Crack development and deformation behaviour of CFRP-reinforced mortars

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ABSTRACT

This paper reports on a research study investigating CFRP-reinforced mortars in uniaxial tension, as a strengthening material for concrete structures. The bare strengthening material was tested on dogbone specimens. Crack formation, crack development and the interaction between the grid and the mortar phase with varying geometrical parameters and mortar compositions have been investigated and evaluated. The use of engineered cementitious composites, exhibiting multiple cracking and enhanced pseudo-ductility in uniaxial tension, was found to result in an improved overall performance.

Keywords: concrete, strengthening, carbon fibre reinforced polymers (CFRP), mortar, mineral based composites (MBC), strain-hardening cementitious composites (SHCC), tensile tests, strain hardening, cracking, pseudo-ductility
1. INTRODUCTION

Fibre-reinforced polymers (FRP) have become a popular material for strengthening and/or retrofitting of existing concrete structures. Externally epoxy-bonded FRP systems have been proven to be an effective strengthening method in repairing or strengthening structures and there has been a large amount of literature published on the topic; see for example [1-4].

Despite their advantages over traditional strengthening methods, the use of epoxy-bonded systems is not entirely problem-free [2]. The epoxy resin creates sealed surfaces. Furthermore, it has poor thermal compatibility with the concrete substrate, it is sensitive to moisture at the time of application, and it creates a hazardous working environment. In cold climates, the use of epoxy is limited because of the minimum temperature of application (typically, 10°C or 50°F) [2]. Therefore, it is of interest to develop alternative strengthening systems where the epoxy bonding agent can be replaced with cementitious materials, for example a polymer-modified or purely cementitious mortar with similar properties to those of the concrete substrate and applicable in a more environmentally friendly and possibly also cost-effective way.

Mortars can be combined with FRP textiles or 2D grids to form an effective strengthening system. This kind of strengthening system has already been tested by [5, 6] on flexural and shear beams.

In the research presented, conventional, quasi-brittle, and “ductile” binders have been combined with CFRP grids. The quasi-brittle binders used are commercially available, pre-mixed, polymer-modified mortars. The ductile binder is strain-hardening cementitious composite (SHCC), namely, a polyvinyl alcohol-reinforced engineered cementitious composite (PVA-ECC), that exhibits strain hardening along with enhanced tensile ductility. The uniaxial tensile tests aimed at obtaining better understanding of the tensional behaviour of the FRP grid-reinforced cementitious composite material.

Only the bare strengthening material is tested here, not considering the interaction with the concrete structure to be strengthened. The main reasons for replacing the conventional mortar with strain-hardening mortars, in a few specimens, were to 1) enhance the loading capacity, 2) enhance the deformation capacity, and 3) prevent brittle failure in the FRP.

2. RESEARCH SIGNIFICANCE

By substituting the traditionally quasi-brittle mortar with a strain-hardening cementitious mortar in an externally bonded strengthening system, the interaction between the cementitious mortar and the FRP reinforcement can be significantly altered and improved, (potentially) leading to a more effective use of the FRP reinforcement. Such cement-based systems may in certain conditions, replace the conventional epoxy-based systems. It is also shown how the behaviour of a strain-hardening cementitious mortar can improve the mechanical properties of a mineral-based strengthening system.

3. RESEARCH QUESTIONS

The study aimed to answer the following questions:
1. Which is the best material combination/orientation leading to the optimal utilization of the FRP?
2. How do the failure modes and the load- and deformation capacity compare, depending on the mortar type? Can a strain-hardening effect be shown in specimens cast with tensile strain-hardening mortars?
3. Can the brittle and/or premature failure of the FRP grid be prevented by applying a ductile matrix?
4. Prove whether the dogbone test setup is a suitable test method for testing MBC in tension.

4. STRENGTHENING WITH CEMENTITIOUS COMPOSITES

The mechanics and design of different FRP reinforcements together with cementitious bonding agents have been extensively researched. The short overview here is selective to applications, which have led to strengthening with grid-reinforced mortars. A more detailed “state-of-the-art” can be found in [7].

Embedded continuous (dry) fibres, fibre reinforced cementitious mortars, textile-reinforced mortar (TRM) or textile-reinforced concrete (TRC) make use of the tensile strength of the FRP reinforcement which is significantly higher than that of the mortar phase. The FRP component in these applications is intentionally aligned accordingly to the principal stresses expected during the lifespan of the structure. In TRC, the load capacity is heavily dependent on the proper penetration of the textile by the mortar, as emphasized by e.g. [8]. If instead of a textile, the FRP is a grid, it offers the advantage of an improved mechanical anchorage due to the rigid joints of FRP in fine-grain mortars and it ensures that all fibre filaments will work together. The pre-cut grid, available with different grid spacing and thickness, can be embedded between two layers of polymer-modified mortar, resulting in a strengthening layer of about 5-10 mm of total thickness [5].

In a cementitious strengthening system, the bond between base concrete (structure to be strengthened) and first layer of polymeric mortar is enhanced by a primer. Several tests have been carried out with grid-reinforced mortars, mainly focusing on flexural [5, 9], and shear strengthening [5, 10]. These tests have shown that that it is possible to achieve a near-perfect bond between the grid and the cementitious matrix so that the composite material will fail with FRP rupture. However, failure of the grid is often premature because of local stress concentrations.

In the mortar phase, mix designs that introduce fly ash and/or silica fume partly replacing the cement in order to densify the microstructure result in higher bond strength between FRP (textiles) and matrix [11]. Mortars can also contain chopped or milled fibres of different kinds. If such a mortar is used together with an embedded reinforcement, improved bond is expected because of the fibre interlock mechanism.

In recent years, micromechanically designed strain-hardening materials have been developed, in which a tensile stress-strain behaviour analogous to that of metals has been achieved. Strain-hardening cementitious composites (SHCC) are defined by an ultimate strength higher than their first cracking strength and the formation of multiple cracking during the inelastic deformation process [12]. However, the inelastic deformation behaviour of SHCC is based on the sequential development of matrix multiple cracking while undergoing strain-hardening [13]. It is more
accurate to refer to the mechanism as pseudo strain hardening in order to differentiate it from the “real” strain hardening observed in metals, that is based on dislocation micromechanics in the plastic deformation regime.

The most typically used SHCC, the engineered composite (ECC) utilizes short, randomly oriented polymeric fibres (e.g. polyethylene, polyvinyl alcohol) at moderate volume fractions - typically less than 2% [13, 14]. ECC has been used in standalone and strengthening applications where ductility is an important criterion. The pseudo strain-hardening behaviour of ECC has been utilized as a mechanism to redistribute concentrated loading and thus prevent sudden failure at critical structural connections where steel and concrete come into contact, i.e., shear studs, fasteners or joints, where a steel beam meets an RC column in a hybrid structure [15]. The high damage tolerance of ECC is valuable to the performance of a structure in terms of collapse resistance, extension of service life, and minimization of repair after an extreme event [15] or strengthening purposes. When used in combination with (steel) reinforcement, the tensile ductility of the ECC matrix can on a macro scale, eliminate the strain difference between reinforcement and matrix material [13].

5. LOAD-DEFORMATION RESPONSE OF CEMENTITIOUS COMPOSITES

5.1. Tensile response of cementitious materials

Cement-based composites can be conveniently classified according to their tensile response [12]. The authors have compared the tensile behaviour of steel fibre reinforced concrete, textile reinforced concrete (TRC), engineered cementitious composites (ECC), and steel-reinforced ECC more in detail, based on the existing literature. Only a comparative figure is being presented here (Figure 1) with brief explanations. The different stages are named in a way that they mean the same for the different materials, so not all stages exist for all materials.

Quasi-brittle and ECC mortars used in the tests behave identical to the mortar components shown in Figures 1a and 1c, respectively. During the formation of multiple, evenly distributed, closely spaced, small cracks (Figure 1c), the ECC matrix shows an overall strain-hardening behaviour, without definitely distinctive parts in the curve from $\sigma_{cc}$ to $\sigma_{pc}$. After localization, there is a gradually decreasing, softening range due to the fibre pullout mechanism.

ECC works well with regular steel reinforcement [16]. The hardening part of the load-deformation curve of the steel reinforced ECC (R/ECC) is not as uniform as in plain ECC but has a steeper and a gradually raising part, in accordance with the elastic stage/yielding of the steel. In the inelastic deformation regime, where both components are yielding, cracking of the ECC and yielding of the reinforcement are uniformly distributed over the length of the specimen, until rupture of the steel. The hatched area in Figure 1d represents the contribution of the ductile matrix compared to steel.

Finally, in TRC [17] where the FRP is linear elastic up to failure and the mortar (typically) is quasi-brittle, the hardening part can be divided in two distinct parts as shown in Figure 1e. After first crack formation, the load-deformation curve shows a small increase in loading capacity due to the formation of additional transverse cracking. The member is softened by the formation of additional crack(s) and the load increase per deformation increment is decreasing with each crack until the stabilized crack pattern (II/b) is reached which is nearly linear.
Crack widths here are governed by the FRP reinforcement and the bond characteristics to the surrounding concrete matrix, and mainly the FRP determines the stiffness of the member. However, the uncracked segments between the cracks still increase the stiffness of the member, as long as they are not debonded from the FRP reinforcement. Since the FRP reinforcement has no inelastic deformation capacity, failure of a TRC is characterized either by slippage in the fibre tows or by linear elastic deformations until the FRP ruptures in a brittle manner upon reaching its tensile failure strain [17]. In practice, final failure normally is a combination of both fibre slippage and rupture [19].

If the TRC matrix contains short fibres, after first cracking the hardening part becomes uniform, similar to that of the ECC [18].
5.2. Tension stiffening effect

The contribution of a cementitious matrix to the load–deformation response in uniaxial tension is generally described as tension-stiffening effect [20]. The response of the reinforced cementitious composite is compared to that of the bare strengthening material (steel or FRP) and the difference in load capacity is attributed to the tensile load carried by the cementitious matrix between transverse cracks. The matrix contribution is most emphasized in steel reinforced ECC (Figure 1d), but also significant in textile reinforced concretes (Figure 1e).

6. EXPERIMENTAL PROGRAM

6.1. Materials

In the experimental program, two different CFRP grids and three types of mortars were used. Material properties are listed in Table 1 (grids) and Table 2 (mortars).

Table 1 – Material properties of the grids used

<table>
<thead>
<tr>
<th>FRP tested values</th>
<th>Spacing L x T [mm]</th>
<th>E_Lt [GPa]</th>
<th>E_Tt [GPa]</th>
<th>f_Lt [MPa]</th>
<th>f_Tt [MPa]</th>
<th>ε_Lt [%]</th>
<th>ε_Tt [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>small</td>
<td>24 x 25</td>
<td>79</td>
<td>51</td>
<td>988</td>
<td>740</td>
<td>12.5</td>
<td>14.0</td>
</tr>
<tr>
<td>medium</td>
<td>42 x 43</td>
<td>85.4</td>
<td>45</td>
<td>944</td>
<td>500</td>
<td>11.1</td>
<td>11.1</td>
</tr>
</tbody>
</table>

FRP supplier data

<table>
<thead>
<tr>
<th>Spacing L x T [mm]</th>
<th>E_Lm [GPa]</th>
<th>E_Tm [GPa]</th>
<th>f_Lm [MPa]</th>
<th>f_Tm [MPa]</th>
<th>ε_Lm [%]</th>
<th>ε_Tm [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>small</td>
<td>24 x 25</td>
<td>262</td>
<td>289</td>
<td>4300</td>
<td>3950</td>
<td>15.0</td>
</tr>
<tr>
<td>medium</td>
<td>42 x 43</td>
<td>284</td>
<td>253</td>
<td>3800</td>
<td>3800</td>
<td>13.4</td>
</tr>
</tbody>
</table>

Table 2 – Material properties of the mortars used

<table>
<thead>
<tr>
<th>Material</th>
<th>E_c [GPa]</th>
<th>f_cc [MPa]</th>
<th>f_ct [MPa]</th>
<th>w/c</th>
<th>notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1 (from supplier)</td>
<td>26.5</td>
<td>53.2</td>
<td>9</td>
<td>0.16</td>
<td>-</td>
</tr>
<tr>
<td>M2 (from supplier)</td>
<td>35</td>
<td>77</td>
<td>2.8</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>ECC (from earlier tests)</td>
<td>19</td>
<td>60</td>
<td>3</td>
<td>0.88</td>
<td>Fly ash 45 mass %; PVA 0.01 mass %</td>
</tr>
</tbody>
</table>

The utilized grids are the C3000 A X1 grid (referred to as “medium grid”) and the C5500 A X1 grid (“small grid”) from Chomarat, U.S. Both grids are epoxy-coated with a fibre volume percentage of 20-25%. We used two pre-mixed, commercially available mortars; StoCrete GM1 (M1) and StoCrete TS100 (M2), from Sto Scandinavia AB. These mortars are one-component, high-strength, polymer-modified, quasi-brittle mortars with tensile strengths of 53.2 MPa (M1) and 77.0 MPa (M2). M1 also has low fibre reinforcement content. The exact mortar compositions or information on the fibrous component is not provided by the manufacturer.

The used primer (Table 3) is a one-component, cement-based and polymer-reinforced powder mixed with water. It is used as a silt-up product on the roughened (sandblasted) concrete surface. Its function is to enhance the bond in the transition zone.

Table 3 – Material properties and mixing ratio for the primer

<table>
<thead>
<tr>
<th>Primer</th>
<th>Density [kg/m³]</th>
<th>d_max [mm]</th>
<th>Mixing ratio, primer:water</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2020</td>
<td>2</td>
<td>1:0.22</td>
</tr>
</tbody>
</table>
The third mortar tested is an ECC mix (“DTU ECO-3 M9 Melflux”) containing PVA fibres (1% by weight, or 2% by volume) and a large portion (44% by weight) fly ash. Its tensile strength is 60 MPa. The exact mix composition is given in Table 4.

<table>
<thead>
<tr>
<th>Material</th>
<th>Fraction [mass %]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (basic Portland)</td>
<td>22</td>
</tr>
<tr>
<td>Fly ash</td>
<td>44</td>
</tr>
<tr>
<td>Sand (&lt;0.15mm)</td>
<td>8</td>
</tr>
<tr>
<td>Quartz powder 100/22</td>
<td>8</td>
</tr>
<tr>
<td>Water</td>
<td>18</td>
</tr>
<tr>
<td>Superplasticizer (Melflux)</td>
<td>0.02</td>
</tr>
<tr>
<td>PVA fibre (oiled)</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 4 – Mix composition of the ECC used

6.2. Test matrix

With the three different mortars (M1, M2, ECC), two different grids (medium and small) and three chosen grid configurations (0°, 90°, and 15° with respect to the applied tensile force), the final test matrix consists of 13 different combinations, 3 specimens per each series, in total 44 specimens. The first five are dummies (2 x M1 without reinforcement, 2 x M1 with medium grid longitudinal direction, and 1 x ECC medium grid longitudinal direction), followed by the 3x13 reinforced specimens. The tested combinations are summarized in Table 5.
Table 5 – Test matrix

<table>
<thead>
<tr>
<th>Series nr.</th>
<th>Mortar</th>
<th>Grid spacing</th>
<th>Grid config.</th>
<th>Test specimen</th>
<th>No. of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>M1</td>
<td>---</td>
<td>---</td>
<td>Dummy</td>
<td>2</td>
</tr>
<tr>
<td>2</td>
<td>M1</td>
<td>medium</td>
<td>0°</td>
<td>Dummy</td>
<td>2</td>
</tr>
<tr>
<td>3</td>
<td>ECC</td>
<td>medium</td>
<td>0°</td>
<td>Dummy</td>
<td>1</td>
</tr>
<tr>
<td>4</td>
<td>M1</td>
<td>---</td>
<td>---</td>
<td>M1 reference</td>
<td>3</td>
</tr>
<tr>
<td>5</td>
<td>M2</td>
<td>---</td>
<td>---</td>
<td>M2 reference</td>
<td>3</td>
</tr>
<tr>
<td>6</td>
<td>M1</td>
<td>medium</td>
<td>0°</td>
<td>M1-0-M</td>
<td>3</td>
</tr>
<tr>
<td>7</td>
<td>M1</td>
<td>small</td>
<td>0°</td>
<td>M1-0-S</td>
<td>3</td>
</tr>
<tr>
<td>8</td>
<td>ECC</td>
<td>medium</td>
<td>0°</td>
<td>ECC-0-M</td>
<td>3</td>
</tr>
<tr>
<td>9</td>
<td>ECC</td>
<td>small</td>
<td>0°</td>
<td>ECC-0-S</td>
<td>3</td>
</tr>
<tr>
<td>10</td>
<td>M2</td>
<td>medium</td>
<td>0°</td>
<td>M2-0-M</td>
<td>3</td>
</tr>
<tr>
<td>11</td>
<td>M1</td>
<td>medium</td>
<td>15°</td>
<td>M1-15-M</td>
<td>3</td>
</tr>
<tr>
<td>12</td>
<td>M2</td>
<td>medium</td>
<td>15°</td>
<td>M2-15-M</td>
<td>3</td>
</tr>
<tr>
<td>13</td>
<td>M2</td>
<td>small</td>
<td>15°</td>
<td>M2-15-S</td>
<td>3</td>
</tr>
<tr>
<td>14</td>
<td>M1</td>
<td>medium</td>
<td>90°</td>
<td>M1-90-M</td>
<td>3</td>
</tr>
<tr>
<td>15</td>
<td>M1</td>
<td>small</td>
<td>90°</td>
<td>M1-90-S</td>
<td>3</td>
</tr>
<tr>
<td>16</td>
<td>M2</td>
<td>small</td>
<td>90°</td>
<td>M2-90-S</td>
<td>3</td>
</tr>
</tbody>
</table>

6.3. Specimens

Earlier “dogbone” tests with textile reinforced concrete and a large-scale experiment were described by [21], on 900 x 100 x 60 mm specimens with a 10mm thick web. Based on those sizes, but considering that a typical MBC strengthening layer consists of two 10 mm thick layers and a grid, new specimens were designed, as illustrated in Figure 2.

![Figure 2 – Test specimen and reinforcement configuration](image-url)
For brittle or quasi-brittle materials, the uniaxial tests are very sensitive, as an uncontrolled deformation at cracking occurs despite the displacement control. In addition to that, the dogbone geometry is very sensitive to cracks initiating towards the ends of the test field, where the thin web meets the bulk end of the specimen. To prevent cracking in these areas, the optimal solution would be a concave slant surface (a) or a specimen with 2-step slant surface (b) to minimize this risk. Due to resources, simpler dogbone geometry was decided upon (c and d), with a total length of 980 mm, and a web thickness of 20 mm.

The length of the test field was set to the longest possible so that it has a high crack potential; yet short enough to be able to de-mould and handle the specimens without breaking them apart. The final dimensions were set to 160 x 160 x 980 mm in order to meet the geometry of the existing moulds, with a representative test section of 400 x 160 x 20 mm. The CFRP grid was placed in the mid-plane of the specimen (Figure 2b) into the slits at the ends. Figure2e through Figure 2h shows the different reinforcement orientations. The 30 x 30 mm wedged slits at the end of each bulk can be used to glue some additional CFRP material to the reinforcement with epoxy as extra anchorage in the case that bond slippage occurs prematurely. The end slits were wedged, to ease the de-molding process. All specimens were de-molded after 24 hours except for the unreinforced reference specimens and the ECC specimens, which were de-molded after 48 hours. All specimens were water cured for at least 28 days prior to testing.

6.4. Anchor clamp

The uniaxial test requires special devices for loading. The anchor clamp is the fixating mechanism between the test machine and the test specimen. Its main purpose is to hold the specimen fixed and centred, and to transfer the tensile force form the test machine evenly into the specimen. It has joints in both planes parallel to the tensile force, to avoid any shear forces in the specimens.

The clamps were designed based on combining multiple clamp designs detailed in [22], shown in Figure 3. A combination of (b) and (d) has been chosen and custom-welded for this experiment. From (b), the concept of two crossbars, transferring the force through a slant surface, and from (d), the double joints were taken, in order to nullify any shear forces in the web. However, in order to save space, the custom device had both joints working in the same plane.

![Figure 3 – Various tensile clamps for direct tensile testing [22]](image)

Between the crossbars and the specimen, neoprene was used to distribute the forces evenly. The main bulk of the specimen is in compression at all times during testing, which means that the
connection between the clamp and the specimen is less vulnerable to fracture. The clamp is self-centring, since the crossbars are rotate-able and automatically wedge the specimen into the right position. The side plates are also able to rotate, to help the easy mounting and demounting of the specimens. The anchor clamps are shown in Figure 4.

![Anchor clamps](image)

**Figure 4 – Custom-welded self-centring anchor clamps**

### 6.5. Measurements and monitoring

The dogbone tests were run under displacement control at a rate of 0.6 mm/s. The load-deformation behaviour was monitored within the representative test section, and recorded by a data logger connected to the test machine. A digital image correlation system (ARAMIS from GOM Optical Measuring Techniques, Germany) was also used to monitor deformations in the test field. ARAMIS is a non-contact and material independent 3D optical measuring system, which analyzes, calculates, and documents material deformations by means of recording and calculating relative displacements of discrete points on a patterned surface. The ARAMIS measurements were conducted primarily to monitor and analyze crack development and crack widths. In addition, strain overlay photos were taken for illustrating crack development within the test field.

### 7. EXPERIMENTAL RESULTS

#### 7.1. Evaluation and presentation of test data

The specimens were compared by their load-deformation behaviour, first cracking strength, failure load, mean number of cracks developed within the test field and average crack width. The ARAMIS documentation consists of plots illustrating crack development (crack widths, crack density) and elongations within the test field.
7.2. Load-deformation response

The un-strengthened mortar specimens failed in a brittle manner as expected for plain mortar. Average failure loads were 8.7 kN and 8.3 kN for M1 and M2, respectively.

Three quasi-brittle specimens had small cracks on them close to the end of the test field after demolding; these are excluded from the results. For the remaining specimens, the tensile response is plotted in Figures 5-15. Here we did not include the plain mortar specimens to save space (the response was linear elastic). Additionally, part of the curves we plotted against the measured properties of the bare grid on Figures 5-9, and 13 [23], which is the straight line starting at first cracking.
Figure 9 – M2-0-M
Figure 10 – M1-15-M

Figure 11 – M2-15-M
Figure 12 – M2-15-S

Figure 13 – M1-90-M
Figure 14 – M1-90-S
Figure 15 – M2-90-S

Parallel to the plots, Table 6 summarizes first cracking and failure loads, also including the plain mortar specimens.

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>Failure load [kN]</th>
<th>First cracking load [kN]</th>
<th>Mean nr. of cracks</th>
<th>Average crack width [mm]</th>
<th>Min. expected load capacity (grid only) [kN] theoretical/measured</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1 reference</td>
<td>8.7</td>
<td>8.7</td>
<td>1</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>M2 reference</td>
<td>8.3</td>
<td>8.3</td>
<td>1</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>M1-0-M</td>
<td>9.2</td>
<td>7.2</td>
<td>2.3</td>
<td>N/A</td>
<td>14/22</td>
</tr>
<tr>
<td>M1-0-S</td>
<td>8.3</td>
<td>7.5</td>
<td>2.7</td>
<td>0.7</td>
<td>7/8.8</td>
</tr>
<tr>
<td>ECC-0-M</td>
<td>15.0</td>
<td>8.3</td>
<td>6.7</td>
<td>0.3</td>
<td>14/22</td>
</tr>
<tr>
<td>ECC-0-S</td>
<td>17.3</td>
<td>9.7</td>
<td>7.0</td>
<td>0.2</td>
<td>7/8.8</td>
</tr>
<tr>
<td>M2-0-M</td>
<td>11.9</td>
<td>6.8</td>
<td>2.3</td>
<td>0.7</td>
<td>14/22</td>
</tr>
<tr>
<td>M1-15-M</td>
<td>7.7</td>
<td>5.7</td>
<td>1.3</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>M2-15-M</td>
<td>8.5</td>
<td>7.1</td>
<td>1.7</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>M2-15-S</td>
<td>6.6</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>M1-90-M</td>
<td>15.9</td>
<td>6.6</td>
<td>4.0</td>
<td>N/A</td>
<td>14/22</td>
</tr>
<tr>
<td>M1-90-S</td>
<td>8.9</td>
<td>7.4</td>
<td>4.3</td>
<td>0.6</td>
<td>9.8/8.1</td>
</tr>
<tr>
<td>M2-90-S</td>
<td>9.0</td>
<td>5.9</td>
<td>6.0</td>
<td>N/A</td>
<td>9.8/8.1</td>
</tr>
</tbody>
</table>

As a general remark, we can conclude that there is a significant difference in the behaviour of the quasi-brittle and “ductile” specimens after first cracking. Curves of quasi-brittle mortars M1 and M2 are jagged and have a significant drop in load carrying capacity right after the first crack (and after every further crack developing). Contrary to the quasi-brittle ones, the ECC specimens after the first crack show a further load increase until the peak load. These curves are smooth due to the fibre-bridging characteristics of the ECC. However, it has to be noted that only a total number of six ECC specimens were tested, so our results are not conclusive.
First cracking strengths are slightly higher in the ECC specimens (8.3-9.7 kN) compared to the quasi-brittle specimens (5.7-7.5 kN). In case of the ECC, it was difficult to determine, even from the test data, when exactly the actual cracks form, because of the smooth transition between the un-cracked and the cracked stage. Studying the load-deformation graphs of the ECC specimens and applying tendency lines between the pre- and post-cracking stages, we approximated the first cracking loads. It revealed that the initial cracking of the ECC does in fact occur at a later stage than for the M1 and M2 specimens.

The most consistent behaviour we could observe in the M1-90-medium and M1-90-small specimens. Interestingly, in both of the ECC series, we had one specimen behaving very differently both in terms of first cracking load and stiffness (see discussion).

7.3. Comparative load-deformation plots

Specimens with similar grid size, and/or orientation have been paired and plotted in Figures. 16-19. Figure 16 shows all medium grids when placed longitudinally, in different mortars. Figure 17 compares all transversally placed grids. Figure 18 shows the small grid placed longitudinally. Finally, Figure 19 illustrates the effects of the (medium) grid orientation from 0° to 90° for one quasi-brittle mortar (M1).
From Figures 16-19 along with the individual plots against the bare grid (Figures 5-9) it can be concluded that a significant and consistent tension-stiffening effect can only be shown for the ECC, while there is only a slight to moderate increase in load capacity with the quasi-brittle mortars. First cracking in ECC happens slightly later than in M1 and M2. In case of the small grid (Figure 18), after first cracking, the load capacity does not increase significantly if we use quasi-brittle mortars, while it does in the ECC-based specimens. All 90° grids are plotted in Figure 17. The M1 mortar gave the most concise results, highest failure loads, and largest deformations, while the most brittle mortar, M2 yielded the most jagged and least concise results allowing very little deformations compared to any other combinations.

Finally, Figure 19 illustrates the changes in stiffness and load capacity with the grid orientation for the medium grid with the M1 mortar. Unexpectedly (and against the material data given in Table 1), the transversally rotated grid showed to be the strongest, while the 15° grid the weakest. In addition, the 90° medium grid combinations accommodated the largest elongations.

7.4. Crack widths, crack patterns

Figures 20-21 show two ARAMIS plots for the specimens M1-90-small and ECC-0-small. The crack widths of the ECC specimens are very small, ranging from 0.20 mm to 0.40 mm of the medium grid, and 0.08 mm to 0.50 mm for the small grid and they are not fully developed. In addition to that, the cracks are more evenly distributed in the ECC. In contrast, the crack widths determined by ARAMIS are ranging from 0.20 mm to 0.80 mm of the M1-90-small and between 0.75 mm and 1 mm for the M2-0-medium. “Average crack widths”, defined as the sum of crack widths divided by the number of the cracks within the test field, are also given in Table 6, where available.
Figure 20 – Crack widths recorded by ARAMIS, M1-90-small

Figure 21 – Crack widths recorded by ARAMIS, ECC-0-small

7.5. ARAMIS “strain images”

The crack development was monitored by ARAMIS. The strain overlay images taken at the moment of failure confirm the characteristic ductile behaviour of the ECC as it shows more numerous, finer and more evenly distributed crack patterns compared to the quasi-brittle (M1 and M2) mortars. Where no strain images were available (Figure 22a, f, g and h), failure photos are given. The cracks in all specimens developed in line with the grid tows.
8. DISCUSSION

We can compare the behaviour of the composite with the tensile properties of the bare grid. The specimens with the medium or small grid have 4 or 7 tows in the test section, respectively. The load capacity of the bare grid can be calculated based on manufacturer data. For the medium grid, it is 3.5 kN/tow in both directions, while for the small grid it is 1.4 kN/tow and 1.0 kN/tow in longitudinal and transversal direction, respectively. Test data from Blanksvärd gave much higher values [23]. Compared to these, the failure loads of the composite should be higher. Examining the failure loads, however, we can conclude that with quasi-brittle mortars we get significantly lower values (we do not even reach the nominal capacity of the grid) in all cases except for the transversally placed medium grid. The ECC specimens exceed the manufacturer-given grid failure loads, but still fall behind the tested values, suggesting premature failure of the grid.

The most concise, and homogenous test series were M1-90-medium and M1-90-small, with quasi-brittle, but fibre-reinforced mortars while the most scattered ones are the M2-series. The significant differences within the same test series (in particular, M2-15-small and M2-90-small)
are attributed to micro-cracks developing during the de-molding process, due to the very fragile geometry.

In case of the ECC, there is one specimen in both series showing a very different behaviour. One possible explanation to this could be the way we mixed the mortars. We could only mix a very small quantity in one batch because we used a dedicated mortar mixer with a limited capacity. It is possible that there were some very slight changes in the fibre content (2 vol. %) and/or water-to-cement ratios in the two separate mixes. Based on the known behaviour of the bare ECC (Figure 1), we believe that ECC-0-medium (3) and ECC-0-small (3) should be cancelled out of the results. The remaining specimens show a similar behaviour in terms of first cracking strength and stiffness.

It was observed that the medium grid performs very differently depending on its orientation. Two possible reasons for the “over-performance” of the medium grid when transversally placed are (1) that the joints can deform more without fibre breakage in that direction. The grid joints look and behave differently depending on the grid orientation; this is due to the “woven” nature of the grid. (2) Another possible explanation is the additional anchorage provided by the epoxy surplus, which for some reason is only present on the transversal fibre tows. In earlier tests by [5] however, it was shown that sanding the grid (giving increased bond) creates stress concentrations and premature rupture, therefore it cannot be stated that the additional anchorage equals to increased load capacity. Further research is needed in this regard.

The 15° orientated specimens, compared to the longitudinally placed reinforcement, do not develop as many cracks, and have a slightly reduced tensile strength. Cracks developing here tend to follow the grid tows. Finally, the behaviour of the small is less direction-dependent than that of the medium grid.

Some specimens initiated cracks near the ends of the test field. We attributed this to the fact that the dogbone geometry is prone to initiating cracks near sharp changes in cross section because of possible local stress concentrations. Using improved (curved) dogbone geometry, however, would have made the testing procedure far more time consuming and expensive. The test setup showed to be particularly sensitive to de-moulding, handling, and testing because of the very thin test field.

In addition to the possibly too low curing time before de-molding, there is an initial curvature in the grid strips because of the factory shape (roll). The grids were tensioned in the slits at both ends before casting in order to reduce the curvature. Yet in such a thin cross-section, the effect of the slightest curvature may be significant, and it might result in reduced performance. This effect has not been investigated more in detail.

9. CONCLUSIONS

The goals of the experimental program have been fulfilled in great part. A wide scale of material combinations has been tested, yet not all possible combinations. Adaptation of the dogbone geometry to testing large MBC-specimens has been successful with some limitations. Most of the specimens have been able to initiate cracks within the pre-defined test field, but an improved geometry would yield results that are more concise.

Multiple cracking and a significant tension stiffening behaviour were observed when applying a ductile, PVA-reinforced ECC as bonding agent, which, based on the two only test series, has
proven to be superior to quasi-brittle mortars. ECC has given significantly higher failure loads and prevented pronounced drops in load capacity by its increased ductility. Recorded peak values in the load-displacement curves show a more balanced behaviour for the ECC specimens. The same plots for the quasi-brittle mortars are jagged indicating a more uneven stress distribution and possible local failures in the grid joints. Typical brittle failure and corresponding crack patterns were recorded in case of the quasi-brittle M1 and M2 mortars.

10. FURTHER RESEARCH

Quasi-brittle mortars rely on high bond strength between CFRP reinforcement and the mortar, to accommodate the high internal stresses. Therefore, bond slip or local high stress concentrations may result in failure of the grid joints. Using a mortar with improved ductile deformation capabilities, where the CFRP reinforcement and the cementitious matrix deform close to identically, results in a composite material where apparently high mechanical bond stresses in local areas, as the grid intersections between the longitudinal and transversal tows are significantly reduced, thereby preventing bond slip damage to the CFRP-reinforcement.

The high fly ash content of ECC results in a refined and densified grain structure. This may improve the bond strength between the grid and the mortar, as confirmed by [11] for yarns/textiles embedded in cementitious matrices. The apparently better bond characteristics of ECC are also partly due to the compatible deformation behaviour between FRP reinforcement and ECC, which makes ECC very attractive for further investigation in combination with FRP grids.

Due to its pseudo strain hardening and fibre bridging properties, and for the additional mechanical anchorage it provides for the FRP grid, ECC will be tested further as a bonding agent for mineral-based strengthening. The different geometry (and associated rigidity, and deformation capacity) of the grid joints in the two perpendicular directions could also be further investigated, as this may have caused the significant “over-performance” of the medium grid in transversal direction.

Finite element modelling of the interaction between grid and mortar would give a better understanding of structural applications where mineral-based strengthening systems are subject to tensional/splitting forces, or axial forces combined with bending.

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