Effect of Exposure on Post-crack Performance of FRC for Tunnel Segments

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Topic: Planning and Designing Tunnels and Underground Structures

Keywords: fibre reinforced concrete, corrosion, embrittlement, design life, durability

1. Introduction

Fibre Reinforced Concrete (FRC) intended for tunnel segments is subject to many requirements during production, handling, installation, and service. Fibres are typically incorporated into this material in order to provide post-crack performance should stresses exceed the flexural-tensile capacity of the concrete matrix. The characteristic post-crack performance for a particular mix will change with time, starting with low levels of performance soon after casting, and evolving to exhibit higher residual strengths as the concrete ages. However, some types of fibres may exhibit a loss of performance as the concrete ages, especially at larger crack widths associated with Ultimate Limit State performance requirements. This phenomenon is widely known as embrittlement due to the reduction in ductility of the FRC composite associated with changes in fibre behavior during pull-out. Steel fibres may also lose performance as a result of corrosion at cracks during service, a process known to be affected by the maximum in-service crack width. The present study has examined how embrittlement and corrosion may combine to degrade the performance of steel FRC over time, and compares this age-dependent change in performance with that exhibited by a macro-synthetic FRC. The study has been undertaken to provide guidance on acceptable maximum crack widths in service for these two types of FRC.

2. Background

Tunnel segments have for many years been reinforced with steel bars, but are now more commonly reinforced with fibres. In some cases, a combination of fibres and bars is used, primarily to control in-service crack widths and other serviceability issues. When steel reinforcement is used, designers are required to specify maximum acceptable crack widths during the service life of the segments in order to limit the potential for corrosion. Ordinarily, maximum acceptable crack widths are determined by the severity of exposure that the structure is likely to be subject to while in service. Unfortunately, the Model Code 2010 [1] and earlier documents such as TC162 [2] provide little guidance on maximum acceptable crack widths for steel FRC in underground environments. Maximum limits of 0.30 mm are prescribed for indoor environments, but no specific limits are provided for more severe environments. Guidance on maximum acceptable crack widths for macro-synthetic FRC are non-existent, but evidence from several published papers, and the recognized endurance of polymers in most corrosive environments (including the alkaline environment within concrete), indicate that macro-synthetic FRC is completely immune to the effects of corrosion and therefore has no maximum allowable in-service crack width. Crack width limits in tunnel linings reinforced with macro-synthetic fibres are therefore determined by other requirements, such as limits on water penetration, or maximum acceptable crack widths for steel bars if these are used in conjunction with macro-synthetic fibres.

The lack of adequate guidance on maximum acceptable crack widths for FRC used in tunnel segments in underground environments has been a constraint on the effective and appropriate use of FRC. The present study was therefore initiated primarily to identify maximum acceptable crack widths with greater certainty than ‘less than 0.30 mm’ as intimated in the Model Code 2010. Since maximum acceptable crack widths for steel bars are known to be more stringent in underground
environments than indoor environments, it has been suspected that limits must be less than 0.30 mm. Nordström identified a maximum acceptable crack width of 0.10 mm for steel Fibre Reinforced Shotcrete (FRS) [3, 4] based on exposure tests undertaken in road tunnels in Stockholm, thereby providing some indication of the sensitivity of steel fibres to corrosion at cracks. However, only limited studies have been performed on macro-synthetic FRC and none has specifically examined whether maximum acceptable in-service crack widths are comparable to that for steel FRC or are greater.

In addition to the effects of corrosion, FRC tunnel segments are also subject to changes in performance due to evolving concrete characteristics as it ages. Unlike reinforcing bars, which exhibit performance across cracks in tension that is largely independent of the characteristics of the concrete, the tensile performance of fibres is highly dependent on the characteristics of the concrete. It is for this reason that flexural performance testing of FRC is routinely included in Quality Assurance programs for major projects like tunnel linings. The post-crack flexural-tensile performance of FRC used in tunnel segments is normally required to satisfy minimum limits prescribed by designers of a tunnel so that the material will perform satisfactorily under SLS and ULS conditions. While these tests are normally performed at 28 days age, the characteristics of the concrete, especially the compressive strength, elastic modulus and hardness of the cement paste, continue to change with age [5, 6]. Previous studies have identified the possibility that changes may occur in the post-crack performance of some types of FRC as aging progresses, with the mode of post-crack fibre behavior changing from an energy-intensive pull-out mode to a more brittle rupture mode. This is manifested as a fall in post-crack performance with age, especially at 2.5 mm maximum crack width or more in flexural tests. Embrittlement therefore primarily affects ULS considerations such as resistance to seismic events or excavations near a tunnel at later ages.

In the present study, the combined effect of embrittlement and corrosion on the post-crack performance of FRC has been examined in such a way as to differentiate the effects of these two concurrently occurring phenomena. This will assist designers to specify appropriate maximum crack width limits in a rational and appropriate manner. However, prior to describing the details of the current investigation, a review of published material on the subjects of age-dependent performance changes and corrosion of fibre at cracks will be undertaken.

2.1 Previous Studies of Aging Effects in FRC

Given the widespread emphasis on a 100-year design life for underground structures such as tunnels, it is surprising that only limited research has been undertaken to ascertain the long-term post-crack performance of FRC both in the cracked and uncracked states in conditions typical of an underground environment. Published research on the effect of aging on the post-crack performance of FRS and FRC has emerged only in recent years, the majority of which have involved field exposure trials undertaken in association with road authorities. One of the first studies to identify changes in post-crack performance of FRC specifically related to aging in infrastructure was a two-year study of the age-dependent behaviour of FRS on the M5 motorway tunnel in Sydney, Australia [7]. This revealed a loss of toughness at late age for shotcrete reinforced with some types of steel fibre. This change in behavior with aging appeared to be caused by the development of high strength and hardness in the enveloping concrete matrix resulting in a change from an energy-intensive pull-out mode of post-crack behaviour to a low-energy rupture mode of fibre failure. This change in failure mode led to a fall in post-crack performance which was unrelated to corrosion or any mechanism of deterioration in the concrete matrix.

Age-dependent changes in post-crack performance at early ages were investigated by Lange & Lee [8] who examined energy absorption in FRC between one and 28 days at narrow crack widths and found differences in the pattern of performance development for different types of fibre. A further investigation involving tests out to four years age was undertaken by Sustersic et al [9] who observed that post-crack performance of SFRC across very narrow cracks increased steadily with aging. Investigations of the post-crack performance of FRS were also reported by Bernard [10] who noted the early onset of performance loss for some FRS mixes reinforced with steel fibres. Recent work by Bjontegaard et al [11] has demonstrated performance losses for steel FRS over a period of three years in Norwegian road tunnels, while Kaufmann [12] observed similar losses for
steel FRS over one year in Switzerland. Bernard [13] conducted tests on FRC beams using high strength concrete mixes typical of tunnel segments and noted falls in post-crack performance for a hooked-end steel FRC but a steady increase in post-crack performance with age for a macro-synthetic fibre. The majority of these investigations were performed on specimens that were uncracked during prolonged curing, and thus corrosion could be discounted as a variable influencing performance.

2.2 Bond and Anchorage Strength

One of the common characteristics of the work described above is the emphasis on field exposure of specimens with age-dependent performance change assessed using standard post-crack performance tests such as beams or panels. This approach has demonstrated changes with aging but has not addressed the mechanics of why the performance of the FRC changes with age. However, observations of crack faces after testing have generally revealed an increase in the proportion of ruptured fibres rather than pulled-out fibres as aging has progressed. This suggests that bond between the steel fibres and the concrete matrix increases with age, though field trials have not provided direct proof of this. To understand why SFRC exhibits this form of performance change with aging, it is instructive to examine the literature on the subject of bond between steel and concrete and the effects of matrix strength and hardness. A substantial amount of work has been undertaken addressing the issue of bond strength between steel reinforcement and concrete and its effect on anchorage and pull-out resistance. Much of this work was initially focused on conventional smooth or deformed steel rebar, and pre-stressed strand, but more recently several investigations have specifically addressed factors influencing bond strength between cement paste and steel fibres. In contrast, very little published research has specifically examined factors influencing the bond strength between macro-synthetic fibres and cement paste.

The literature on bond and anchorage of smooth and deformed rebar in concrete is extensive [eg. 14-16]. However, not many of these studies have specifically examined the influence of concrete strength development with aging on bond development. Among the few that have, Chapman and Shah [17] found that bond strength for deformed bars in early-age concrete (up to 28 days) was highly dependent on the strength of the concrete. Lorrain and Maurel [18] conducted tests on deformed bar in normal and high strength concrete (up to 100 MPa) and found that bond strength was proportional to the strength of the concrete matrix, but all tests were performed at the same age. Gjov et al [19] observed that bond strength increased with compressive strength of the concrete matrix for deformed and smooth bars, especially when silica fume was included in a mix. This appeared to be associated with an increase in the density of the cement paste in the vicinity of the bars. Work on pre-stressing strand has come to similar conclusions, although there is no definitive conclusion as to whether the compressive strength or density/hardness of the cement paste has the greater influence on bond. Laldji and Young [20] found that bond strength between strand and grout increased with cement paste strength in ducts. Similar results were obtained by Mo and Chan [21]. Gustavson [22] conducted pull-out tests on three-wire pre-stressing strand and concluded that the density of the grout in proximity to the strand (which can be related to the strength of the matrix) has the greater effect on the magnitude of bond strength.

Research into the effect of concrete strength on the pull-out resistance of steel fibres has extended over several years. Pinchin and Tabor [23] examined bond strength between steel fibres and cement paste and found that friction (and the associated resistance to post-crack pull-out) increased in proportion to the hardness of the paste adjacent to the surface of the fibre. It has been noted in several studies [5-6] that paste hardness increases with age, and can also be increased by reducing the water/binder ratio, or by adding pozzolans such as silica fume to a mix. Banthia and Trottier [24] observed that steel fibre performance in higher strength concrete may be curtailed by the occurrence of rupture (as a result of excessive bond strength development), rather than the more energy-intensive pull-out mode of post-crack behavior, leading to a reduction in ductility.

Shapiro [25] found that bond strength for steel fibres increased with compressive strength of the paste but did not change appreciably with the yield strength of the steel comprising the fibres. Naaman and Najm [26] noted an increase in pull-out resistance for hooked-end steel fibres as the matrix strength increased. Naaman and Al-Khairi [27] also noted changes in the shape of load-
deflection curves for hooked-end steel fibres as matrix strength increased, with performance at smaller crack widths increasing more than at larger crack widths. Like many others, Wille and Naaman [28] observed that bond strength increases for hooked-end and other deformed steel fibres as matrix strength is increased from normal to Ultra High Strength (120+ MPa) and observed that a higher yield strength is required for the steel comprising the fibre if rupture is to be avoided. Hamoush et al [29] noted that deformations such as hooked ends significantly increase post-crack fracture energy in single-fibre pull-out tests, but that fracture energy is substantially reduced when rupture of the fibre occurs compared to a non-rupturing fibre. Rupture can be avoided by maintaining a sufficiently high tensile yield strength in the steel to ensure post-crack ductility. Wille [30] conducted an extensive investigation of the pull-out resistance of hooked-end steel fibres in concrete matrices ranging from 28 to 192 MPa, and found that increased post-crack performance can only be achieved if fibre yield strength is appropriately matched to the matrix strength. However, like most of the previously cited investigations [23-29], these tests were undertaken at relatively young ages (typically 28 days) and the effect of increasing paste strength and hardness with age was not specifically addressed.

The general consensus of results obtained through bond strength research on smooth and deformed bars, pre-stressing strand, and steel fibres, supports the argument that bond (whether chemical or mechanical in nature) generally increases with increasing matrix strength. This increase is probably related to the increasing density and hardness of the cementitious paste. Paste hardness is known to be influenced by the presence of certain pozzolanic materials, and increases with age for a considerable period of time as the paste matures [6]. Considered together, these observations support the hypothesis that bond strength is not purely the product of matrix strength but will increase with aging of the concrete even when strength gain has plateaued. Observations of FRC performance at 28 days will therefore provide only a ‘snap shot’ of the post-crack performance evolution for a FRC mix. Since rupture of steel fibres is a real and frequently observed phenomenon in field tests at mid- to late ages (beyond 91 days), satisfactory performance at 28 days is demonstrably not a guarantee of similar performance at later ages.

2.3 Corrosion at Cracks in FRC

The present study has examined the combined effect of corrosion and embrittlement on the long-term performance of FRC. To understand how corrosion can affect long-term post-crack performance quite independently of embrittlement, it is useful to examine the body of published literature on the effects of weathering and exposure on the durability of FRC. Due to the much longer history of steel fibre use, and their inherent susceptibility to corrosion, research on corrosion of steel FRC in the field is much more extensive than durability of macro-synthetic FRC.

The ability of steel fibres within uncracked concrete to resist corrosion under conditions of normal atmospheric exposure has been demonstrated through several long-term exposure trials [31-33]. While carbonation may promote corrosion and loss of structural performance, including ductility, for near-surface steel fibres [33], any steel fibres that corrode due to proximity to a concrete surface have been shown to exert insufficient expansive pressure to disrupt the enveloping concrete [34, 35]. Localized surface corrosion therefore does not develop into structurally-threatening through-corrosion of the kind that is commonly observed in conventionally-reinforced concrete [36]. In contrast to the relatively good durability of uncracked FRC, the presence of cracks is recognized as possibly leading to rapid degradation in the performance of FRC when reinforced with steel fibres. Most laboratory and field tests have shown that exposure of cracked steel FRC surfaces to aggressive environments can degrade post-crack performance [34, 37, 38]. The maximum width of cracks is believed to control the rate at which corrosion will progress. However, the degree to which corrosion may be controlled by limiting crack widths has been obscured by the sharply varying test results obtained in some laboratory-based studies of corrosion and crack width. Many of these studies have involved static environments in which corrosion has apparently been controlled by diffusion of oxygen and aggressive agents to the sites of corrosion, rather than more realistic conditions of exposure involving intermittent flushing of water or transport of water under pressure through a crack as will typically occur in underground structures. The relevance of some laboratory-based test results involving static exposure conditions to design crack width limits for underground structures is therefore questionable.
Among field-based corrosion studies conducted to date, Nordström [3, 4] performed tests in an actual road tunnel environment subject to splashing water from traffic and found that the rate of corrosion of steel FRS increased with crack width but that late-age hydration may have the effect of overcoming some of the deterioration in performance at small crack widths caused by corrosion of fibres. Nordström also noted that negligible corrosion and deterioration occurred for steel FRS exhibiting narrow (<0.1 mm) cracks. Kaufmann [12] conducted more recent tests on pre-cracked steel and macro-synthetic FRS panels subject to intermittent flushing of water and found rapid performance losses for steel FRS over periods of less than one year. In contrast, parallel tests on macro-synthetic FRS revealed only minor performance losses. The effect of cracking on the performance of macro-synthetic fibres is in general not as well documented as for steel fibres, but the complete absence of corrosion and immunity to salt ingress characteristic of these fibres has been noted by several researchers [38-40].

The publication of TC 162 [1] and the more recent Model Code 2010 [2], has provided, at best, vague guidance on maximum acceptable crack widths for steel FRC. For indoor environments, maximum acceptable crack widths in service are listed as 0.30 mm for steel FRC, but no maximum values are provided for underground and other severe exposure applications. Maximum acceptable crack widths for macro-synthetic or other types of fibres have not been included. Limited published evidence from the underground mining industry [41] indicates that very wide cracks (of 5 mm and more) are acceptable for macro-synthetic FRS due to the absence of corrosion even in the most severe exposure conditions encountered underground. This corroborates the author’s professional experience in mines located under salt lakes in which extremely saline high pressure ground water (for example, at the Mariner Mine in Western Australia) was found to have no effect on the performance of macro-synthetic fibres in shotcrete while at the same time causing rapid corrosion and breakdown of all ferrous materials associated with ground control and underground operations.

3. Experimental Program

The present lack of clear guidance on acceptable maximum crack widths for FRC in underground environments has driven the design of the current research project. Sydney Water owns a large amount of underground and near-surface concrete infra-structure for which exposure conditions vary significantly and for which acceptable in-service crack width limits are unclear. They were interested in quantifying maximum crack width limits for FRC used in underground structures, and provided a facility at the North Head Sewage Treatment Plant (STP) for exposure of pre-cracked specimens. The site consisted of flat ground exposed to coastal weather. The results were intended to be used in a variety of structural conditions, including for underground tunnel linings subject to pressurized groundwater. However, modelling water ingress through cracks under a hydrostatic head is difficult experimentally, so crevice corrosion (brought about by flushing water into cracks rather than through the cracks) was adopted as a substitute means of exposure. This was similar to the procedure used by Kaufmann [12] for FRS panels tested in Switzerland, but intermittent rain was presently the only means by which water was flushed into the cracks. ASTM C1609/C1609M beam specimens were used as the basis of performance assessment both for the pre-cracked specimens exposed at the North Head STP and for uncracked controls kept in water tanks at the laboratory.

The post-crack performance of cast FRC has for many years been evaluated using the ASTM C1609/C1609M beam test method [42] as these un-notched flexural specimens provide a realistic means of assessing the flexural performance of thick FRC members in the field. ASTM C1609/C1609M beam tests involve three-point loading of a rectangular prism using a servo-controlled machine that is used to impose load such that the central deflection increases monotonically at a prescribed constant rate. Central deflection is measured using two side-mounted LVDTs (Figure 1a). The supporting rollers must be free to rotate with a minimum of friction to preclude errors in post-crack residual strength measurements. In the present tests, the rollers had a constant effective coefficient of friction of 10% [43]. Tests were continued up to a maximum central deflection of 4 mm, but performance was only assessed to a maximum central deflection of 3 mm (which is equivalent to a maximum crack width of 4 mm for a centrally located crack) as shown in Figure 1b.
The investigation took the form of casting trials involving the production of two sets of ASTM C1609/C1609M beams. The first trial involved macro-synthetic FRC incorporating 8 kg/m$^3$ of Barchip BC54 fibres, while the second trial involved steel FRC incorporating 40 kg/m$^3$ of Dramix RC65/60BN fibres (Table 1). The mix designs used for each are listed in Table 2. The mix design selected for the steel FRC included slightly more GGBFS in order to provide additional protection against chloride ion ingress and carbonation. This was not required for the macro-synthetic FRC mix resulting in a lower durability-related cementitious content requirement. Despite this, the late-age strength gain for mix B was considerably greater than for mix D.

About half the specimens in each set were kept as uncracked controls to assess the evolution of performance with age; these were tested at ages of 7, 14, 28, 56, 91, 180, and 365 days, and then 2 years and 3 years. The remaining specimens were pre-cracked at age 56 days and then subjected to either six months, 1 year, 2 years, or 3 years exposure at the North Head STP site before retrieval and testing for comparison with the uncracked controls. In this way the effects of embrittlement and corrosion on post-crack performance could be differentiated. The pre-cracked specimens were deflected in the initial test at 56 days to induce a nominal maximum crack width of either 0.10, 0.20, or 0.30 mm. Crack widths were measured prior to exposure and upon retrieval from site and were found to be stable over the exposure period. Seven beams were tested at each age as controls and 21 pre-cracked beams (7 for each nominal maximum crack width) were tested at each exposure age. A total of 290 beams were tested in this investigation.

**Table 1.** Specimen sets tested in the investigation.

<table>
<thead>
<tr>
<th>Set</th>
<th>Fibre</th>
<th>Dosage (kg/m$^3$)</th>
<th>Mixture</th>
<th>UCS (MPa) 28 days</th>
<th>UCS (MPa) 3 years</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>Barchip BC54</td>
<td>8</td>
<td>A</td>
<td>44</td>
<td>60</td>
</tr>
<tr>
<td>D</td>
<td>Dramix RC65/60BN</td>
<td>40</td>
<td>B</td>
<td>65</td>
<td>72</td>
</tr>
</tbody>
</table>

The Barchip BC54 fibre was selected because it has been used in tunnel segments for several projects internationally [44, 45], while the Dramix RC65/60BN fibre was selected because it has been widely used in tunnel segments. The concrete was produced at a local ready-mixed batching plant and fibres were added only after the agitator truck had arrived at the laboratory. The concrete was mixed vigorously for about 20 minutes after addition of fibres to ensure they were well distributed throughout the load before casting of the specimens commenced. Cylinders measuring $\varnothing 100 \times 200$ mm were cast as companions to the beams for testing at each of the required ages. All specimens were cured under plastic for 2 days before being stripped and cured in lime-saturated water at 23°C until the required age of testing. All beams were tested in a surface-moist condition.
4. Results

Each ASTM C1609/C1609M beam test resulted in a load-deflection curve revealing performance up to 4 mm central deflection. Performance was summarized in terms of post-crack residual strengths at 0.75 and 3.0 mm central deflection. An example of a load-deflection curve for initially uncracked beams tested at 28 days, and beams pre-cracked to 0.3 mm crack width and subject to re-loading after 3 years exposure in the field, are shown in Figure 2a for the Barchip BC54 macro-synthetic FRC, and in Figure 2b for the Dramix RC65/60BN steel FRC. It is clear that the macro-synthetic FRC exhibited a slight increase in post-crack performance at 3 years compared to 28 days, while the steel FRC exhibited greater variability in performance and a substantial fall in post-crack performance with age and exposure, especially at larger deformations.

4.1 Effect of Corrosion on Performance of Beams

Post-crack performance for the initially uncracked control specimens, and the pre-cracked beams subject to field exposure, are summarized in Figure 3 for the macro-synthetic FRC and in Figure 4 for the steel FRC. These graphs show the mean performance of each set of seven uncracked control beams tested at between 7 days and 3 years (with error bars at one standard deviation), and superimposes the mean performance of each set of seven pre-cracked and exposed beams tested at either 6 months, 1 year, 2 years, or 3 years. The variation in unloading deflection apparent for some of the FRC beams (as shown in Figure 2a) was caused by differences in the location of the crack between the third points which led to differences in the point of unloading required to achieve the same crack width.

Table 2. Mix designs.

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Mixture (kg/m³)</th>
<th>A</th>
<th>B</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>330</td>
<td>330</td>
<td></td>
</tr>
<tr>
<td>Fly Ash</td>
<td>114</td>
<td>114</td>
<td></td>
</tr>
<tr>
<td>GGBFS</td>
<td>106</td>
<td>142</td>
<td></td>
</tr>
<tr>
<td>20 mm</td>
<td>640</td>
<td>630</td>
<td></td>
</tr>
<tr>
<td>10 mm</td>
<td>360</td>
<td>376</td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>568</td>
<td>580</td>
<td></td>
</tr>
<tr>
<td>Water Reducer (mL)</td>
<td>1645</td>
<td>1150</td>
<td></td>
</tr>
<tr>
<td>Slump (mm)</td>
<td>120</td>
<td>140</td>
<td></td>
</tr>
</tbody>
</table>

Residual strength at 0.75 mm deflection is plotted in the left hand graph, and residual strength at 3.0 mm deflection on the right. Referring to Figure 3a, it is apparent that the 0.75 mm residual strength for the macro-synthetic FRC was somewhat diminished in the pre-cracked beams compared to the uncracked controls. This occurred because the re-load rigidity of these beams was quite low and thus the 0.75 mm deflection corresponded to the ascending part of the re-loading curve for many of the beams (especially those deflected to 0.3 mm initial crack width). In contrast, the residual strength at 3.0 mm deflection in the pre-cracked and field exposed macro-synthetic FRC beams was essentially unchanged compared to the uncracked controls.

Fig 2. Load-deflection curves at 28 days and load-reload curves at 3 years age pre-cracked to 0.3 mm crack width, for ASTM C1609/C1609M beams reinforced with a) 8 kg/m³ Barchip 54, and b) 40 kg/m³ Dramix RC65/60BN.
Fig 3. Residual strength at 0.75 mm and 3 mm as a function of age for pre-cracked and uncracked ASTM C1609/C1609M beams reinforced with 8 kg/m³ Barchip BC54.

Fig 4. Residual strength at 0.75 mm and 3 mm as a function of age for pre-cracked and uncracked ASTM C1609/C1609M beams reinforced with 40 kg/m³ Dramix RC65/60BN.

The summarized post-crack performance of the steel FRC beams is shown in Figure 4. These results revealed that the residual strength at 0.75 mm deflection in the beams pre-cracked to 0.10 mm crack width and exposed at the field site exhibited little change in performance compared to the uncracked controls, but all the other pre-cracked beams exhibited a fall in performance compared to the controls both at 0.75 mm and 3.0 mm deflection. Moreover, the average residual strength at 3.0 mm deflection for the uncracked controls fell substantially compared to the 28 day performance (which normally represents the age at which Quality Assurance testing is performed). This fall in the performance at 3.0 mm was accompanied by a steady rise in the proportion of ruptured as opposed to pulled-out steel fibres at the crack faces. These results closely resemble the pattern of post-crack performance development for uncracked FRC previously published for fibre reinforced shotcrete and concrete [7, 10-13] in long-term aging studies. The fact that the same result has now been obtained in so many independently conducted trials suggests that hooked-end steel fibres exhibit a problem in regard to long-term performance retention.
4.2 Effect of Corrosion on Relative Performance

It is clear from the above that age-dependent changes in performance caused residual strength to change for the uncracked steel FRC controls, while the uncracked macro-synthetic FRC controls exhibited a steady or slight increase in performance with age across the full range of deformation. All of these results were obtained for specimens maintained in curing tanks within the laboratory environment. Beams that had been pre-cracked at 56 days and exposed to weathering at the field site exhibited more dramatic changes in performance with aging. In these specimens, the effects of both embrittlement and potential corrosion of the fibres could be assessed, thereby differentiating the effects of corrosion from embrittlement by comparison back to the uncracked controls.

For each set of beams retrieved from the exposure site at ages of 6 months, 1 year, 2 years, and 3 years, average residual strengths at 0.75 mm and 3.0 mm central deflection were determined and are shown in Figures 3 and 4. Of relevance to corrosion effects is the relative performance of the pre-cracked field specimens compared to the uncracked controls of the same age (see Tables 3 and 4). The relative performance of the pre-cracked field specimens compared to the uncracked
controls of 28 days age indicates the cumulative effect of both embrittlement and corrosion. These relative performance values have been plotted as a function of period of exposure in Figure 5 for the macro-synthetic FRC, and in Figure 6 for the steel FRC. The results indicate that the pre-cracked macro-synthetic FRC exposed in the field exhibited a minor increase in relative performance at 3.0 mm deflection compared to the uncracked controls over the three year exposure period, whereas the pre-cracked steel FRC exposed in the field exhibited a precipitous fall in performance at 3.0 mm deflection compared to the uncracked controls.

The results shown in Figures 5 and 6 indicate that cracks of up to 0.30 mm maximum width can exist in macro-synthetic FRC members without any detrimental effect on post-crack structural performance. This corroborates evidence from civil and marine projects [39, 40] and the underground mining industry [38] that very wide cracks can be tolerated in macro-synthetic FRC due to the absence of corrosion in this composite material. In addition, age-dependent changes in performance lead to a general increase in post-crack performance, at least up to three years age, primarily because the mode of post-crack fibre behaviour does not change with age. It can therefore be conservatively concluded that the post-crack performance of cracked macro-synthetic FRC is insensitive to both age and in-service crack width.

In contrast to the results for the macro-synthetic FRC, the results for the pre-cracked steel FRC beams revealed substantial changes in performance with age and falls in performance for all but the narrowest initial crack widths. In uncracked steel FRC, the residual strength at 0.75 mm tended to increase steadily with age to 91 days and then stabilize, but the residual strength at 3.0 mm reached a peak at 56 days age and then fell significantly. This closely mirrors previously published results for aging trials on steel FRC and shotcrete [7, 10-13]. The fact that a change occurred from the pull-out mode to the more brittle rupture-based mode of post-crack steel fibre behaviour over a prolonged period of time reflects the slow rate at which cement paste hydrates, as shown by Sanahuja et al [6], and the importance of paste hardness to steel-paste bond strength [19, 22, 26-30]. For the pre-cracked specimens exposed in the field, the effects of corrosion were critical for crack widths of 0.20 mm and more (Table 4). Again, this closely mirrors previously published data from field exposure trials [3, 4]. The combined effect of corrosion and age-dependent performance increase led to a relatively small fall in performance at 0.75 mm deflection but a very substantial fall in performance at 3.0 mm deflection relative to the measured 28 day result for crack widths of 0.2 mm and more. From this it can be concluded that the maximum permissible crack width for steel FRC is no more than 0.10 mm. However, it must be noted that the present results extend only to three years field exposure, so extrapolation to a 50-100 year design life is subject to considerable uncertainty.

Table 3. Relative performance of pre-cracked FRC beams reinforced with Barchip BC54 fibres and exposed at North Head STP compared to uncracked controls cured in laboratory.

<table>
<thead>
<tr>
<th>Crack width During Exposure Period (mm)</th>
<th>Age (years)</th>
<th>Relative Performance of Cracked Beams Subject to Field Exposure - Compared to Reference Beams in Laboratory Tested at Same Age</th>
<th>Compared to Reference Beams in Laboratory Tested at 28 Days</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.10</td>
<td>0.5</td>
<td>0.9258 1.0236 1.0078 1.1189</td>
<td>0.75 mm 3.0 mm 0.75 mm 3.0 mm</td>
</tr>
<tr>
<td></td>
<td>1</td>
<td>0.7775 0.8544 0.8931 0.9796</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.8658 0.9420 0.9910 1.0870</td>
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Table 4. Relative performance of pre-cracked FRC beams reinforced with Dramix RC65/60BN steel fibres and exposed at North Head STP compared to uncracked controls cured in laboratory.

<table>
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<th>Crack width During Exposure Period (mm)</th>
<th>Age (years)</th>
<th>Relative Performance of Cracked Beams Subject to Field Exposure - Compared to Reference Beams in Laboratory Tested at Same Age 0.75 mm</th>
<th>Compared to Reference Beams in Laboratory Tested at 28 Days 3.0 mm</th>
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5. Conclusions

An experimental investigation was undertaken into the combined effect of corrosion and embrittlement on the post-crack flexural performance of macro-synthetic FRC and steel FRC. The primary objective of the investigation was to determine maximum acceptable crack widths in macro-synthetic and steel FRC used in aggressive environments typical of underground infrastructure. Laboratory and field-based exposure trials examined the performance between 7 days and 3 years age of uncracked control specimens and field specimens pre-cracked to maximum crack widths between 0.1 and 0.3 mm. These specimens were used to assess the effect, respectively, of embrittlement on its own, as well as a combination of embrittlement and corrosion, on performance out to 3 years age.

The results of the investigation indicate that post-crack performance at 0.75 mm deflection in the steel FRC beams was maintained with age for a crack width of 0.10 mm, and suffered larger falls in performance at 0.2 and 0.3 mm crack width following three years exposure. The performance of the steel FRC at 3.0 mm deflection suffered a substantial fall at crack widths of greater than 0.1 mm. Much of this fall in performance was attributable to embrittlement, and the rest to corrosion of the fibres at the crack. The macro-synthetic FRC exhibited no signs of corrosion and exhibited only minor variations in post-crack performance with aging and exposure. This investigation has confirmed that macro-synthetic FRC suffers no change in performance with aging or exposure to aggressive environments for crack widths of up to 0.30 mm. It has also confirmed that steel FRC will exhibit a substantial fall in performance with aging as a result of embrittlement, and will also exhibit additional falls in post-crack performance when cracks are present with a width in excess of 0.10 mm in aggressive environments typical of underground infrastructure.

References


14. ACI Committee *Report 408*, "Bond and Development of Straight Reinforcing Bars in Tension" American Concrete Institute, Farmington Hills, MI.


