after 320 and 480 or more days of cyclic chloride solution exposure respectively for mortar and SHCC specimens, contributes to the discrepancy. Although the C15 mm SHCC specimens clearly show the highest corrosion rates and visual damage (see Figs. 9, 13), however, in this limited data set of corroded rebar tensile tests, they do not lead to the largest drop in resistance.

4.6. Chloride (Cl) profile in SHCC

4.6.1. Cl penetration in un-reinforced SHCC

Table 5 summarises, and Fig. 16 shows the chloride penetration by capillary suction through the cracked, unreinforced SHCC specimens at different time periods. It is clear that high penetration depths occur at the location of cracks, and increase in time. It is also interesting to see that the penetration widens, i.e. expands horizontally along the crack height with time. So, a crack width range of between 50 to 100 µm provides a quick path for chloride penetration into the SHCC. Another study by the authors found that SHCC with crack widths below 50 µm also allow quick penetration of chloride and a maximum chloride penetration depth of 14.5 mm from the exposed surface was found in the uncracked region after 90 days of accelerated drying and wetting exposure (Paul and van Zijl 2013a).

4.6.2. Cl profile in cracked R/SHCC specimens

The Cl profiles determined by XRF for cracked R/SHCC and R/Mortar specimens are shown in Fig. 17. These profiles have been obtained from the specimens as described in section 3.3. The specimens exhibited different corrosion rates before they were used for chloride profiling. The profiles are clearly different from those usually obtained from un-cracked NC specimens (Luping and Nilsson 1992; Spiesz 2013). Recall that test samples were taken by drilling in the cracked region. The 16 mm drill diameter exceeds the crack spacing, whereby more than one crack is represented by the chloride profiles. It is postulated that the geometry of the cracks, which may vary with depth contributes to local high internal Cl concentrations (% by wt of binder) seen in Fig. 17. From the figure it is clear that a significant amount of chloride penetrates the cracked R/SHCC and R/mortar specimens right up to and beyond the steel bar. By considering the rebar depths, the chloride content at the lower surface of the respective rebars vary in the range 0.30% - 0.59% for C15, 0.22% - 0.58% for C25 and 0.17% - 0.40% for C35 but with no correlation between cover depth and chloride content at the rebar surface. The XRF data reflects the total chloride content, while free chloride remains to be determined in future work through chemical analysis.

5. Conclusions and Recommendations

This paper presents results of chloride penetration and chloride-induced corrosion in R/SHCC and R/Mortar specimens. Cyclic wetting and drying accelerated chloride exposure was continued for periods of 620 days in the case of pre-cracked reinforced SHCC beams, and 458 days in the case of R/mortar beam specimens of the same cement-based composite. The following conclusions can be drawn on corrosion potential and rate:

- The half cell potential readings indicate a higher probability of corrosion for smaller cover depth. Specimens with 15 mm cover show higher corrosion potential and rate than with covers of 25 and 35 mm.
- Corrosion rates measurements for cracked R/SHCC

Table 5 Chloride penetration depth in SHCC at different duration of exposure.

<table>
<thead>
<tr>
<th>Type</th>
<th>Duration of exposure</th>
<th>Penetration depth (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>In uncracked region</td>
</tr>
<tr>
<td>FS-SHCC</td>
<td>1 hour</td>
<td>4.5</td>
</tr>
<tr>
<td></td>
<td>3 hours</td>
<td>5.0</td>
</tr>
<tr>
<td></td>
<td>6 hours</td>
<td>8.0</td>
</tr>
<tr>
<td>CS-SHCC</td>
<td>6 hours</td>
<td>5.5</td>
</tr>
</tbody>
</table>
and R/mortar specimens are variable in time, most likely due to variations in moisture content and temperature. However, when integrated to obtain corrosion depth, smooth corrosion curves are obtained which clearly indicate higher corrosion rate for steel with a cover of 15 mm than for 25 and 35 mm. It appears that a threshold cover depth between 15-25 mm exists for the materials tested here, beyond which cover no significant reduction in corrosion rate is observed.

- Removal of the rebars from specimens after extended periods of accelerated corrosion confirmed corrosion damage to rebars from R/mortar as well as R/SHCC specimens. From the limited data set, distributed corrosion damage was found in R/mortar specimens, as opposed to localised damage in R/SHCC specimens.
- A correlation was found between average pitting corrosion depth and loss of yield resistance force of rebars. However, the data set was small and should be extended for meaningful conclusions to be drawn.
- The localised nature of corrosion damage a R/SHCC specimens complicates corrosion damage prediction from corrosion rate measurements. Although more significant corrosion pitting damage could be observed by visual inspection of the bars with 15 mm cover than for rebars with 25 and 35 mm covers, which agrees with the higher measured corrosion rates for these C15 specimens, the loss in rebar yield resistance was not the highest for the C15 rebars.
- After 620 days of accelerated chloride-induced corrosion, relatively little corrosion damage was found in the R/SHCC and R/mortar specimens. Average and maximum pitting depths in the ranges 0.2 – 0.3 mm and 0.3 – 0.6 mm respectively were found, and a maximum loss of tensile yield resistance of 13.2%.
- Significant amounts of chloride penetrate to the steel bar surface in R/SHCC and R/mortar specimens. From a limited study, no correlation could be found between the total amount of chloride at the rebar surface and the total or average crack width, number of cracks, or cover depth.
- Pre-cracked R/mortar specimens have fewer cracks than their R/SHCC counterparts, more distributed corrosion damage, but pitting depths towards the lower limits of the above ranges. This remains to be investigated thoroughly to characterise the corrosion rates, damage patterns and associated loss in resistance. It must be kept in mind that the mortar used here had the same composition as the SHCC except for the fibres. It remains to be established in ongoing research what the significance and role are of chloride binding in these cement-based matrices.

References


