Flow Characteristics of Macro-Synthetic Fiber-Reinforced Self-Consolidating Concrete

by D. Forgeron and A. Omer

Synopsis: To evaluate the flow characteristics of macro-synthetic fiber-reinforced self-consolidating concrete (MSFRSCC), a total of 20 non-air entrainment self-consolidating concrete (SCC) mixtures with varying w/c ratios, macro-synthetic fiber lengths, and fiber dosages rates were evaluated. The flow characteristics of each mixture were evaluated using four typical SCC workability test methods: slump flow, filling capacity, L-box, and V-funnel tests. The plastic shrinkage cracking resistance, compressive strength and flexural strength of each mixture were also evaluated. The objective was to develop an understanding of the factors that influence the flow characteristics of MSFRSCC and determine if criteria set for conventional SCC can be applied to MSFRSCC.

The testing results demonstrated that fiber lengths of 50 mm cause significant internal friction leading to mixture stability issues when attempting to increase the volume of high range water reducer to produce acceptable slump flow values without viscosity modifying admixtures. Reducing fiber length to 38mm led to reduction in the internal friction allowing satisfactory slump flow, filling capacity, and V-funnel flow time to be achieved with slight mixture modifications and no viscosity modifying admixtures were required. The addition of fibers did cause lower than acceptable L-Box test results where mixtures were made to change direction and flow between closely spaced bars. It was concluded that the slight increase in internal friction produced by the addition of fibers caused the low L-Box results and not any form of blockage. The plastic shrinkage test results showed that the addition of 0.40% fibers by volume led to as much as 70% reduction in total crack area and up to 50% reduction in maximum crack width as compared to plain concrete. The results obtained from this research clearly shows that is it possible to develop highly crack resistant MSFRSCC mixtures for concrete structures.

Keywords: flowability; internal friction; macro-synthetic fiber reinforcement; mixture stability; passability; plastic shrinkage cracking resistance; self-consolidating concrete; self leveling concrete
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Mr. Alkilani Omer currently works as a project manager for the Libyan government. Mr. Alkilani completed his M.A.Sc in 2006 where he developed macro-synthetic fiber-reinforced self-consolidating concrete for marine structures in hot climates. Mr. Alkilani will begin his Ph.D studies in 2010 researching the structural response of macro-synthetic self-consolidating fiber reinforcement at Dalhousie University, Nova Scotia Canada. His research interests include self-consolidating concrete, fiber-reinforced-self-consolidating concrete, fiber-reinforced concrete.

INTRODUCTION

During the construction of conventional concrete structures many factors, such as improper or inadequate placement and consolidation techniques as well as inadequate curing regimes, can lead to poor compaction and surface cracking which, when combined, results in high concrete permeability. The aformentioned consolidation, compaction and curing issues reduce the ability for concrete resistance to the ingress of aggressive agents (chloride ions, moisture, oxygen, etc) and can lead to the corrosion of reinforcing steel. Corrosion leads to serviceability issues and, in extreme cases, can cause large reductions in overall structural capacity or even failure of the member or structure. Ultimately, a mixture that possesses both a high cracking resistance and the ability to self consolidate would mitigate against many of these “serviceability” problems.

The addition of macro-synthetic fiber reinforcement has been shown to improve cracking resistance of normal concrete while self-consolidating concrete has been used extensively to reduce the occurrence of improper consolidation and placement methods (Trottier et al. 2002, Khayat 1999). It follows that the combination of these two technologies would eliminate many of the issues mentioned above.

It has been shown that the addition of macro-fibers leads to a reduction in workability of concrete mixtures due to the additional surface area and the internal friction produced by the addition of fibers (Balaguru and Shah 1992). Most of the research that has been conducted on fiber-reinforced concrete has been focused on the effect of steel fibers on the rheology of SCC mixtures and the development of steel fiber-reinforced self-consolidating concrete mixture design procedures (Khayat and Roussel 2000, Grunewald 2004, Bui et al 2003, De Larrard 1999, Ferrara et al. 2007).

Fewer studies have been conducted on macro-synthetic fiber reinforcement and its effect on the flow characteristics of SCC. One study in the literature details an invention into the potential synergistic effects of incorporating steel and synthetic macro-fibers in various hybrid (single, binary, and ternary) combinations into SCC. They noted that the addition of 40 and 50mm macro-synthetic fibers caused a significant reduction of slump flow and blockage in the L-Box test (Moncef and Ladanchuck, 2004). Others noted that SCC can be achieved when macro-synthetic fibers are used but total fiber volume fractions must be less than 0.4% by volume. They also suggested that reduced fiber length as well as size and content of coarse aggregates are necessary to maintain adequate flow properties of FRSCC (Grunewald and Walraven 2001).

Recent advancements have lead to development of an innovative self-fibrillating macro-synthetic fibers that partially fibrillate upon mixing. This increased the surface area in contact with the cement matrix and improved the resulting mechanical properties (Trottier and Mahoney 2001). These fibers
become flexible after mixing and therefore will not behave like steel or conventional macro-synthetic fibers when introduced into self-consolidating concrete mixtures. It is therefore important to study the flow characteristics of self-consolidating concrete mixtures that incorporate this type of macro-synthetic fiber reinforcement in order to identify and characterise the main factors affecting its flow. The information garnered from this study can then be used in the development of macro-synthetic fiber-reinforced self-consolidating (MSFRSCC) mixtures and mixture design procedures.

Overall it is important to provide sufficient cement paste to allow free movement of the coarse aggregate particles past one another as the mixture flows. The paste must also be sufficiently fluid to allow movement in a relatively rapid manner while possessing adequate yield stress to prevent segregation of the mixture while at rest. It follows that the addition of fiber reinforcement will not only require additional paste to coat the fibers but the paste fluidity will also need to be increased in order to allow movement of the aggregate through a paste that contains fiber reinforcement. The balance comes in attempting to maintain the stability of the mixture (ensuring no segregation) while increasing fluidity. One option is to reduce the coarse aggregate content of the mixture in order to compensate for the additional fiber induced friction and surface area. This will lead to less volumetric stability and higher potential for cracking. The key is to ensure that the mixture adjustment required to produce acceptable flow characteristics does not cause more cracking potential than the added fibers can prevent.

A total of 20 non-air entrainment self-consolidating concrete (SCC) mixtures with varying w/c ratios, self-fibrillating macro-synthetic fiber lengths and dosage rates plus one normal concrete control mixture (NC) were evaluated. The flow characteristics of each mixture were evaluated using four typical SCC workability test methods: slump flow, filling capacity, L-box, and V-funnel flow time tests. The cracking resistance, compressive strength and flexural strength of each mixture were also evaluated to determine the influence of various mixture adjustments on the engineering properties of each mixture. Ultimately, this research was conducted to further develop an understanding of the factors that influence the flow characteristics of MSFRSCC.

**TESTING PROGRAM**

In this study, a total of 21 non-air-entrained mixtures were investigated; one normal concrete mixture (NC), shown in Table 1 and 20 SCC mixtures, shown in Table 2. The aggregate grading for the normal concrete mixture and SCC mixtures are shown in Figure 1. The coarse to fine aggregate volume ratio was maintained at 50/50 therefore one curve describes the combined aggregate grading for all 20 SCC mixtures in this study.

<table>
<thead>
<tr>
<th>Table 1. Reference Normal Concrete Mixture Composition</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Mix No.</strong></td>
</tr>
<tr>
<td>NC</td>
</tr>
</tbody>
</table>

<sup>*</sup> Type GU (St. Lawrence Cement)  
<sup>**</sup> 19mm max. Crushed rock (SG=2.72, abs = 0.9 %)  
<sup>+</sup> Natural sand (FM = 3.0, SG = 2.6, abs = 1.5%)  
<sup>++</sup> Superplasticizer = Euclid Chemical (Plastol 5000 SCC)

The plain SCC mixtures were proportioned with w/c ratios of 0.40, 0.42, and 0.45 and constant water contents (190L/m³) labelled as SCC1, SCC2, and SCC3, respectively. The SCC mixture designs were created to evaluate the effect of volume of paste contents, Vₚ, while also changing compressive strength of the resulting mixtures. The normal concrete reference mixture had a w/c ratio of 0.4 and was labelled NC.
All 20 SCC and MSFRSCC mixtures (0.15m³ each) were prepared in a 0.25 m³ (9.5 ft³) laboratory drum mixer. The batch sequence used for all SCC mixtures was as follows:

- Add the sand to the mixer and mix for 30 seconds (moisture content evaluation and correction, as established by ACI recommended practices 211.1)
- Add the coarse aggregate to the mixer and mix for 30 seconds
- Add the fibers and mix for 3 minutes (not conducted for the plain SCC)
- Add 50% of the adjusted water and mix for 30 seconds
- Add all the cement to the mixer and mix for 30 seconds
- Add HRWR and the balance of the water and mix for 4 minutes
- Let stand for 2 minutes and then mix for 4 additional minutes

A commercially available self-fibrillating macro-synthetic fiber was selected for this study. This fiber is unique in that it is a monofilament fiber that partially fibrillates when mixed in concrete. The cross sectional geometry, specific gravity, tensile strength and modulus of elasticity of the fiber are given in Table 2. Two fiber lengths (38 mm and 50 mm) were used in this study to investigate the effects of fiber length on the flow characteristics of SCC.

To evaluate the influence of fiber length on the flow characteristics of SCC, two separate self-fibrillating macro-synthetic fiber lengths (38 mm and 50 mm) were evaluated in mixture SCC2. Initially, the influence of 50mm fibers on the flow characteristics of SCC was evaluated in mixture SCC2. Two fiber volume fractions 0.20% and 0.50% were used and labelled as MSFRSCC2-6 and MSFRSCC2-7 as shown in Table 3. Acceptable flow characteristics without the use of viscosity modifying admixtures could not be achieved therefore a shorter 38 mm self-fibrillating macro-synthetic fiber was used for the balance of the study. The influence of the 38 mm self-fibrillating macro-synthetic fibers on each of the SCC mixtures (SCC1, SCC2, SCC3) was then evaluated at five different fiber volume fractions ranging from 0.2% to 0.4%. The MSFRSCC mixtures were labelled as MSFRSCC#-1, MSFRSCC#-2, MSFRSCC#-3, MSFRSCC#-4, and MSFRSCC#-5, which contained the 38mm long fiber at fiber volume fractions of 0.20%, 0.25%, 0.30%, 0.35%, and 0.40% respectively. The volume fractions of 0.20%, 0.25%, 0.30%, 0.35%, 0.40%, and 0.5% correspond to fiber addition rates of 1.8kg/m³ (3.0lbs/yd³), 2.3kg/m³ (3.9lbs/yd³), 2.8kg/m³ (4.7lbs/yd³), 3.2kg/m³ (5.4lbs/yd³), 3.7kg/m³ (6.2lbs/yd³), and 4.6kg/m³ (7.75lbs/yd³), respectively.
After batching and mixing, the various workability tests used to characterize the flow characteristics of each SCC mixture were conducted in the following sequence: slump flow test, filling capacity test (34mm between bars), L-box test (34mm spacing between bars, as the most common arrangement of reinforcing bars) then V-funnel test. A photo of each test is shown in Figure 2.
To evaluate and compare the shrinkage potential of each mixture, plastic shrinkage cracking tests were conducted immediately after the fresh properties were evaluated. Two identical concrete slab specimens were cast in plywood moulds 610 mm x 915 mm x 50 mm (24” x 36” x 2”) with uniformly distributed internal and side restraints. The specimens were finished and immediately exposed to a uniform drying condition for 24 hours. The temperature during testing ranged from 21 °C to 23 °C (70°F -73°F). The relative humidity in the concrete laboratory testing room was monitored during all tests and ranged from 65% to 75% during the period of testing. Two fans were used to generate a constant air speed of approximately 4 m/s (13.1fps) over the surface of the specimens. The fans were placed at the short edge of each plate as shown in Figure 3. Plastic shrinkage testing was not conducted on mixture MSFRSCC2-6 and MSFRSCC2-7 because acceptable flow characteristics could not be achieved.

After 24 hours of exposure, the total area of cracking, total number of cracks, and the average crack widths were measured and the results were averaged. The total area of cracking of each specimen was calculated by summing up the product of the length and average width of each crack on the surface of the specimen.
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RESULTS

The slump flow, filling capacity, L-Box blockage ratio and V-funnel time test results for the 38 and 50mm fiber-reinforced mixtures are presented in Table 4. Comparing the flow characteristics of MSFRSCC2-1 (38mm fiber @ 0.2% by volume) to MSFRSCC2-6 (50mm fiber @ 0.2% by volume) it is clear that acceptable slump flows can be achieved for both fiber lengths. However, when filling capacity, L-Box, and V-funnel test are performed on both mixtures, the effect of fiber length became clear. It was clearly evidenced that fiber length plays a very important role in the flow characteristics of SCC when moving around, over or through obstacles. It is important to reiterate that the same fiber cross section was used in the 38mm and 50mm fiber therefore the surface area of the fibers in both mixtures is almost identical. The number of fibers for a given fiber volume fraction is 31% higher for the 38mm compared to the 50mm fiber. This indicates that fiber length has a stronger influence on flow characteristics than the number of synthetic fibers in a mixture.

Table 4. Fresh Properties

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Slump Flow mm/(in)</th>
<th>Filling Capacity %</th>
<th>L-Box Blockage Ratio H2/H1</th>
<th>V-Funnel Time Sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCC1</td>
<td>637/ (25.1)</td>
<td>85.1</td>
<td>0.74</td>
<td>2.3</td>
</tr>
<tr>
<td>MSFRSCC 1-1</td>
<td>705/ (27.8)</td>
<td>86.8</td>
<td>0.67</td>
<td>2.9</td>
</tr>
<tr>
<td>MSFRSCC 1-2</td>
<td>628/ (24.7)</td>
<td>79.6</td>
<td>0.70</td>
<td>2.9</td>
</tr>
<tr>
<td>MSFRSCC 1-3</td>
<td>665/ (26.2)</td>
<td>82.6</td>
<td>0.67</td>
<td>3.0</td>
</tr>
<tr>
<td>MSFRSCC 1-4</td>
<td>715/ (28.1)</td>
<td>89</td>
<td>0.67</td>
<td>3.0</td>
</tr>
<tr>
<td>MSFRSCC 1-5</td>
<td>700/ (27.6)</td>
<td>93.9</td>
<td>0.70</td>
<td>2.8</td>
</tr>
<tr>
<td>SCC2</td>
<td>625/ (24.6)</td>
<td>78.4</td>
<td>0.70</td>
<td>2.9</td>
</tr>
<tr>
<td>MSFRSCC 2-1</td>
<td>640 / (25.2)</td>
<td>82.6</td>
<td>0.74</td>
<td>2.8</td>
</tr>
<tr>
<td>MSFRSCC 2-2</td>
<td>625/ (24.6)</td>
<td>82.7</td>
<td>0.67</td>
<td>2.6</td>
</tr>
<tr>
<td>MSFRSCC 2-3</td>
<td>680/ (26.8)</td>
<td>90.5</td>
<td>0.73</td>
<td>2.4</td>
</tr>
<tr>
<td>MSFRSCC 2-4</td>
<td>700/ (27.6)</td>
<td>91.7</td>
<td>0.67</td>
<td>2.6</td>
</tr>
<tr>
<td>MSFRSCC 2-5</td>
<td>668/ (26.3)</td>
<td>82.6</td>
<td>0.60</td>
<td>3.0</td>
</tr>
<tr>
<td>MSFRSCC 2-6</td>
<td>680/ (26.8)</td>
<td>72</td>
<td>0.6</td>
<td>10</td>
</tr>
<tr>
<td>MSFRSCC 2-7</td>
<td>550/ (21.6)</td>
<td>65</td>
<td>0</td>
<td>Blockage</td>
</tr>
<tr>
<td>SCC3</td>
<td>675/ (26.6)</td>
<td>84.7</td>
<td>0.86</td>
<td>2.0</td>
</tr>
<tr>
<td>MSFRSCC 3-1</td>
<td>635/ (25.0)</td>
<td>76.8</td>
<td>0.63</td>
<td>2.5</td>
</tr>
<tr>
<td>MSFRSCC 3-2</td>
<td>667/ (26.3)</td>
<td>79</td>
<td>0.63</td>
<td>2.6</td>
</tr>
<tr>
<td>MSFRSCC 3-3</td>
<td>632/ (24.9)</td>
<td>79.7</td>
<td>0.60</td>
<td>2.8</td>
</tr>
<tr>
<td>MSFRSCC 3-4</td>
<td>640/ (25.2)</td>
<td>82</td>
<td>0.53</td>
<td>2.7</td>
</tr>
<tr>
<td>MSFRSCC 3-5</td>
<td>655/ (25.8)</td>
<td>76.1</td>
<td>0.53</td>
<td>2.7</td>
</tr>
</tbody>
</table>

Coefficient of Correlation 0.749 0.147 0.086

50mm long self-fibrillating macro-synthetic fiber

Figure 4 shows the slump flow versus filling capacity and L-Box results. The graph includes trendlines for each parameter and the associated coefficient of determination, $R^2$ and the 70% limit for acceptable L-Box and Filling Capacity.
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To evaluate the correlation between slump flow and the other parameters, the coefficient of correlation between slump flow and filling capacity was evaluated for the 38mm fiber mixtures and it was found to be 0.75 indicating a strong correlation between these two variables. When evaluated for slump flow versus L-Box and versus V-Funnel the correlation coefficients were calculated as 0.147 and 0.086 which indicated very weak correlation between these variables. The correlation between slump flow and filling capacity is clear while the L-Box results are scattered and do not show any strong correlation.

The nature of the flow in a slump flow test and filling capacity test is similar in that the concrete moves horizontally driven by gravity or the energy induced during the filling process. It appears that the movement over horizontal obstructions, as is found in the filling capacity test, is very different than movement through narrowly spaced vertical obstructions as is the case in the L-Box test. It is also important to note that the movement of concrete in the L-Box test is very different from the slump flow and filling capacity tests. In the L-Box test the vertical chamber of the apparatus is filled and then the gate opened and the concrete is forced to move down and then change direction while being forced around closely spaced reinforcements. Figures 5 and 6 show that with the addition of fibers, the L-Box ratio is decreased due to the presence of a small shelf of concrete behind the reinforcing bars. Within the horizontal portion of the L-Box, the FRSCC remains almost horizontal as was the case in the slump flow and filling capacity tests. To determine if the concrete in the shelf was being held due to some sort of blockage a small amount of additional concrete was poured into the vertical chamber of L-box apparatus and the shelf of concrete was pushed through the reinforcing bar however the additional poured concrete remained behind the bars. This is attributed to the additional internal friction provided by the fiber reinforcement.

Table 4 shows that the macro-synthetic fiber-reinforced mixtures performed better in the L-Box test when lower water to cement ratios were used. To illustrate the difference, compare the photos in Figure 5 (series 2, w/c = 0.42) to those in Figure 6 (series 3, w/c = 0.45). It is clear that the additional paste content in Series 2 produced lower internal friction and better L-Box test results.
in Figure 5 (series 2, w/c = 0.42) to those in Figure 6 (series 3, w/c = 0.45). It is clear that the additional paste content in Series 2 produced lower internal friction and better L-Box test results.

Figure 5. L-Box Test (Series 2)

Figure 6. L-Box Test (Series 3)
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The total cracking area, average crack widths and maximum crack widths are presented in Table 5. The total area of cracking of each specimen was calculated by summing up the product of the length and average width of each crack on the surface of the specimen. Plastic shrinkage testing was not conducted on mixture MSFRSCC2-6 and MSFRSCC2-7.

The relationship between fiber content and total cracking area for each mixture type used in this study is shown in Figure 7. The average crack width versus fiber volume fraction was also evaluated and is shown in Figure 8. It is clear that there is a strong correlation between total cracking area and fiber volume fraction in each mixture type (correlation coefficient from -0.94 to -0.97) as well as between average crack width and fiber volume fractions in each mixture type (correlation coefficient from -0.84 to -0.93). The correlation can be evaluated visually in Figure 7 and Figure 8.

### Table 5. Plastic Shrinkage Test Results

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>W/C</th>
<th>Total Cracking Area (mm²/(in²))</th>
<th>Average Crack Width mm/(in)</th>
<th>Reduction in Crack Area %</th>
<th>Max Crack Width mm/(in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC</td>
<td>0.40</td>
<td>318/(0.49)</td>
<td>0.35/(0.138)</td>
<td>-</td>
<td>0.50/(0.02)</td>
</tr>
<tr>
<td>SCC1</td>
<td>0.40</td>
<td>490/(0.76)</td>
<td>0.31/(0.012)</td>
<td>54.0</td>
<td>0.40/(0.016)</td>
</tr>
<tr>
<td>MSFRSCC 1-1</td>
<td>387/(0.60)</td>
<td>0.27/(0.0106)</td>
<td>21.0</td>
<td>0.30/(0.012)</td>
<td></td>
</tr>
<tr>
<td>MSFRSCC 1-2</td>
<td>308/(0.48)</td>
<td>0.15/(0.005)</td>
<td>37.1</td>
<td>0.30/(0.012)</td>
<td></td>
</tr>
<tr>
<td>MSFRSCC 1-3</td>
<td>220/(0.34)</td>
<td>0.15/(0.005)</td>
<td>55.0</td>
<td>0.20/(0.008)</td>
<td></td>
</tr>
<tr>
<td>MSFRSCC 1-4</td>
<td>176/(0.27)</td>
<td>0.15/(0.005)</td>
<td>64.0</td>
<td>0.20/(0.008)</td>
<td></td>
</tr>
<tr>
<td>MSFRSCC 1-5</td>
<td>145/(0.22)</td>
<td>0.15/(0.005)</td>
<td>70.4</td>
<td>0.20/(0.008)</td>
<td></td>
</tr>
<tr>
<td>SCC2</td>
<td>0.422</td>
<td>287/(0.44)</td>
<td>0.25/(0.010)</td>
<td>-</td>
<td>0.40/(0.016)</td>
</tr>
<tr>
<td>MSFRSCC 2-1</td>
<td>159/(0.25)</td>
<td>0.18/(0.007)</td>
<td>44.6</td>
<td>0.30/(0.012)</td>
<td></td>
</tr>
<tr>
<td>MSFRSCC 2-2</td>
<td>150/(0.23)</td>
<td>0.15/(0.006)</td>
<td>47.7</td>
<td>0.20/(0.008)</td>
<td></td>
</tr>
<tr>
<td>MSFRSCC 2-3</td>
<td>123/(0.19)</td>
<td>0.15/(0.006)</td>
<td>57.1</td>
<td>0.20/(0.008)</td>
<td></td>
</tr>
<tr>
<td>MSFRSCC 2-4</td>
<td>122/(0.19)</td>
<td>0.15/(0.006)</td>
<td>57.5</td>
<td>0.20/(0.008)</td>
<td></td>
</tr>
<tr>
<td>MSFRSCC 2-5</td>
<td>117/(0.18)</td>
<td>0.15/(0.006)</td>
<td>59.2</td>
<td>0.15/(0.006)</td>
<td></td>
</tr>
<tr>
<td>SCC3</td>
<td>0.45</td>
<td>123/(0.19)</td>
<td>0.15/(0.006)</td>
<td>-</td>
<td>0.35/(0.014)</td>
</tr>
<tr>
<td>MSFRSCC 3-1</td>
<td>81/(0.13)</td>
<td>0.13/(0.005)</td>
<td>34.1</td>
<td>0.20/(0.008)</td>
<td></td>
</tr>
<tr>
<td>MSFRSCC 3-2</td>
<td>74/(0.11)</td>
<td>0.13/(0.005)</td>
<td>40.0</td>
<td>0.20/(0.008)</td>
<td></td>
</tr>
<tr>
<td>MSFRSCC 3-3</td>
<td>71/(0.11)</td>
<td>0.13/(0.005)</td>
<td>42.3</td>
<td>0.20/(0.008)</td>
<td></td>
</tr>
<tr>
<td>MSFRSCC 3-4</td>
<td>66/(0.10)</td>
<td>0.13/(0.005)</td>
<td>46.3</td>
<td>0.20/(0.008)</td>
<td></td>
</tr>
<tr>
<td>MSFRSCC 3-5</td>
<td>70/(0.11)</td>
<td>0.10/(0.004)</td>
<td>43.1</td>
<td>0.15/(0.006)</td>
<td></td>
</tr>
</tbody>
</table>

By comparing the total cracking area of mixture NC (490mm²) and mixture SCC1(318mm²), both having a w/c ratio of 0.4, the effect of paste content on the cracking resistance is evident. The higher cement paste volume \( V_p \) of SCC1(166L/m³) vs NC \( V_p =139L/m³ \) and therefore the lower volume stabilizing coarse aggregate content results in a 54% greater crack area on the surface of the panel. It is important to note that slabs with a high number of cracks will have a lower average and maximum crack width due to the configuration of this testing procedure. In effect, as a particular crack increases due to internal shrinkage induced strain, mixtures with higher cracking potential will tend to form another crack nearby causing the first crack to contract. For this reason mixture SCC1, which has a higher cracking potential has an average and maximum crack widths lower than those of NC mixture. Within the same mixture design, the effect of fiber addition can be compared by looking at total crack area, average crack width and maximum crack width in a similar way. As the fiber dosage rate is increased, a marked reduction in all three parameters is noted in all mixtures.

To determine how mixture design and cracking potential are affected by the addition of self-fibrillating macro-synthetic fiber reinforcement, the total cracking area is plotted against the fiber volume fraction in Figure 7 and average crack width is plotted against fiber volume fraction in Figure 8. It is clear from these graphs that the addition of fibers increases the cracking resistance (total crack area, average crack width and maximum crack width) of all mixtures, however the effectiveness (slope of the curves
Flow Characteristics of Macro-Synthetic Fiber-Reinforced Concrete

in Figure 7 and Figure 8) of this fiber decreases as the paste content \( V_p \) and therefore cracking resistance of the mixture increases. In practical terms, an additional 0.1% by volume will have more of an effect in a mixture with high paste volume than one of low cracking potential. Figures 7 and 8 illustrate the large impact that relatively low fiber dosage rates of 0.4% by vol. (3.7kg/m\(^3\) (6.2lbs/yd\(^3\)) can have on SCC mixtures reducing the cracking area by 70% and average crack width by 50%.

![Figure 7. Fiber Volume Fraction versus Total Cracking Area](image)

![Figure 8. Fiber Volume Fraction versus Average Crack Width](image)

The addition of a modest 0.3% by volume (2.8kg/m\(^3\)(4.7lbs/yd\(^3\)), of self-fibrillating macro-synthetic fiber reinforcement to the mixture reduced the cracking area to 220mm\(^2\)/(0.34in\(^2\)) and the average crack and maximum crack width to 0.20mm/(0.008 in). Comparing this to the NC mixture we observe a total cracking area of 318mm\(^2\)/(0.49in\(^2\)), an average crack width of 0.35mm/(0.138 in) and a maximum crack width of 0.50mm/(0.02 in). Therefore it is clear that a MSFRSCC mixture can be produced with superior cracking resistance to a comparable normal concrete mixture and still maintain adequate flow characteristics.

The compressive strength and flexural strength of all mixtures, except MSFRSCC2-6 and MSFRSCC2-7, are shown in Table 6. The consistency in both compressive strength and flexural strength between all SCC mixtures confirms that the control measure put in place to accurately batch each mixture was successful. Comparing the NC mixture and SCC1 mixture shows that the alternate SCC mixture designed
in this study produced a very similar compressive strength to lab consolidated cylinders. In the field, the self compacting nature of the SCC mixture will ensure relatively uniform compressive strengths in a structure while the normal concrete mixtures “in place” compressive strength will depend on the quality of compaction achieved.

The slight reduction in flexural strength of all MSFRSCC mixtures compared to their respective control mixture could be attributed to the slight change in compaction of the granular skeleton caused by the presence of the macro-synthetic fiber reinforcement.

### Table 6. Compressive Strength and Flexural Strength Results at 28 days

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Compressive Strength MPa/(psi)</th>
<th>Flexural Strength MPa / (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC</td>
<td>49.8/(7223)</td>
<td>N/A</td>
</tr>
<tr>
<td>SCC1</td>
<td>48.2/(6991)</td>
<td>7.50/(1088)</td>
</tr>
<tr>
<td>MSFRSCC 1-1</td>
<td>48.2 /(6991)</td>
<td>7.35/(1066)</td>
</tr>
<tr>
<td>MSFRSCC 1-2</td>
<td>49.4/(7165)</td>
<td>7.20/(1044)</td>
</tr>
<tr>
<td>MSFRSCC 1-3</td>
<td>49.4/(7165)</td>
<td>7.12 /(1033)</td>
</tr>
<tr>
<td>MSFRSCC 1-4</td>
<td>52.7/(7644)</td>
<td>7.15/(1037)</td>
</tr>
<tr>
<td>MSFRSCC 1-5</td>
<td>50.5 /(7325)</td>
<td>7.20/(1044)</td>
</tr>
<tr>
<td>SCC2</td>
<td>44.1/(6396)</td>
<td>7.00/(1015)</td>
</tr>
<tr>
<td>MSFRSCC 2-1</td>
<td>44.6/(6469)</td>
<td>6.90/(1001)</td>
</tr>
<tr>
<td>MSFRSCC 2-2</td>
<td>44.5/(6454)</td>
<td>6.80/(986)</td>
</tr>
<tr>
<td>MSFRSCC 2-3</td>
<td>45.3/(6570)</td>
<td>6.85/(994)</td>
</tr>
<tr>
<td>MSFRSCC 2-4</td>
<td>46.6/(6759)</td>
<td>6.82/(989)</td>
</tr>
<tr>
<td>MSFRSCC 2-5</td>
<td>45.6/(6614)</td>
<td>6.90/(1001)</td>
</tr>
<tr>
<td>SCC3</td>
<td>42.0/(6092)</td>
<td>6.45/(936)</td>
</tr>
<tr>
<td>MSFRSCC 3-1</td>
<td>42.7/(6193)</td>
<td>6.17 /(895)</td>
</tr>
<tr>
<td>MSFRSCC 3-2</td>
<td>42.5 / (6164)</td>
<td>6.53/(947)</td>
</tr>
<tr>
<td>MSFRSCC 3-2</td>
<td>45.0/(6527)</td>
<td>6.21/(901)</td>
</tr>
<tr>
<td>MSFRSCC 3-4</td>
<td>42.6/(6179)</td>
<td>6.13/(889)</td>
</tr>
<tr>
<td>MSFRSCC 3-5</td>
<td>43.7/(6338)</td>
<td>6.13/(889)</td>
</tr>
</tbody>
</table>

**CONCLUSIONS**

Macro-synthetic self-fibrillating self-consolidating concrete mixtures can be designed with 38mm self fibrillating macro synthetic fibers with dosage rated up to 0.4% by volume with only ternary blend cement and high range water reducing admixtures. A strong correlation was found between slump flow and Filling capacity while the L-Box results do not correlated well with slump flow indicating that the movement of MSFRSCC in the L-Box test is very different from the slump flow and filling capacity tests. Mixtures with lower water to cement ratios (higher cement contents) performed much better in the L-Box test. Interestingly, fiber length was found to have a stronger influence on flow characteristics of SCC than the number of fibers in a mixture. If unrestricted flow is desired, as seen in the slump flow and filling capacity tests, fiber volume fractions can be increased up to 0.5%.

The addition of modest dosage rates (<0.4%) of self-fibrillating macro-synthetic fiber reinforcement can significantly reduce the cracking area, the average crack and maximum crack width of SCC. MSFRSCC mixture can be produced with superior cracking resistance to a comparable strength normal concrete mixture and still maintain adequate flow characteristics.
REFERENCES


D. Forgeron and A. Omer
Maximum Fiber Content and Passing Ability of Self-Consolidating Fiber-Reinforced Concrete

by S. Grünewald and J. C. Walraven

**Synopsis:** Self-consolidating fiber-reinforced concrete (SCFRC) combines the benefits of self-consolidating concrete (SCC) in the fresh state and an enhanced performance of fiber reinforced concrete (FRC) in the hardened state. The application of SCC improves the efficiency at building sites, allows rationally producing prefabricated concrete elements and improves the working conditions, the quality and the aesthetical appearance of concrete structures. By adding fibers to SCC bar reinforcement can be replaced, crack widths reduced, the durability improved and the load bearing capacity of a structure increased.

An extensive research study was carried out on the characteristics and the mix design of SCFRC that consisted of three parts: the fresh as well as the hardened state of SCFRC and the influence of the production process determined in three full-scale studies. This paper discusses two aspects of the mix design of SCFRC: the maximum fiber content and the required spacing of reinforcement at which blocking does not occur. Based on the analysis of experimental results mix design tools are proposed that allow predicting the maximum fiber content and the passing ability of SCFRC, which is essential information to obtain a homogeneous distribution of the fibers in a structure.

**Keywords:** flow behavior; maximum fiber content; mixture composition; passing ability; self-consolidating concrete; steel fibers
RESEARCH SIGNIFICANCE

Optimized SCFRC is a tailor-made concrete that might be an alternative for either typical SCC- or FRC-applications. Due to the special characteristics of SCFRC new fields of application can be explored. SCFRC can be optimized for various purposes: to apply the highest possible fiber content, to obtain the highest performance-cost ratio, to design the granular skeleton for the highest packing density or to produce with the lowest possible material costs. The knowledge about the fiber percent addition to SCC at a given mixture composition and how the fibers perform in the hardened state allows composing economical and performance-based mixtures. Tools to predict design characteristics and to optimize the mixture composition of SCFRC reduce the number of ‘trial and error’ experiments and indicate possibilities and limits.

INTRODUCTION

Fibers are known to affect characteristics of concrete in the fresh and the hardened state. Fibers bridge cracks and restrict their propagation. They contribute to an increased energy absorption compared to plain concrete. Steel and plastic fibers are the most common fiber types in the building industry; other fiber types like glass and carbon fibers contribute with a smaller part to the market.

Fibers and workability
Fibers affect the characteristics of concrete in the fresh state. They are needle-like particles that increase the resistance to flow and contribute to the formation of an internal structure of aggregate grains and fibers. In order to optimize the performance of each fiber, the fibers need to be homogeneously distributed and clustering of fibers has to be counteracted. The size of the fibers relative to the aggregates determines their distribution. To be effective in the hardened state it is recommended to choose fibers not shorter than the maximum aggregate size. Usually, the fiber length is 2-4 times that of the maximum aggregate size. The effect of fibers on the workability is mainly due to four reasons: First, the shape of the fibers is more elongated compared to the aggregates; the surface area at the same volume is higher. Second, stiff fibers change the structure of the granular skeleton, whereas flexible fibers fill the space between them. Stiff fibers push apart particles that are relatively large compared to the fiber length, which increases the porosity of the granular skeleton. Third, the surface characteristics of fibers differ from that of cement and aggregates, e.g. plastic fibers might be hydrophilic or hydrophobic. Finally, fibers can be deformed (i.e. have hooked ends or are wave-shaped) to improve the anchorage between them and the surrounding matrix.

Maximum fiber content
The maximum fiber content is the critical fiber dosage at which the compactability drastically decreases. The size, the shape and the content of the coarse aggregates as well as the geometry and the volume fraction of steel fibers affect the workability of concrete. The relative fiber to coarse aggregate volume and the ‘balling up’ phenomenon govern the maximum possible content of steel fibers.
Maximum Fiber Content

more fibers the mixture contains the more likely the occurrence of fiber balling; a maximum of 2 Vol.-% of steel fibers (1 Vol.-% at a high aspect ratio) is considered as a maximum. Narayanan & Kareem-Palanjian found that the 'optimum fiber content' increased at increasing percentage sand of total aggregate; both parameters were linearly correlated. The 'optimum fiber content' was defined as the content of the steel fibers beyond which fiber balling took place. SCFRC can maintain self-compactability in spite of the addition of fibers. Groth & Nemegeer performed parameter-studies on the effect of steel fibers on the characteristics of SCC in the fresh state (slump flow, flow-time T50 and J-ring test). Khayat & Roussel tested SCFRC with the IBB-Rheometer; the applied fiber contents were 0.5 and 1.0 Vol.-%. Higher contents of short steel fibers have been added to self-consolidating mortar.

Passing ability

Passing ability is the ability of the concrete to pass narrow gaps between reinforcement bars without blocking. Passing ability of SCFRC is required to guarantee a homogeneous distribution of the components during the flow in the vicinity of obstacles. The minimum bar distance to avoid blocking depends on the flowability of the concrete, on the maximum aggregate size, the paste content and the distribution and the shape of the aggregates. The size of the components of SCFRC relative to the bar spacing affects the passing ability; the fibers usually are the largest components.

The passing ability of SCC can be predicted by calculating the risk of blocking\(^{13,14}\), which takes into account the size, the shape (nature or crushed) and the distribution of the aggregates. The risk of blocking calculated with the CBI-model\(^ {14}\) is used as an input parameter to predict the minimum bar spacing for SCC with steel fibers. Figure 1 shows the relative effect of the aggregates on the passing ability of SCC (\(n_{abi}\)) related to the ratio of the bar spacing \(c\) divided by the diameter of the equivalent aggregate particle diameter \(D_{af}\). The bar spacing \(c\) is the opening between the rebars of the L-box, which is a test method to determine the passing ability of SCC\(^{15}\). The equivalent diameter \(D_{af}\) of an aggregate fraction can be calculated with Equation 1.

\[
\text{Fraction diameter: } D_{af} = \frac{M_i}{2} + \frac{3}{4} (M_i - M_{i1}) \tag{1}
\]

in which

- \(M_i\) upper sieve dimension of aggregate [mm]
- \(M_{i1}\) lower sieve dimension of aggregate [mm]; 1 inch = 25.40 mm

The points of Figure 1 were obtained by an inverse analysis performed with experimental results and can be applied to calculate the risk of blocking according to Equation 2; blocking occurred in the experiments when the risk of blocking was higher than 1. Similar to the Miner-rule on fatigue, the contribution of each aggregate fraction is accumulated with Equation 2. The 'risk of blocking' has to be smaller than or equal to one.\(^ {14}\)

\[
\text{Risk of blocking: } \sum_{i=1}^{n} \frac{n_{abi}}{n_{abi}} = \sum_{i=1}^{n} \frac{V_{abi}}{V_{abi}} \leq 1 \tag{2}
\]

in which

- \(n_{abi}\) aggregate contribution of group i to blocking [-]
- \(n_{abi}\) blocking volume ratio of group i [-]; \(n_{abi} = \frac{V_{abi}}{V_t}\)
- \(V_t\) total volume of the concrete mix [m\(^3\)]
- \(V_{abi}\) aggregate volume of group i [m\(^3\)]
- \(V_{abi}\) blocking volume of aggregate group i [m\(^3\)]

Filling and passing ability are interrelated. Tests on the slump flow and the blocking ratio (L-Box) were carried out in the European ‘Testing SCC’-project.\(^ {16}\) The blocking ratio increased at a given risk of blocking at increasing slump flow. Nemegeer\(^ {17}\) reported on a study on the passing ability of SCFRC; the J-ring in combination with the slump flow test was used to determine the bar spacing required to avoid blocking. A bar spacing of about two times the fiber length was found to prevent blocking. Groth\(^ {18}\)
proposed a guideline (Table 1) for hooked-end steel fibers with circular cross-sections to counteract blocking of SCFRC, which is independent of the mixture composition of SCFRC. Groth normalized the bar spacing (bar spacing divided by the fiber length) to take into account the effect of different types of the steel fibers on the passing ability.

EXPERIMENTAL SET UP

Sixteen SCCs were composed as a reference to study the effect of the type and the content of the steel fibers. The maximum aggregate size was 4 mm/0.16 inch, 8 mm/0.31 inch or 16 mm/0.63 in. The applied aggregates of the mixtures of Series 1-16 were round and are assumed to be ‘natural’ aggregates according to Figure 1. The mixtures also differed in the content and the composition of the paste and the distribution of the aggregates. Table 2 shows the mixture compositions (in Vol.-%) and the characteristics in the fresh state (slump flow, T50 and filling degree Kajima test). The applied test methods for SCC are described in detail by Bartos et al. The air content of mixtures 1-13 was assumed for the mix design to be 2.0 Vol.-%. The target slump flow of mixtures 1-13 was 710 mm/27.95 inch ± 20 mm/0.79 inch, whereas it was a mortar flow of 245 mm/9.65 inch ± 20 mm/0.79 inch for mixtures with a maximum aggregate size of 4 mm/0.16 inch. The filling degree (Kajima test) of the reference mixtures was higher than 90% in each case. The Kajima test was not carried out for mixtures 14-16, since the prepared volume of concrete was not sufficient. Due to the smaller aggregate size and the high filling ability a very high filling degree can be assumed.

Fiber types with lengths between 6 mm/0.24 inch and 60 mm/2.36 inch (mixtures 1-13: 20-60 mm (0.79-2.36 inch), mixtures 14-16: 6-13 mm (0.24-0.51 inch)) were tested. The content of the fibers was varied in order to determine the maximum fiber content and to determine the passing ability of SCFRC. The fiber contents were increased stepwise, which allowed studying the change in flow patterns at increasing fiber content. The volume of the aggregates was decreased by the fiber volume compared to the reference mixture (Series 1-4: against coarse aggregates, other series: the ratio sand to coarse aggregate was kept constant). Table 3 summarizes the tested fiber amounts and the maximum fiber contents of different reference mixtures (Dramix: D, Eurosteel: E, Harex: H). The first fiber index is the aspect ratio (L/d); the second index is the fiber length. The study consisted of 121 mixtures (including 16 mixtures without fibres). The effect of filling ability on the prediction of passing ability of SCFRC is considered small in this study; the slump flow of all self-consolidating mixtures with steel fibers (Series 1-13) was in a small range of 600-660 mm (23.62-25.98 inch).

The mixtures in parenthesis (Table 3) were not self-consolidating; to be considered ‘self-consolidating’ SCFRC had to meet the following criteria:

- Slump flow > 600 mm (23.62 inch)
- No segregation of the fibers
  - judged by their distribution in the mixer
- Homogeneous distribution of SCFRC on the flow table
  - uniform distribution of the fibers/aggregates/cement paste
- a round shape of the flow spread

A forced pan type of mixer (Zyklos) with a maximum capacity of 120 liters/0.157 yd³ was used (Series 1-13). The volume of a batch with fibers was kept constant at 40 liters/0.052 yd³. A smaller forced pan type of mixer (Zyklos) with a maximum capacity of 25 liters/0.033 yd³ was used to prepare mixtures of Series 14-16. The volume of each batch was 16 liters/0.021 yd³. Figure 2 shows the mixing procedure.

Besides the slump flow test, the slump flow test with J-ring was carried out to evaluate the passing ability of SCFRC. A J-ring with seventy-two drilled holes was used to vary the distance of the bars. The diameter of the J-ring was 300 mm/11.81 inch (distance of the center of the holes). The applied bars (d=16 mm/0.63 inch) were smooth and made of stainless steel. Blocking is defined to occur in case the difference of the heights of the concrete in- and outside the J-ring was more than 10 mm/0.39 inch. The height difference was measured at four positions (rotation of 90°); the smallest and the largest differences were omitted. The bar spacing of the J-ring was varied three times in order to obtain one
Maximum Fiber Content

height difference above and one below the blocking criterion (blocking: \(\Delta h > 10 \text{ mm/0.39 inch}\)). The applied bar spacings of the J-ring were 36, 49, 62, 74, 87, 99, 111 and 122 mm (1.42, 1.93, 2.44, 2.91, 3.43, 3.90, 4.37, and 4.80 inch).

MAXIMUM FIBER CONTENT

Up to a defined fiber content, which depends on the fiber type and the mixture composition, a homogeneous distribution of the fibers was observed. However, the steel fibers affect the filling ability of SCC. The more fibers are added and the higher their aspect ratio, the more the slump flow decreased compared to a reference SCC without fibers. Dependent on the type and the content of the fibers, the slump flow decreases and the flow-time T50 increases compared to plain SCC. The fiber factor \(V_f \cdot \frac{L_f}{d_f}\) is the product of the volume of the steel fibers times the aspect ratio and can be applied to compare the effect of different types and contents of the steel fibers on the slump flow. Three deviating flow patterns were observed when the fiber dosage exceeded the “maximum fiber content.” The types of flow pattern (A, B and C) depend on the fiber type and are shown by Figures 3-5.

- **Flow pattern A**: Fiber types having a large surface area have a pronounced effect on the filling ability of SCC. The fibers are homogeneously distributed but the shape of the concrete flow is not round. This flow pattern often coincided with a flow spread smaller than 600 mm/23.62 inch (example: Dramix 80/30 BP).
- **Flow pattern B**: This flow pattern can be observed in two cases: First, the mixture is not stable, the fibers segregate or second, the maximum content of long fibers is surpassed. In the latter case, this flow pattern was observed even when the fibers did not segregate in the mixer. This flow pattern coincided with a larger slump flow, since the flow was barely obstructed by the fibers (Example: Dramix 80/60 BN).
- **Flow pattern C**: Fiber types of intermediate aspect ratios \((L_f/d_f: 45-65)\) often showed a combination of flow patterns A and B. The free flow is obstructed and a cluster of fibers and/or aggregates remains in the centre of the flow table (Example: Dramix 65/40 BN).

The ‘maximum fiber content’ is defined to be the highest possible amount of steel fibers, which can be added to SCC; SCFRC is self-consolidating below this fiber dosage. Below the maximum fiber content, the effect of different types and contents of the steel fibres can be described by the fiber factor \(V_f \cdot \frac{L_f}{d_f}\). For example, the fiber factor is 1.6 with an aspect ratio of 80 and 2 Vol-% steel fibers \((80 \cdot 0.02 = 1.6)\). The design of SCFRC at high fiber dosages requires to compensate for the effect of the fibers while keeping the fibers homogeneously distributed. High filling ability and sufficient segregation resistance often are irreconcilable demands for SCC, which only leaves a small range within which SCFRC is self-consolidating. At a very high filling ability the fibers might segregate. At too low a filling ability already low fiber contents counteract the flow. In this study, the reference mixtures had an intermediate filling ability (mixtures 1-13: slump flow ± 710 mm/27.95 inch), which balanced the contrary demands for SCFRC. The degree to which the slump flow spread is affected might range from almost no effect compared to the reference mixture (flow pattern B) to a significant decrease of the slump flow (flow pattern A). Figures 3-5 can be applied to decide whether SCC with fibers is self-consolidating or not.

In order to predict the maximum content of the steel fibers, the characteristics of the aggregates have to be taken into account. The bar spacing to aggregate fraction diameter ratio \((c/D)\) of the ‘risk of blocking’ approach (Fig. 1) was replaced by the ratio of the fiber length to aggregate fraction diameter \((L_f/D)\) in order to take into account the relative size of aggregate fraction \(i\). The influence factor \(n_{a,mfi}\) of the CBI-model was replaced by the MFC-ratio \(n_{a,mfi}\): MFC stands for the maximum fiber content (the MFC-ratio describes the relative effect \(n_{a,mfi}\) of aggregates on the maximum fiber content). Similar with the risk of blocking of the CBI-model, the MFC-volume can be calculated with Equation 2. The MFC-volume is the sum for all aggregate fractions of the volume of each aggregate fraction \(i\) \((V)\) divided by the MFC-ratio \((V_{a,mfi})\), which is equal to the influence factor \(n_{a,mfi}\). Figure 6 shows the relation between the ratio fiber length to aggregate diameter and \(n_{a,mfi}\). The characteristic points of \(n_{a,mfi}\) (Fig. 6) were varied to obtain the best correlation of experimental results and the maximum fiber factor. Relatively large
aggregates \((L_f/D_{af} < 1.8)\) result in a lower MFC-ratio; \(n_{\text{a,mfi}}\) is approximately constant at higher ratios of \(L_f/D_{af}\).

Figure 7 compares the maximum fiber factor of mixtures of different series with the maximum fiber content volume (MFC) of the aggregates. The fiber length is the weighted average in case a hybrid mixture of short fibers (6 mm/0.24 inch and 13 mm/0.51 inch) was added. The maximum fiber content \((V_fL_f/d_f)\) can be determined with the MFC-volume as the intersection of the regression line (Equation 3). SCC is self-consolidating at a given fiber factor in case the MFC-volume is equal to or lower than the corresponding value on the regression line of Figure 7.

\[
\text{Maximum fiber factor} = \frac{0.781 - \text{MFC}}{0.211} \quad (3)
\]

in which

- \(\text{MFC}\) is the maximum fiber content volume [-]

The analysis was performed with experimental results of mixtures with steel fibers and fiber lengths in the range of 6 mm/0.24 inch to 60 mm/2.36 inch. The mixture which was optimized for the production of slender prestressed sheet piles\(^2\), is also included in Figure 7; its maximum aggregate size was 1 mm. The free spacing between the mould and the strands of the sheet piles was 16 mm/0.63 inch; 125 kg/m\(^3\)/210.8 lb/yd\(^3\) of Dramix OL 13/0.16 steel fibers \((L_f=13 \text{ mm}/0.51 \text{ inch})\) were added. In order to increase the maximum fiber content the following parameters might be altered: to apply fibers with a lower aspect ratio, to increase the content of cement paste and/or to replace coarse with finer aggregates. Once aggregate fractions with a ratio \(L_f/D_{af}\) smaller than 1.8 are excluded, the paste content becomes the governing parameter. The model on the maximum fiber content was calibrated with mixtures that contained round aggregates. A single mixture was tested, which contained crushed aggregates. The content of the aggregates was the same as for mixture 8 (Table 2); crushed instead of round coarse aggregates were applied. In spite of the elongated shape and a rough surface of the coarse aggregates the mixture was self-consolidating. An upper limit (the fibers cluster no matter what the composition of SCC is) fiber content was found for Dramix 80/60 BP; 80 kg/m\(^3\)/134.9 lb/yd\(^3\) of this fiber type caused fiber clustering without any interaction of the aggregates. The maximum fiber content depends on the mixture composition, whereas the upper limit is a characteristic of the fiber type.

**PASSING ABILITY OF SCFRC**

The minimum bar spacing required to avoid blocking is essential information to optimize the mixture composition of SCFRC. Table 4 summarizes test results concerning the non-blocking bar spacing for Series 1-4; some mixtures (with fiber dosages in parenthesis) were not self-consolidating. Only mixtures were considered that were self-consolidating according to the criteria presented before, which excludes the effect of fiber clustering on the passing ability. The experiments showed that the presence of the fibers increased the bar spacing for non-blocking compared to the reference SCC. The type and the content of the fibers mainly determine the required bar spacing. However, different bar spacings for non-blocking were found for different reference mixtures in spite of the same fiber type and fiber content. The mixture composition (the paste content, the distribution and the maximum size of the aggregates) also determines the required bar spacing. The bar spacing required to avoid blocking of mixtures with long steel fibers might be 99 mm/3.90 inch or larger. According to Table 1, the bar spacing for 30 kg/m\(^3\)/50.6 lb/yd\(^3\) of 60 mm/2.36 inch steel fibers has to be 180 mm/7.09 inch. The bar spacings found in the tests for 60 mm fibers were smaller. For example, the required bar spacing at a fiber content of 40 kg/m\(^3\)/67.5 lb/yd\(^3\) \((L_f=60 \text{ mm}/2.36 \text{ inch})\) was 87 mm/3.43 inch (Table 4: Mixture 2). The blocking factor (Equation 4) was calculated and is the product of three parameters that affect the passing ability of SCFRC. These parameters are the risk of blocking (Equation 2), the fiber content and the blocking diameter (Equation 5).
The higher the risk of blocking (effect of the aggregates) the larger the required bar spacing has to be to counteract blocking for a given type and content of the steel fibers. Higher contents of sand and paste were applied for SCFRC in order to provide 'space' for the fibers. The use of round aggregates also contributed to a low risk of blocking, which was typically lower than 0.90. The parameter BD (Equation 5 relates the dimensionless parameter BD and df) was applied to characterize different fiber types having the same length and their effect on the passing ability. The effect of the fibers on the passing ability increased at decreasing diameter. Figure 8 shows the relation between the diameter of the steel fibers and the factor BD.

\[
\text{Blocking diameter (BD)} = 0.553 \cdot df^{-1.51}
\]

In order to predict the 'non-blocking' bar spacing of SCFRC, the free opening c of the J-ring was normalized to the fiber length. A good correlation between the normalized bar spacing and the blocking factor was obtained (linear regression line: \( R^2 = 0.89 \)). The actual bar spacing for 'blocking' is not exactly known since the bar spacing of the J-ring had to be increased in predefined steps. It is between the bar spacing for 'blocking' and 'non-blocking'. The predictions are on the safe side in case none of the data points is lower than Equation 6 predicts. The models' line was shifted parallel to the original regression line; Equation 6 was obtained as the intersection with the data point at the largest distance from the regression line.

\[
\text{BF} = 0.607 \cdot \frac{c}{L_f} - 0.857
\]

\( BF \) blocking factor of SCFRC [-]

\( c \) clear spacing between reinforcement [mm]

\( L_f \) fiber length [mm]; 1 inch = 25.40 mm

Equation 6 was also applied to analyze the results of Series 14-16. Figure 10 summarizes experimental results and predictions of Series 1-16. A correction (0.25 times the blocking factor) had to be applied to obtain results for Series 14-16, which were comparable with the results of mixtures containing longer steel fibres (\( L_f \geq 20 \text{ mm} \)). The effect of very short steel fibres on the passing ability was relatively small (a reduction factor of 0.25 indicates a small influence on passing ability). Blocking was measured in one case only, when non-blocking was predicted.
The optimization of the mixture composition with the normalized bar spacing and the blocking factor is an iterative process, since the risk of blocking ($ROB_{(BS)}$) depends on the actual bar spacing (Equation 2). The Blocking-Factor approach can be applied in two cases: First, in case the design bar spacing is known, the blocking factor can be calculated under consideration of the chosen fiber length (Equation 4). The fiber diameter can be calculated from the aspect ratio. The remaining parameters, the fiber volume and the ROB ‘Risk of Blocking’ (Equation 2), might be chosen. The fiber content has to remain below the maximum fiber content (Equation 3). The content, the shape and the grading of the aggregates determine the ROB at this bar spacing. The actual blocking factor has to be smaller than the predicted blocking factor. Second, the model can be applied to calculate the bar spacing required for non-blocking. The blocking factor has to be determined from the mixture composition. In order to calculate the ROB, the required bar spacing needs to be estimated. A comparison between the estimated and calculated bar spacings is necessary to control the assumed bar spacing.

**FULL-SCALE APPLICATION: SHEET-PILE**

Flowable mixtures are preferred in production in order to minimize the effort to compact the concrete. The applicability of SCFRC in practice and the influence of the casting method was determined in a case-study with sheet piles (developed by SPANBETON, a Dutch producer of prefabricated concrete elements). Figure 11 shows the innovative sheet pile in comparison with a conventional concrete sheet pile; the length of the sheet piles is 12.5 m/41 feet. Sheet piles are ground- and water-retaining structures, which are usually made of steel. Prestressed sheet piles produced with SCFRC have several advantages compared to a standard concrete sheet pile: the placement of the concrete is easier (no rebars have to be placed), the storage of the segments requires less space, more sheet piles can be transported with one truck (they are lighter and occupy less space) and the placement in the ground is easier (less resistance).

With the earlier discussed mix design tools an optimized mixture was developed at the Delft University of Technology that fulfilled all design criteria, i.e. a 1-day compressive strength of at least 60 MPa/8700 psi (28 days: C90/105). The mixture was self-consolidating and contained 125 kg/m³/210.8 lb/yd³ Dramix OL13/0.16 ($L_f=13$ mm/0.51 inch, $d_f=0.16$ mm/0.0063 inch) steel fibers. The free opening between the strands and the mould was 16 mm/0.63 inch. Table 5 shows the mixture composition; the volume of the superplasticizer is subtracted from the content of free water. The applied mixture is more expensive than the conventional concrete; due to the more slender shape of the sheet pile (less material) and other savings (production, placement of bar reinforcement, transport and faster construction) these costs are compensated. The fibers oriented along the flow due to the effect of the walls and the strands. Especially in the lower flanges the flow distance was rather long. The fibers oriented along the flow in the lower flanges, whereas their orientation was more random in the upper flange. The performance of the sheet piles in the hardened state was according to the expectations, taking into account the orientation of the steel fibers and their effect on the tensile strength.

**CONCLUSIONS**

SCFRC is an innovative type of concrete, which combines the advantages and extends the possibilities of both SCC and fiber reinforced concrete. Two tools are discussed that facilitate the mix design of SCFRC concerning the maximum fiber content and the passing ability. An application (sheet piles) with SCFRC is discussed that shows the applicability of the tools. Based on the experimental studies and the analysis of the behavior of SCFRC the following conclusions can be drawn:

- Steel fibers affect the key characteristics filling ability and passing ability of SCC. The applicable range of mixture compositions and characteristics in the fresh state of SCC containing high fiber contents (close to the maximum fiber content) is smaller compared to SCC. By optimizing the mixture composition and balancing filling ability and segregation resistance, SCC can be self-consolidating at relative high fiber contents.
• The effect of the fibers on the maximum fiber content and the passing ability can be predicted with the type and the content of the fibers and the characteristics of the aggregates as input parameters. Visual observations on the slump flow test are proposed to provide information concerning the maximum fiber content.
• Due to an optimized mixture composition of SCFRC the fibers are homogeneously distributed in a structural element (without blocking) and effectively embedded in the matrix (without clustering).

ACKNOWLEDGEMENTS

The Dutch Technology Foundation STW and the Priority Program Materials (PPM)—research program ‘Cement-Based Materials’ (grant number 4010 III)—funded the project. They are gratefully acknowledged for their support.

REFERENCES

24 S. Grünwald and J. C. Walraven


Table 1—Recommendation on the normalized bar spacing to avoid blocking of SCFRC (fiber dosage: \(m_f\))

<table>
<thead>
<tr>
<th>(c/L_r)</th>
<th>(L/d_f)</th>
<th>(Max. m_f) [kg/m³]</th>
<th>(Max. m_f) [lb/yd³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>≥ 3</td>
<td>80</td>
<td>30</td>
<td>50.59</td>
</tr>
<tr>
<td></td>
<td>65</td>
<td>60</td>
<td>101.18</td>
</tr>
<tr>
<td>≥ 2</td>
<td>65</td>
<td>30</td>
<td>50.59</td>
</tr>
<tr>
<td></td>
<td>45</td>
<td>60</td>
<td>101.18</td>
</tr>
<tr>
<td>≥ 1.5</td>
<td>45</td>
<td>30</td>
<td>50.59</td>
</tr>
</tbody>
</table>

Note: 1 mm = 0.0394 inch

Table 2—Mixture composition and characteristics of sixteen self-consolidating reference mixtures

<table>
<thead>
<tr>
<th>No.</th>
<th>Max. aggregate size</th>
<th>Fine aggregates (0.125-4 mm)</th>
<th>Coarse aggregates (4-16 mm)</th>
<th>Paste content (&lt;0.125 mm)</th>
<th>Slump flow/Mortar flow</th>
<th>Flow-time T50</th>
<th>Filling vessel test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>16</td>
<td>23.40</td>
<td>39.50</td>
<td>35.10</td>
<td>718</td>
<td>2.4</td>
<td>91.1</td>
</tr>
<tr>
<td>2</td>
<td>16</td>
<td>24.59</td>
<td>36.52</td>
<td>36.89</td>
<td>708</td>
<td>3.3</td>
<td>97.6</td>
</tr>
<tr>
<td>3</td>
<td>16</td>
<td>25.95</td>
<td>33.13</td>
<td>38.92</td>
<td>716</td>
<td>4.2</td>
<td>97.7</td>
</tr>
<tr>
<td>4</td>
<td>16</td>
<td>27.31</td>
<td>29.72</td>
<td>40.97</td>
<td>743</td>
<td>3.5</td>
<td>98.9</td>
</tr>
<tr>
<td>5</td>
<td>16</td>
<td>35.10</td>
<td>26.40</td>
<td>36.50</td>
<td>703</td>
<td>1.9</td>
<td>93.8</td>
</tr>
<tr>
<td>6</td>
<td>16</td>
<td>33.63</td>
<td>25.37</td>
<td>39.00</td>
<td>693</td>
<td>2.1</td>
<td>90.4</td>
</tr>
<tr>
<td>7</td>
<td>16</td>
<td>41.82</td>
<td>19.68</td>
<td>36.50</td>
<td>730</td>
<td>2.1</td>
<td>93.0</td>
</tr>
<tr>
<td>8</td>
<td>16</td>
<td>40.12</td>
<td>18.88</td>
<td>39.00</td>
<td>725</td>
<td>2.6</td>
<td>93.9</td>
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<td>16</td>
<td>43.52</td>
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<td>95.8</td>
</tr>
<tr>
<td>10</td>
<td>16</td>
<td>41.82</td>
<td>19.68</td>
<td>36.50</td>
<td>728</td>
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<td>98.8</td>
</tr>
<tr>
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<td>16</td>
<td>40.12</td>
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<td>39.00</td>
<td>720</td>
<td>2.7</td>
<td>97.8</td>
</tr>
<tr>
<td>12</td>
<td>8</td>
<td>42.21</td>
<td>18.09</td>
<td>37.70</td>
<td>690</td>
<td>2.0</td>
<td>91.3</td>
</tr>
<tr>
<td>13</td>
<td>8</td>
<td>40.60</td>
<td>17.40</td>
<td>40.00</td>
<td>693</td>
<td>2.4</td>
<td>91.3</td>
</tr>
<tr>
<td>14</td>
<td>4</td>
<td>40.0</td>
<td>—</td>
<td>60.0</td>
<td>830/265</td>
<td>1.2</td>
<td>—</td>
</tr>
<tr>
<td>15</td>
<td>4</td>
<td>50.0</td>
<td>—</td>
<td>50.0</td>
<td>865/266</td>
<td>1.6</td>
<td>—</td>
</tr>
<tr>
<td>16</td>
<td>4</td>
<td>55.0</td>
<td>—</td>
<td>45.0</td>
<td>800/244</td>
<td>2.0</td>
<td>—</td>
</tr>
</tbody>
</table>

Note: 1 mm = 0.0394 inch
### Table 3—Applied types and contents of steel fibers

(mixtures 1-13: kg/m³ and lb/yd³; mixtures 14-16: Vol.-%; Lf, in mm)

<table>
<thead>
<tr>
<th>Fiber type (Lf/df; Lf)</th>
<th>D 45/30 BN</th>
<th>E 50/50</th>
<th>D 65/40 BN</th>
<th>D 80/60 BN</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 kg/m³</td>
<td>40/(60/(80)</td>
<td>-</td>
<td>-</td>
<td>20/(40)</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>67.5/(101.2)/134.9</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>2 kg/m³</td>
<td>80/(100)</td>
<td>60/(80)</td>
<td>40/60</td>
<td>40/(60)</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>134.9/(168.6)</td>
<td>101.2/(134.9)</td>
<td>67.5/101.2</td>
<td>67.5/(101.2)</td>
</tr>
<tr>
<td>3 kg/m³</td>
<td>80/(100)/(120)</td>
<td>60/80/(100)</td>
<td>40/60/(80)</td>
<td>40/(60)/(80)</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>134.9/168.6/(202.4)</td>
<td>101.2/134.9/ (168.6)</td>
<td>67.5/101.2/(134.9)</td>
<td>67.5/(101.2)/</td>
</tr>
<tr>
<td>4 kg/m³</td>
<td>100/(120)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>168.6/(202.4)</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5 kg/m³</td>
<td>80/(100)</td>
<td>-</td>
<td>40/(60)</td>
<td>40/(60)</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>134.9/(168.6)</td>
<td>-</td>
<td>67.5/(101.2)</td>
<td>67.5/(101.2)</td>
</tr>
<tr>
<td>6 kg/m³</td>
<td>202.4/(236.1)</td>
<td>-</td>
<td>101.2/(134.9)</td>
<td>67.5/(101.2)</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>120/(140)</td>
<td>-</td>
<td>40/(60)</td>
<td>40/(60)</td>
</tr>
<tr>
<td>7 kg/m³</td>
<td>202.4/(236.1)</td>
<td>-</td>
<td>67.5/(101.2)</td>
<td>67.5/(101.2)</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>120/(140)</td>
<td>-</td>
<td>40/(60)</td>
<td>40/(60)</td>
</tr>
<tr>
<td>8 kg/m³</td>
<td>202.4/236.1/(269.8)</td>
<td>168.6/(236.1)</td>
<td>101.2/(134.9)</td>
<td>101.2/(134.9)</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>100/(120)</td>
<td>-</td>
<td>60/(80)</td>
<td>60/(80)</td>
</tr>
<tr>
<td>9 kg/m³</td>
<td>168.6/(202.4)</td>
<td>-</td>
<td>40/(60)</td>
<td>40/(60)</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>120/(140)</td>
<td>-</td>
<td>67.5/(101.2)</td>
<td>67.5/(101.2)</td>
</tr>
<tr>
<td>10 kg/m³</td>
<td>202.4/(236.1)</td>
<td>-</td>
<td>101.2/(134.9)</td>
<td>67.5/(101.2)</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>120/(140)</td>
<td>-</td>
<td>60/(80)</td>
<td>60/(80)</td>
</tr>
<tr>
<td>11 kg/m³</td>
<td>202.4/(236.1)</td>
<td>134.9/(168.6)</td>
<td>101.2/(134.9)</td>
<td>101.2/(134.9)</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>120/(140)</td>
<td>80/(100)</td>
<td>60/(80)</td>
<td>60/(80)</td>
</tr>
<tr>
<td>12 kg/m³</td>
<td>202.4/(236.1)</td>
<td>-</td>
<td>101.2/(134.9)</td>
<td>67.5/(101.2)</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>120/(140)</td>
<td>20/(40)</td>
<td>-</td>
<td>40/(60)</td>
</tr>
<tr>
<td>13 kg/m³</td>
<td>-</td>
<td>-</td>
<td>33.7/(67.5)</td>
<td>-</td>
</tr>
<tr>
<td>lb/yd³</td>
<td>-</td>
<td>-</td>
<td>40/(60)</td>
<td>40/(60)/(80)</td>
</tr>
<tr>
<td>14</td>
<td>1.0/3.0/4.0/(5.0)</td>
<td>1.5/2.0/(3.0)</td>
<td>(1.5+1.5)</td>
<td>2.0-0.5</td>
</tr>
<tr>
<td>15</td>
<td>3.0/(4.0)</td>
<td>1.0/1.5/(2.0)</td>
<td>(1.5+1.5)</td>
<td>2.0-0.5</td>
</tr>
<tr>
<td>16</td>
<td>1.5/(3.0)</td>
<td>1.0/(1.5)</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

**Fiber type (Lf/df; Lf) D OL6 D OL13 D OL6+D OL13 D OL6+D OL13**

<table>
<thead>
<tr>
<th>Fiber type (Lf/df; Lf)</th>
<th>D OL6</th>
<th>D OL13</th>
<th>D OL6+D OL13</th>
<th>D OL6+D OL13</th>
</tr>
</thead>
<tbody>
<tr>
<td>14</td>
<td>1.0/3.0/4.0/(5.0)</td>
<td>1.5/2.0/(3.0)</td>
<td>(1.5+1.5)</td>
<td>2.0-0.5</td>
</tr>
<tr>
<td>15</td>
<td>3.0/(4.0)</td>
<td>1.0/1.5/(2.0)</td>
<td>(1.5+1.5)</td>
<td>2.0-0.5</td>
</tr>
<tr>
<td>16</td>
<td>1.5/(3.0)</td>
<td>1.0/(1.5)</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
### Table 4—Required bar spacings $c_{NB}$ for non-blocking of Series 1-4 (in mm/inch)

<table>
<thead>
<tr>
<th>Steel fiber content</th>
<th>0</th>
<th>20</th>
<th>40</th>
<th>60</th>
<th>80</th>
<th>100</th>
<th>120</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[kg/m³]</td>
<td>[lb/yd³]</td>
<td>[kg/m³]</td>
<td>[lb/yd³]</td>
<td>[kg/m³]</td>
<td>[lb/yd³]</td>
<td>[kg/m³]</td>
</tr>
<tr>
<td>0</td>
<td>33.7</td>
<td>67.5</td>
<td>101.2</td>
<td>134.9</td>
<td>168.6</td>
<td>202.4</td>
<td></td>
</tr>
</tbody>
</table>

**Mixture 1**
- Dramix - 45/30 RL: 49/1.93
- Dramix - 80/60 RC: 99/3.90

**Mixture 2**
- Dramix - 45/30 RL: 36/1.42
- Eurosteel 50/50: 74/2.91
- Dramix - 65/40 RC: 99/3.90
- Dramix - 80/60 RC: 87/3.43

**Mixture 3**
- Dramix - 45/30 RL: 36/1.42
- Eurosteel 50/50: 74/2.91
- Dramix - 65/40 RC: 87/3.43
- Dramix - 80/60 RC: 62/2.44

**Mixture 4**
- Dramix - 45/30 RL: 36/1.42
- Eurosteel 50/50: 74/2.91
- Dramix - 65/40 RC: 87/3.43
- Dramix - 80/60 RC: 62/2.44

### Table 5—Mixture composition of SCFRC for sheet piles

<table>
<thead>
<tr>
<th>Mixture composition</th>
<th>[kg/m³]</th>
<th>[lb/yd³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>CEM I 52.5 R</td>
<td>358</td>
<td>603,7</td>
</tr>
<tr>
<td>CEM III/A 52.5</td>
<td>555</td>
<td>935,9</td>
</tr>
<tr>
<td>Silica fume</td>
<td>61</td>
<td>102,9</td>
</tr>
<tr>
<td>Sand (0.125-0.5 mm)</td>
<td>549</td>
<td>925,8</td>
</tr>
<tr>
<td>Sand (0.5-1.0 mm)</td>
<td>549</td>
<td>925,8</td>
</tr>
<tr>
<td>Steel fibers (OL13/0.16)</td>
<td>125</td>
<td>210,8</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>21</td>
<td>(35,4)</td>
</tr>
<tr>
<td>Total water</td>
<td>226</td>
<td>381,1</td>
</tr>
<tr>
<td>w/c-ratio</td>
<td>0.25</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Note: 1 mm = 0.0394 inch

---

**Figure 1**—Relation between the ratio of the bar spacing $c$ to fraction diameter $D_{af}$ of the aggregates and the blocking volume ratio $n_{abs}$ (numbers: X/Y coordinates)
Figure 2—Mixing procedure for SCFRC

Figure 3 — Flow pattern A

Figure 4 — Flow pattern B

Figure 5 — Flow pattern C

Figure 6—Relation between the ratio fiber length to aggregate diameter and the MFC-ratio ($n_{a,mfi}$)
Figure 7—Relation between the maximum fiber factor and the MFC-volume

\[ y = -0.211x + 0.781 \]
\[ R^2 = 0.90 \]

Figure 8—Relation between the diameter of the steel fibers and the blocking diameter (BD); 1 mm = 0.0394 inch

\[ BD = 0.553d_f^{1.51} \]
\[ R^2 = 0.96 \]
Figure 9—Determination of the bar spacing sufficient for non-blocking compared to the test results of SCFRC (Series 1-13; $L_i \geq 20$ mm; 1 mm = 0.0394 inch)

Figure 10—Determination of the bar spacing sufficient for non-blocking compared to the test results of SCFRC (Series 1-16; $L_i \geq 6$ mm; 1 mm = 0.0394 inch)
Figure 11—Prestressed sheet piles cast with SCFRC (left: C90/105) and conventional concrete (right: B55/67 concrete with additional bar reinforcement)
Prediction of Stress Development and Cracking in Steel Fiber-Reinforced Self-Compacting Concrete Overlays Due to Restrained Shrinkage

by J. Carlswärd and M. Emborg

Synopsis: Shrinkage cracking of self-compacting concrete (SCC) overlays with and without steel fibres has been assessed through laboratory testing and theoretical analysis. Test results verified that steel fibre reinforcement has a crack width limiting effect. However, the contribution in case of fibre contents up to 0.75 vol% was not found to be sufficient to distribute cracks in situations where bond to the substrate was nonexistent. Thus, even higher steel fibre contents (or other types of fibres) are required in order to control cracks.

A distributed pattern of fine cracks was however obtained even for unreinforced SCC within bonded areas of the overlays. This implies that steel fibres, or other crack reinforcement, are not required if high bond strength is obtained.

An analytical model, proposed to assess the risk of cracking and to predict crack widths in overlays, was found to give reasonable correlation with experimental results.

Keywords: bond; cracks; overlay; restraint; SCC; shrinkage; steel fibers; theoretical model.
INTRODUCTION

Bonded overlays on concrete substrates are applied in a wide range of applications such as for repairs or strengthening of deteriorated bridge and parking decks, damaged industrial floors or as finishing layers on prefabricated elements. In order to ensure that the overlayed system remains durable and fully functioning during the intended service life it is essential to limit crack widths and to prevent delamination.

Many researchers and building clients have pointed out that the single most important factor determining the service life of an overlayed structure is the shrinkage of the newly cast overlay (Granju et al. 2004; Rahman et al. 2000; Weiss et al. 1998; Yuan & Cai 2002). Cracking, delamination and edge lifting (Fig. 1) are typical consequences of the shrinkage, or rather the difference in shrinkage between the overlay and substructure.

Two measures may be undertaken to minimise the negative effects of restrained shrinkage: (1) reduce the shrinkage of the overlay concrete and/or (2) provide reinforcement to distribute and control cracks. Shrinkage reduction may be accomplished for example by adding Shrinkage Reducing Admixture (SRA) to the mix. The effect of SRA has been studied quite extensively over the last 15-20 years (Ohama et al. 1988; Shah et al. 1992; Grzybowski & Ohama 1996; Weiss & Shah 2002; Petersson 2007) and the technique may now be regarded as an accepted method to control concrete shrinkage.

Reinforcement is provided either by welded mesh or steel bars or by mixing fibres into the concrete matrix. An attractive feature of fibre reinforced concrete (FRC) is of course that the demanding handling of traditional reinforcement is eliminated. The improvement in working environment, and possibility to reduce labour intensity, is even more accentuated when fibres are combined with self-compacting concrete (SCC). Experience from real overlay castings has further demonstrated that the use of SCC leads to improved surface finish with regard to flatness demands, thus reducing the need for expensive levelling screeds (Peterson 2008). These factors certainly give the contractor incentives to select FRSCC, both from a productivity perspective as well as from a working environmental and economical point of view.

Despite the fact that fibres, mainly steel, are used on a regular basis to control cracking there is no method available to design overlays, and other structures, on this regard. This means that the steel fibre addition is often selected/designed based on recommendations relying on experience. Several experience based design proposals are available according to Granju & Turatsinze 2005. Although such methods occasionally may prove to result in “crack-free” overlays it is obvious that more reliable approaches need to be developed for the future use of SFRC and SFRSCC in overlays.

Thus, main incentives of the research presented in the article are: (1) to experimentally verify the effect of steel fibres on widths and distribution of shrinkage cracks in thin overlays and (2) to establish a theoretical model for the estimation of crack risk due to restrained shrinkage and to assess the contribution of steel fibres on crack widths.

The work presented in the article summarises previous studies conducted within a doctoral thesis (Carlswärd 2006).

RESEARCH SIGNIFICANCE

The technique of applying thin bonded concrete overlays is widely adopted in the construction industry. Examples of applications are repairs and levelling of floors. Experience shows that cracking
and debonding, caused by the difference in shrinkage between the new overlay and the often old and rigid substructure, are frequent concerns. Despite this fact there are still no reliable design methods or recommendations available to ensure that such harmful damages can be avoided. It is believed that the work presented in this article will provide some useful guidance in the search for a solution to the cracking and debonding problem.

**TEST METHODS TO ASSESS THE FIBER EFFECT**

Three main categories of test setups are frequently used to study shrinkage cracking: (1) end-restrained, (2) base restrained and (3) ring tests (see Fig. 2). Category (2) tests, overlays cast on a substrate, are clearly the most suitable from the viewpoint that the restraint condition represents the real overlay conditions. A drawback is that it is difficult to control the bond conditions, which means that the cracking response will rely on the particular restraint situation (bond quality) obtained in each test.

From this viewpoint it is favourable to use end-restrained setups or ring tests, as they offer a direct evaluation of the effect of for instance type or amount of fibre. The ring test, in which the shrinkage of an external concrete ring is restrained by an inner steel ring, is certainly the most popular test method. Favourable features are the simplicity of the setup and that the degree of restraint is well defined. A review of test results reported on the ring test method (see Carlswärd 2006) indicates however that the ring test may overestimate the influence of fibres. Multiple cracking is regularly obtained already at steel fibre dosages as low as 0.25 vol%. In fact, more than one crack has even been observed for plain concrete in some investigations (Shah et al. 1992; Petersson 2007; Groth 2000; Bissonnette et al. 2005).

As crack distribution would clearly not take place in unreinforced concrete this leads to speculations regarding the validity of the method for assessing the fibre effect.

Thus it was decided to adopt an end-restrained setup in the present study to assess the effect of steel fibres on cracking potential of SCC. The test setup was inspired by a previous test method proposed in Banthia et al. 1996. A series of base restrained tests were also conducted in order to study the effects of restrained shrinkage more realistically in regard to real overlays.

**End-restrained test series**

A test setup as shown in Fig. 3 was adopted, consisting of an HEA steel beam with a grounded and polished upper flange, giving a smooth and slippery surface to ensure that restraint would only develop at the ends of the concrete specimen. L-shaped supports of steel 80x45 mm (3.15 x 1.77 in.) with 20 mm (0.79 in.) thick legs were bolted to the flange at a distance of 1 m (~40 in.).

A 10 mm (0.39 in.) thick steel platen of 800 mm (31.5 in.) length with oiled and smooth surface was placed at the upper flange to decrease the thickness of the concrete in midsection (100 mm = 4 in. from the end-zones). The ends of the platen were chamfered in order to give a smooth transition between the thick end section and the thinner internal parts of the concrete. The configuration was designed in order to minimize stress concentrations and to ensure that cracking would occur within the internal parts. Furthermore, the platen was divided in two parts by a wedge at mid-section. The wedge was removed the day after casting allowing the steel platen to slide more or less freely underneath the concrete layer. End-restraint was achieved through three 10 mm (0.39 in.) threaded bars at each L-shaped support extending approximately 75 mm (3 in.) into the concrete.

Plastic foil was applied to the concrete surface immediately after casting to allow for a curing period of 24 hours. Testing was initiated by removing the cover, the form along the sides and the wedge in the steel platen underneath the concrete. Furthermore, an air proof tape was put along the concrete sides to prevent drying through the side faces and to ensure one-sided drying, similar to real overlay conditions.

Target points for deformation measurements were glued to the concrete surface at an individual distance of 200 mm (7.9 in.) starting 100 mm (4 in.) from each support, giving four measuring distances along the concrete specimen. Measuring was conducted using a mechanical device of the type Staeger placed in between two successive target points to obtain the deformation.

An experimental program was conducted including reference specimens of plain SCC as well as steel fibre reinforced SCC with fibre contents of 0.25, 0.375 and 0.5 vol% (20 (33.7), 30 (67.4) and 40 (50.5) kg/m³ (lb/yd³)) The basic mix proportions are given in Table 1, corresponding to SCC class C28/35 with maximum aggregate size of 8 mm (0.31 in.). The cement was of the type CEM II/A-L 42.5 R and the sand
was of a natural occurring type. A limestone filler of the type Limus 40 by Nordkalk was added in an amount of 100 kg/m³ (169 lb/yd³) in order to enhance the stability of the SCC. Furthermore, a high-range water-reducing admixture of the type Sikament 56 by Sika, a chemical compound based on polycarboxylate-ether, was used to obtain the target slump flow of 700 mm (27.5 in.). The fibre type was Dramix RC 65/35 BN, an end-hooked steel fibre with length and aspect ratio of 35 mm (1.38 in.) and 65 respectively.

Two test rigs (Fig. 3) were carried out simultaneously for each type of concrete in order to improve the statistical result basis. The rigs were stored in a climate chamber with temperature and relative humidity of 25°C (77°F) and 50% respectively. Additional unrestrained specimens for free shrinkage measurement, see Fig. 4, with the same cross section but slightly shorter, width x depth x length = 100 x 35 x 500 mm (4 x 1.38 x 19.7 in.) were stored horizontally aside the test rigs.

Target points for measuring deformation were glued to the surface immediately after demoulding at 24 hours. Shortly after this a zero-reading was recorded on both restrained and unrestrained specimens.

Material properties – Some material properties measured in the test program are given in Table 2. The compressive strength was determined at an age of 28 days while flexural beam tests were conducted after 28 days. Flexural testing was performed in accordance with ASTM C1018-92. The procedure is still rather common in Sweden despite the fact that it was withdrawn in 2006 as a standard test method. Results obtained from the beam tests are the crack strength \( f_{cr} \) and residual strength factors \( R_{10,x} \) as shown in the table. Residual strength is obtained by multiplying the crack strength with the residual factor.

Free shrinkage – To simulate a real overlay situation the sides and the bottom face of free shrinkage specimens were covered with air proof material. Thus, drying was only permitted through the upper face as illustrated in Fig. 4. A consequence is that a nonlinear shrinkage distribution developed over the depth initially. The curvature resulting from uneven shrinkage distribution is the explanation as to why the bottom face even expands slightly initially, as observed in the diagram. Note that measurements of free shrinkage deformation at the bottom face, carried out by turning the specimens and measuring between target points, were only conducted for the mixes with 0.25 and 0.375 vol% of steel fibres, denoted SFRSCC 20 and 30.

It can be seen that the response of the unrestrained specimens varied somewhat. In particular it may be observed that SFRSCC 40 had a somewhat slower shrinkage development. The variation is most likely related to a slight change in the relative humidity in the climate chamber during this test rather than to effects of the fibre addition.

It may be noted that despite the rather small size of the free shrinkage specimen, the exact free shrinkage is not obtained due to non-even humidity and eigenstresses within the specimen. These effects, that indeed influence the documented strains, will be studied in a newly initiated Scandinavian research project.

Restrained shrinkage – Deformation results obtained at the upper face of restrained specimens of SCC, SFRSCC 20, 30 and 40 (0.25, 0.375 and 0.5 vol% of steel fibres) are shown in Fig. 5 (a)-(d). It should be pointed out that only one crack developed, in other words crack distribution due to the fibre additions was not observed. Thus, crack strain is the strain measured in the zone where the crack developed while strain in uncracked parts corresponds to the mean strain obtained in uncracked parts. Note that results are given for two specimens in each diagram.

Even though steel fibres did not distribute cracks it is clear when comparing the results shown in Fig. 5 (a) with Figs. 5 (b)-(d) that a positive contribution was obtained due to fibres, resulting in a decreased crack strain and also in lower strain in uncracked parts. It can further be seen that the difference between the free shrinkage strain and the strain in uncracked parts was more significant for fibre reinforced mixes as compared to plain SCC. The reason is that the initially restrained specimens are completely relieved in case of unreinforced SCC, Fig. 5 (a), while some restraint remained after cracking in fibre reinforced specimens, Fig. 5 (b)-(d), due to fibres bridging the crack.

Although it may not be exactly correct to make a direct comparison between SCC and SFRSCC mixes as the free shrinkage development was not identical for the different mixes (Fig. 4) the results indicate that addition of 0.25 vol% of steel fibres (b) did not alter the response very much in comparison to plain SCC. Cracks developed at approximately the same time (prior to 5 days after start of drying) and the crack strains were similar in magnitude.

From Fig. 5 (c) and (d) it may be seen that increased fibre amounts resulted in a more significant influence
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on the crack widths. A particularly good response was obtained for SFRSCC 40 (0.5 vol%) where only one of the specimens cracked. It can also be seen when comparing Fig. 5 (d) with (a) that the crack strain was considerably lower and that the difference between strain in uncracked parts and free shrinkage was much higher for the fibre reinforced specimen.

A rather good response was also obtained for one of the specimens of SFRSCC 30 in Fig. 5 (c). The reason as to why the crack width reduction was not as good for the other specimen of the same mix is believed to be due to a lower number of fibres crossing the crack. This was verified by a fibre count, which showed that the number of fibres was 30% higher in the cracked section of the specimen with the “best” response.

It can further be seen that the time to cracking seems to have been prolonged by increased fibre volume, compare the two specimens of SFRSCC 30. The same situation applies for SFRSCC 40, where the average number of fibres crossing a section in the uncracked SFRSCC 40 specimen was approximately 35% higher than in the cracked specimen. Although not observed within the test period it is believed that a crack would have developed in this specimen as well if testing had been prolonged with a few days.

A prolonged time to cracking for increasing fibre volume fractions has been reported in some previous studies (Shah et al. 2003; Shah et al. 2004; Voigt et al. 2004). This is usually believed to be due to fibres bridging and hindering cracks already at the micro stage.

The fact that the free shrinkage response differed somewhat between the tests (see Fig. 4) makes it difficult to assess the effect of steel fibres on crack widths. In order to make a reasonably fair comparison it was decided to select the measured crack width at the time when a free shrinkage value of 0.6 ‰ was recorded. This value was selected as it corresponded to the maximum shrinkage obtained for SFRSCC 40 within the studied period (see diagram in Fig. 4). Crack width results for the situation are given in Fig. 6. As the actual number of fibres crossing the crack did not always correspond with the amount of fibres added to the mix the volume fraction in the crack zone was determined based on the counted number and by using fibre orientation theory as described by Soroushian & Lee 1990 and Löfgren 2005.

With one exception (one of the specimens of SFRSCC 30) it can be seen in Fig. 6 that crack widths decreased with increasing fibre volume fraction. Addition of 0.4-0.5 vol% of steel fibres reduced the crack width by approximately 50% compared to plain SCC.

Examples of results from two studies in which ring tests were used to evaluate the effect of steel fibres on cracks are given for comparison. The fibre type used by Voigt et al. 2004 was an end-hooked fibre with length of 50 mm (2 in.) and an aspect ratio of 50 while a crimped fibre with length 25 mm (1 in.) was used in the study of Shah & Weiss 2006. Noticeable is the substantial effect of fibres that was achieved already at dosages as low as 0.25 vol%. This implies that the ring shape gives a somewhat more favourable stress situation as compared to linear specimens. A thorough discussion on possible reasons for the difference can be found in Carlswärd 2006.

Base restrained test series

The base restrained tests consisted of out-stretched overlay strips (50 x 150 x 2500 mm = 2.0 x 5.9 x 98 in.) cast on large concrete bottom slabs (300 x 2000 x 3000 mm = 118 x 79 x 12 in.), see Fig. 7 (a). Four slabs were produced approximately a year in advance to minimise the remaining shrinkage, in order to maximise the differential shrinkage between overlays and slab. Different substrate preparations were applied in order to achieve a variation in bond quality. One of the slabs was grinded and dry at the time of overlaying, one was grinded and wet, one was milled and wet and the last slab was milled and primed with MD 16 from Maxit. Grinding procedure gave a rather smooth texture while milling resulted in a rough texture, see Fig. 7 (b).

The ends of the overlay strips were fastened to the slabs by means of vertical expander bolts in order to avoid complete delamination.

Ten overlay strips were cast on each slab; two strips of plain SCC, two steel bar reinforced strips SBRSCC (a centrally placed 10 mm (0.4 in.) steel bar), two strips with shrinkage reduced SCC (SRASCC with 1.5 % SRA type Sika Control 40) and four SFRSCC strips (two with 0.375 vol% (30 kg/m³ or 50.5 lb/yd³) of fibres and two with 0.75 vol% (60 kg/m³ or 101 lb/yd³)). The steel fibre type was the same as in the end-restrained series (Dramix RC-65/35-BN).

The same basic concrete recipe and materials as for the end-restrained test series was used for all mixes (Table 1), corresponding to SCC of class C28/35 $D_{max}$ 8 mm (0.31 in.) and a w/c-ratio of 0.58 with a
target slump flow of 700 mm (27.6 in.).

The overlays were cured underneath plastic coverage for five days before exposed to a rather harsh laboratory environment of 15-20% relative humidity and 20°C (68°F) giving a rapid drying shrinkage development and associated overlay cracking within a rather short period of time (prior to 30 days). Cracking and debonding were followed for a period of approximately 3 months.

**Material properties and free shrinkage** – The SCC mixes produced in the base restrained test series were only evaluated with respect to the compressive strength. A more thorough analysis was conducted within the end-restrained series (see Table 2) on identical mixes, although not exactly the same fibre amounts. Results from compressive strength tests gave similar results as in the previous tests, ranging from 35.5 MPa (5149 psi) for SRASCC to 39.5 MPa (5729 psi) for SFRSCC 60.

Free shrinkage was obtained in the same way as for the end-restrained test series described previously, from thin concrete prisms sealed at the bottom and along the sides to allow drying only through the upper face (Fig. 8). A thickness of 50 mm (2 in.) was adopted in order to simulate the overlay strips. Deformations were measured at the bottom and upper faces of the specimens, using a handheld mechanical measuring device, to capture the shrinkage gradient. The prisms were placed aside the overlays to evaluate the unrestrained deformation under identical drying conditions.

Measured free shrinkage development is shown in Fig. 8. Noticeable is the extensive difference in strain between the top and bottom faces of the specimens, resulting from the one sided drying conditions. It can further be seen that the addition of SRA reduced the average shrinkage by approximately 25%. There were no differences between fibre reinforced and plain SCC, which verifies that the addition of fibres did not influence the shrinkage.

**Cracking and debonding** – The first visible cracks were observed between 1 and 3 weeks after the start of drying in most of the overlay strips and new cracks were then established until the end of the test period 3-4 months later. From the crack width summary given in Fig. 9 it can be seen that most of the cracks had a width of only between 0.05 and 0.25 mm (0.002-0.01 in.) at the end of the measuring period while a few cracks with more significant widths developed in overlay strips of slabs 2, 3 and 4.

The reason as to why none of the cracks observed in overlays on slab 1 had a width that exceeded 0.15 mm (0.06 in.) is that the bond strength obtained was sufficiently high and evenly distributed to prevent debonding. The result was somewhat unexpected as a smooth and dry substrate is generally believed to be unfavourable from a bond perspective. For slabs 2-4 the bond situation was found to be considerably more varying, ranging from nonexistent to adequately high bond to prevent debonding. This was surprising as the preparations of the substrates of these slabs were intended to provide better bond as compared with slab 1. It is not fully clarified as to why the opposite situation occurred.

Major cracks always coincided with the development of a major, internal debonded zone. A crack mapping of the overlays on slab 3 is given as an example in Fig. 10. For overlays with a major internal debonded zone (shaded area) a single crack developed within this zone for SCC, SRASCC and SFRSCC 30 and 60 (see SCC II, SRASCC II, SFRSCC 30 II and SFRSCC 60 II). This indicates that distribution of cracks, and corresponding crack control, was not achieved for the SFRSCC applied in the tests. It can however be seen that the crack in strip SFRSCC 60 II was only 0.2 mm and did not extend all the way through the width. This implies that the addition of 0.75 vol% of steel fibres was quite effective. For SBRSCC II an additional crack developed within the major debonded zone, which proves that the centrally placed steel bar (Ø10 mm = 0.4 in.) did provide some crack distribution.

The reason as to why sufficient bond strength seems to have been obtained for half of the strips (strips denoted I) while the remaining 5 strips (denoted II) seem to have been poorly bonded is unknown. Exactly the same preparation was applied and the strips were cast at the same occasion and cured in the same way. Furthermore, concrete from the same batch was used for the two strips of the same mix, for example SCC I and II. This clearly gives an indication of the sensitiveness of the bond mechanism. The result also clarifies the difficulty in securing high and even bond in real overlay situations.

**Effect of steel fibres on cracks** – Based on the results presented above it can be concluded that steel fibres, or conventional steel bar reinforcement, are not required in cases when the bond strength is adequate to prevent debonding. In order to validate a possible effect of fibres it is necessary to consider the case where a major debonded area developed.

A summary of crack widths obtained for SCC and SFRSCC overlays in which a major debonded zone was established can be seen in Fig. 11. Observe that the crack widths have been normalised with
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respects to the deformed length recorded in the tests in order to enable comparisons. It should also be mentioned that the actual fibre contents were calculated based on the counted number of fibres crossing the crack by using fibre orientation theory as described by Sorouhi & Lee 1999 and Lofgren 2005. An interesting observation is that the actual fibre content was lower (3 cases) or equal (1 case) to the expected amount. This shows that the fibre content varied in the SFRSCC overlays and that a major crack is more likely to form in a section with low fibre content.

From the results in Fig. 11 it is quite clear that increased fibre content resulted in a decreased crack width. Based on the trend line it can be seen that a content of 0.5 vol% reduced the crack width by approximately 50% while 0.75 vol% of steel fibres reduced the crack width to approximately one fourth of the width of cracks in plain SCC. This reduction correlates rather well with the results of the end-restrained test series, see Fig. 6. The reason for this is believed to be that the restraint situations were similar as the overlay strips under consideration were restrained only at the ends of the debonded zones.

Important to recognise is however that as long as only a single crack develops (which was the case for SCC and SFRSCC overlays) the actual crack width will be proportional to the size of the deformed zone, and it is thus not possible to determine a certain content of steel fibres required to limit crack widths. A distributed crack pattern would be required in order to enable crack limitation within a deformed zone.

THEORETICAL MODEL

An analytical model to determine the risk of cracking and the crack width of concrete overlays exposed to shrinkage has been proposed by Carlswärd 2006. The analysis is divided into two stages (Fig. 12); (1) an initial analysis in the uncracked stage, and (2) crack width analysis. In the first stage (1) the tensile stress development is calculated based on beam theory for a composite member, assuming full bond between the two layers. The analysis, which is based on a previous model described in Silfwerbrand 1997, includes the rate of shrinkage and creep, the development of stiffness and tensile strength and the situation of restraint.

Two situations are distinguished in the cracked stage (2), assumed to be reached when the tensile stress exceeds the overlay strength. The first is that an internal deformed area develops and the second that the overlay is still bonded after cracking. In case of debonding it is assumed that just one single crack occurs in the unbonded region while a distributed crack pattern is expected for a bonded overlay. The assumed cracking response is based on observations from the base restrained test series.

(1) Uncracked stage

The stress state in the overlayed structure is evaluated using beam theory for a statically determined system, see (1) in Fig. 12. An incremental procedure is adopted to calculate the shrinkage stress development including the favourable effect of creep, or rather stress relaxation (Fig. 13a and b).

Assuming an even shrinkage distribution over the overlay, see $\varepsilon_c(t)$ in Fig. 12, the average stress in the overlay at time $t$ due to the increment of shrinkage strain applied at $t$, $\Delta \varepsilon_c(t)$, may be calculated using Eq. (1). For more details on how the expression is actually derived reference is made to Carlswärd 2006.

$$\sigma_c(t,t_i) = \frac{m(t,t_i) \cdot (1-\alpha) \cdot \left[ m(t,t_i) \cdot (1-\alpha)^3 + \alpha^3 \right]}{m(t,t_i) + (m(t,t_i) - 1) \cdot \left[ m(t,t_i) \cdot (1-\alpha)^4 - \alpha^4 \right]} \cdot R_c(t,t_i) \cdot \Delta \varepsilon_c(t_i) \quad (1)$$

$\alpha$ in the above equation corresponds to the relative overlay depth (see Fig. 12 for definition) and $m(t,t_i)$ is the stiffness relation between overlay and substrate calculated as:

$$m(t,t_i) = \frac{E_b}{R_c(t,t_i)} \quad (2)$$

where $E_b$ is the elastic modulus of the substrate concrete and $R_c(t,t_i)$ is the relaxation function corresponding to the shrinkage strain applied at $t = t_i$.

It should be observed that for the situation of an extremely thick overlay in relation to the substrate ($\alpha \to 1$), the stress in Eq. (1) will equal zero and oppositely, for a very thin overlay ($\alpha \to 0$), the...
stress will reach the value of \( R_c(t, t_i) \Delta \varepsilon_c(t) \). At the moment when the increment of shrinkage strain \( \Delta \varepsilon_c(t) \) is applied the relaxation modulus \( R_c(t, t_i) \) corresponds to the elastic modulus of the overlay concrete \( E_c(t_i) \), implying that the momentary stress at \( t = t_i \) \( (\Delta \sigma(t_i) = \sigma(t, t_i)) \) equals the elastic stress \( E_c(t_i) \Delta \varepsilon_c(t_i) \). The stress will however successively decrease due to stress relaxation as the relaxation modulus \( R_c(t, t_i) \) decreases with time.

Simplifications of the model are that creep and shrinkage of the substrate are neglected and that the influence of selfweight is not considered. It is also assumed that the Poisson’s ratio equals zero.

The total stress at any time \( t \) is calculated by adding the contribution from each increment as:

\[
\sigma(t) = \sum_{i=1}^{n} \sigma(t, t_i)
\]  

(3)

**Verification of stress analysis** – The development of stresses and strains was calculated for the overlay strips of the base restrained test series using the methodology outlined above based on measured free shrinkage as shown in Fig. 8. The relative depth \( \alpha \) of the overlays were calculated to 0.14 by dividing the overlay depth with the total depth of base slab and overlay. For estimation of the factor \( m(t, t_i) \) in Eq. (1) it is necessary to know the relaxation development of the overlay material and the elastic modulus of the base slab according to Eq. (2). The latter property \( (E_c) \) was assumed to be 35 GPa (5 076 142 psi). The relaxation functions for different time increments of the overlay \( R_c(t, t_i) \) was calculated based on the creep and elastic modulus development as follows:

\[
R_c(t, t_i) = \frac{1}{E_c(t_i)} \cdot \frac{\varphi(t, t_i)}{E_{cm}}
\]  

(4)

A formulation given in Eurocode 2 was adopted to calculate the elastic modulus at different times from the mean compressive strength (Eq. (5)).

\[
E_c(t_i) = \left( \frac{f_{cm}(t_i)}{f_{cm}} \right)^{0.3} \cdot E_{cm}
\]  

(5)

where \( f_{cm} \) and \( E_{cm} \) are the mean compressive strength (in MPa) and elastic modulus (in GPa) at an age of 28 days, and \( f_{cm}(t_i) \) and \( E_c(t_i) \) are the corresponding values at time \( t_i \). The elastic modulus at 28 days and the compressive strength development were calculated according to Eurocode 2 as:

\[
E_{cm} = 10 \cdot f_{cm}^{1/3}
\]  

(6)

\[
f_{cm}(t) = e^{s \left( 1 - \left( \frac{28}{T} \right)^{0.3} \right)} \cdot f_{cm}
\]  

(7)

The factor \( s \) in Eq. (7) is a parameter to consider the type of cement used and a value of 0.25 was adopted here. Creep development \( \varphi(t, t_i) \) was also calculated in accordance with Eurocode 2. More details can be found in Carlswärd 2006.

A comparison of measured and calculated strains at mid section at the upper face of some of the overlay strips is given in Fig. 14 (a). The response of the overlay strips that were more or less completely debonded (see overlay strips denoted with II in Fig. 10) have been left out as a basic condition for the model is that the overlay is fully bonded to the base.

From the results shown in Fig. 14 (a) it can be concluded that the restraint was not 100% even though the base slabs had a rather significant depth (300 mm or 11.8 in.) in relation to the overlay strips
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(50 mm or 2 in.), as such situation would have resulted in a zero strain reading. It can also be seen that the actual strain development was reasonably well captured by the theoretical calculation within the first 25-30 days. At this age however, some major cracks developed in mid section of at least two of the strips (SCC I and SFRSCC 30 I), which resulted in sudden strain changes.

Calculated stress development in reference overlays (SCC) and shrinkage reduced overlays (SRASCC) is shown in Fig. 14 (b). It can be seen that crack initiation (time at which the stress exceeds the strength) was estimated to approximately 20 days for the reference mix SCC and 30 days for the shrinkage reduced mix (SRASCC). The tensile strength development shown in the diagram was calculated from the mean compressive strength (in MPa) using the following relation from Eurocode 2.

\[ f_{cm}(t) = 0.30 \cdot f_{cm}(t)^{2/5} \]  

Crack mapping of the overlay strips was only conducted at a few different times including ages 14 and 28 days after casting. Results from the inspections showed that most of the bonded strips were crack-free at the first age (14 days) while quite a number of cracks had developed at 28 days. This verifies that the prediction shown in Fig. 14 (b) is reasonably accurate for the reference concrete (SCC). The prolonged time to cracking predicted for SRASCC was however not verified by experiments. It should also be mentioned that the predicted age of cracking for SCC with steel fibres (SFRSCC 30 and 60) was approximately the same as for the reference concrete (SCC), as the tensile strength as well the shrinkage, creep and elastic modulus development was similar.

(2) Cracked stage

There are two possible situations that need to be considered after crack initiation according to above (see Fig. 12). For the first situation, where the bond strength is sufficient to prevent debonding, it has been proposed in Concrete report 13 2008 to estimate the crack width based on the free shrinkage strain and the depth of the overlay as follows:

\[ w_{cr} = \epsilon_{cs} \cdot 3 \cdot h_{o} (1 - \frac{R_{10,20}}{100}) \]  

where \( \epsilon_{cs} \) is the free shrinkage strain, \( h_{o} \) is the overlay depth and \( R_{10,20} \) is the residual strength factor. The equation implies that the crack width will diminish with increasing residual strength factor. For strain hardening fibre concrete \( (R_{10,20} \geq 100 \%) \) the theoretical crack width will be zero, which is of course not true. However, it is believed that multiple cracking will occur for this situation resulting in cracks with insignificant width.

For case 2, where a major debonded area develops after cracking, it was observed in the base restrained tests that crack distribution was not obtained for SFRSCC, that is for the volume fractions and type of steel fibres applied. This means that the crack width will depend on the length of the debonded zone in addition to the reinforcing effect of the fibres. The following formulation to estimate the crack width (Eq. (10)) was proposed in Carlswärd 2006 for this situation.

\[ w(t) = (\epsilon_{c}(t_{cr}) + \epsilon_{cs}(t,t_{cr})) \cdot \left(1 - \frac{R_{10,20}}{100}\right) L_{deb} \]  

where \( w(t) \) is the crack width at time \( t \) after cracking, \( \epsilon_{c}(t_{cr}) \) is the elastic concrete strain at the time of cracking, \( \epsilon_{cs}(t,t_{cr}) \) is the shrinkage strain that develops from the age of cracking to time \( t \) and \( L_{deb} \) is the length of the debonded zone. Time of cracking \( t_{cr} \) is obtained from the analysis in the uncracked stage described above.

For plain concrete, for which the residual strength factor \( R_{10,20} \) is practically zero, the crack width at the time of cracking \( w(t_{cr}) \) will be equal to the elastic strain capacity, which is approximately 0.1 \( \% \), multiplied by \( L_{deb} \). The crack width will then increase in direct proportion to the shrinkage strain that
remains after cracking. It should be pointed out that the sum of shrinkage strain after cracking $\varepsilon_{sc}(t_{cr})$ and the elastic concrete strain $\varepsilon_e(t_{cr})$ will differ from the total shrinkage strain from time zero ($\varepsilon_{sh}(t_0)$) due to creep strain developed prior to cracking. In other words, $\varepsilon(t_{cr})$ includes both shrinkage strain and creep strain developed until cracking occurs.

The effect of fibre reinforcement is considered in a similar way as proposed in Eq. (9). It is simply assumed that the crack width will decrease in proportion to the residual strength factor $R_{10,20}$.

A major problem of the above method – important to know – is of course that it is not possible to know the extent of the debonded zone beforehand.

Verification of predicted crack widths – Crack widths observed at the end of the test period in the base restrained test series were approximately 0.05-0.1 mm (0.002-0.004 in.) in parts of the overlay strips that were bonded to the substrate (see Fig. 10). A free shrinkage strain of approximately 0.6 ‰ at the end of the test period for plain SCC (Fig. 8) and a depth $h$ of 50 mm (2 in.) gives a crack width according to Eq. (9) of 0.09 mm (0.0035 in.), which agrees reasonably well with experimental results for unreinforced SCC. For fibre reinforced overlays the $R_{10,20}$ values were estimated to 40 and 80 % for SFRSCC 30 and SFRSCC 60 respectively, see Carlswärd 2006 for details. With a similar value for the free shrinkage strain as for SCC (0.6 ‰) the theoretical crack width would thus be 0.05 and 0.02 mm respectively for SFRSCC 30 and SFRSCC 60 using Eq. (9). It was not really possible to observe this difference between plain and fibre reinforced overlay strips.

For the situation that the bond strength is insufficient to prevent debonding and assuming that the length of the debonded zone is known it can be shown that Eq. (10) gives reasonable results. A comparison of predicted and measured widths of cracks within debonded areas of the overlay strips in the base restrained test series is given in Fig. 15 (a).

The effect of steel fibres is shown in Fig. 15 (b), where relative crack width ($w/L_{deb}$) is given as a function of residual strength factor $R_{10,20}$ for overlay strips with a major debonded zone (> 1 m or 40 in.). The $R_{10,20}$ values in Fig. 15 (b) were estimated based on the actual volume fraction of fibres found in the crack zones, see Carlswärd 2006 for details. It may be seen that calculated (Eq. (10)) and measured crack widths correlated rather well. It is further evident that increasing $R_{10,20}$ resulted in decreased crack widths. A rather high $R_{10,20}$ value was however required in order to provide a substantial reduction in crack width. For an $R_{10,20}$ value of 80 %, corresponding to SCC with 0.75 vol% of steel fibres in the present study, the crack width was found to be reduced to approximately 0.1 mm/m. In case of a crack width demand of 0.2 mm (0.08 in.), which is rather common, the debonded length may thus not exceed 2 m. This is of course only valid for this particular situation. In other cases, where shrinkage development, restraint, fibre types etc are different the results may not be the same.

DISCUSSION

Results obtained from small scale tests, as described in the paper, are always disputable. One major issue regards the dimensions of the test specimens. The use of relatively small sizes makes it difficult to select a representative sample of concrete, which may have a significant impact on the fibre distribution. This was verified in both test methods of the present study when the number of fibres crossing cracks was counted. It is also well known that uneven fibre distribution is the reason for the often very high coefficients of variation that are typically obtained when evaluating the residual strength through small scale beam tests, see RILEM TC 162-TDF. In the present study this problem was overcome to some extent by calculating a theoretical fibre volume fraction based on a known number of fibres in the crack zone.

Another issue that may be discussed is the crack reinforcing function of steel fibres. In real construction the volume fraction of fibres added seldom exceeds 0.5 vol%. According to test results the crack width reduction corresponding to this addition was approximately 50 % compared with unreinforced SCC in situations with end-restraint, see Figs. 6 and 11. Crack distribution was however not observed in the end-restrained test series or within unbonded regions of the base restrained test series. This implies that fibre reinforcement may not be as effective from a crack limiting point of view as is often expected. It is believed that the reason why a crack distributing effect was not achieved in the tests is that the fibre volumes added were not sufficient to provide strain hardening properties, that is the residual stress in the crack zone is lower than the strength of the matrix.

It should however be kept in mind that the uniaxial stress field generated in the small scale test specimens does not reproduce the stress state in real overlay conditions, where non-uniform internal
Prediction of Stress Development and Cracking

Stresses occur in multiaxial directions. In other words it is possible that a better effect of fibres may be found in reality. This hypothesis is investigated within an ongoing Swedish project, where larger overlay tests are conducted to assess the crack limitation potential of SFRSCC.

There are some issues of the presented theoretical model to discuss as well. The first is that, although numerous studies have been conducted over the years to study the effect of various factors on bond (Shin & Lange 2004; Bonaldo et al. 2005; Júlio et al. 2004; Silfwerbrand & Paulsson 1998; Momayez et al. 2005; Courard 2005), it is difficult to find a method of preparation, or an overlay system, which will ensure high and even bond distribution. This is clearly a major drawback of this particular structural application since the bond strength determines the potential of debonding and thus the cracking response and lifetime of the overlay. Some research has also been conducted to study the debonding mechanism (Granju et al. 2004; Granju & Turatsinze 2005) and methods have been proposed to estimate the debonding stress (Carlswärd 2006; Jonasson 1978). However, as long as the real bond strength cannot be controlled such methods can only be used to estimate the risk of debonding. As a consequence it is not possible to determine, on beforehand, which of the two proposed equations for crack width estimation to select (Eqs. (9) or (10)).

Another weakness of the theoretical model is that the size of a potential debonded zone, \(L_{\text{deb}}\) in Eq. (10), is an unknown variable. Thus, in order to predict the crack width in cases where the bond strength is insufficient to prevent major debonding it is necessary to make some kind of estimation of the debonding zone. This is clearly not a good solution, particularly not in situations when crack control is essential. The only way of overcoming this problem for overlay situations where the potential of achieving sufficient bond strength is questionable, is to provide adequate reinforcement to distribute cracks. Results from the present study imply that this would require fibre concrete with a rather high residual strength. This is also reflected in the Concrete report no 13 2008, where it is recommended that the \(R_{10,20}\)-value should be at least 85 % for overlays where the potential of achieving high and even bond to the substrate is uncertain.

**SUMMARY AND CONCLUSIONS**

Two different test methods have been used to assess the effect of steel fibres on the cracking potential of SCC due to restrained shrinkage, an end-restrained and a base restrained type of method. Results from the end-restrained setup showed that increasing steel fibre contents resulted in a prolonged time to cracking and decreasing crack widths. At a volume fraction of 0.5 vol%, the crack width was approximately 50 % of that observed in unreinforced SCC. However, for the certain type of fibre and volume fractions considered in the study (\(\leq 0.5\) vol%) only one crack developed, in other words crack distribution was not achieved. This implies that it is not possible to draw any conclusions regarding the volume fraction of fibres required for crack control in a real case of end-restraint.

The results were verified in the base restrained test series, where thin overlay strips were cast on a rigid concrete slab. In cases when a major internal debonded zone developed, giving a similar restraint situation as for the end-restrained test, only one crack occurred in SFRSCC. Although it was observed in the tests that increasing fibre contents did give smaller crack widths the results thus imply that the extent of the debonded zone will be decisive for the crack width. In overlay strips with equal debonded length the crack width for a steel fibre volume fraction of 0.75 vol% was found to be approximately similar to that obtained in steel bar reinforced SCC. Rather high amounts of steel fibres are thus required in order to control cracks.

It was further shown in the base restrained test series that, if high and even bond to the substrate is obtained, reinforcement is not required to distribute cracks in thin overlays (~50 mm or 2 in.). Test results revealed that a distributed pattern of fine cracks (0.05-0.1 mm or 0.002-0.004 in.) developed for this situation both for unreinforced and steel fibre reinforced SCC. A main conclusion from the base restrained test series is, thus, that in order to ensure crack control for thin SCC overlays it is essential that sufficient bond strength is achieved.

A theoretical model to predict overlay cracking and crack widths was also proposed. Comparisons with results from the base restrained test series showed a good correlation. For the situation that the overlay is not sufficiently bonded to the substrate it is however difficult to predict potential crack widths of SFRSCC as the extent of debonding will be quite influential.
ACKNOWLEDGEMENTS

The financial support provided by Betongindustri AB, a ready mix concrete company within the Heidelberg Cement Group, and SBUF (the Development Fund of the Swedish Construction Industry) is gratefully acknowledged. Appreciation is also directed to Testlab, the testing laboratory at Luleå University of Technology, for the assistance in conducting the experimental parts of the study.

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Granju, J.L. and Turatsinze, A., 2005 “Repairs by the thin bonded overlay technique: the RILEM TC 193-RLS and last findings about the debonding mechanism,” In: Proceedings of the Conference on Concrete Repair, Rehabilitation and Retrofitting, Nov., Cape Town, South Africa, pp. 43-52


Prediction of Stress Development and Cracking


Table 1 – Basic concrete mix composition used in the base-restrained test series.

<table>
<thead>
<tr>
<th>Materials</th>
<th>Cement kg/m³ (lb/yd³)</th>
<th>Sand 0/8 kg/m³ (lb/ft³)</th>
<th>Filler kg/m³ (lb/ft³)</th>
<th>Sika 56 kg/m³ (lb/ft³)</th>
<th>Water kg/m³ (lb/ft³)</th>
<th>w/c</th>
</tr>
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<tr>
<td>Basic mix proportions</td>
<td>370 (624)</td>
<td>1613 (2719)</td>
<td>100 (169)</td>
<td>4.1 (7)</td>
<td>215 (362)</td>
<td>0.58</td>
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</table>

Table 2 – Cylinder compressive strength at 28 days and flexural strength and residual strength factors obtained from four point bending tests in accordance to ASTM C1018-92.

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>$f_{cm}$</th>
<th>$f_{fl,cr}$</th>
<th>$R_{10,20}$</th>
<th>$R_{10,30}$</th>
<th>$R_{10,40}$</th>
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</thead>
<tbody>
<tr>
<td>SCC</td>
<td>30.5 (4425)</td>
<td>4.2 (609)</td>
<td>4</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>SFRSACC 20</td>
<td>36.5 (5294)</td>
<td>4.4 (638)</td>
<td>26</td>
<td>26</td>
<td>26</td>
</tr>
<tr>
<td>SFRSACC 30</td>
<td>39.0 (5656)</td>
<td>4.2 (609)</td>
<td>37</td>
<td>38</td>
<td>38</td>
</tr>
<tr>
<td>SFRSACC 40</td>
<td>36.5 (5294)</td>
<td>4.7 (682)</td>
<td>59</td>
<td>60</td>
<td>60</td>
</tr>
</tbody>
</table>

$^*$Mean cylinder strength
Figure 1 – Cracking and edge lifting of a concrete overlay due to differential shrinkage.

Figure 2 – Common types of test methods used to study the effect of restrained shrinkage.

Figure 3 – Test setup adopted for evaluating the effect of end-restrained shrinkage.
Figure 4 – Illustration of a free shrinkage specimen and results from unrestrained shrinkage deformation of the mixes within the test program. Recordings on the bottom face were conducted for mix SFRSCC 20 and 30 (0.25 and 0.375 vol% of steel fibres).

Figure 5 – Strain development measured in the cracked zone (crack strain) and in uncracked parts compared to free shrinkage results measured on the upper face of unrestrained companion specimens. (a) Plain SCC. (b) SFRSCC 20 (0.25 vol% steel fibres). (c) SFRSCC 30 (0.375 vol% of steel fibres). (d) SFRSCC 40 (0.5 vol% steel fibres).
Figure 6 – Effect of steel fibre addition on crack widths obtained for the end-restrained test series. Relative crack width equals the crack width of SFRSCC \( w_{\text{sfrscc}} \) divided by the crack width of plain SCC \( w_{\text{scc}} \). Examples of results from two studies (Voigt et al. 2004; Shah & Weiss 2006) where ring shaped specimens (see Fig. 2) were used to assess the effect of steel fibres are given for comparison.

Figure 7 – (a) Overlay strips on one of the slabs directly after casting. (b) Smooth and rough texture of substrate obtained by grinding and milling.

Figure 8 – Test method to determine shrinkage development and measured shrinkage at the upper and lower faces as well as the mean shrinkage of the mixes of the base restrained test series.
Figure 9 - Measured crack widths at the end of the measuring period approximately 4 months after initiation of drying. (Smooth = grinded, rough = milled).

Figure 10 – Example of cracks in overlays on slab 3. Shaded areas indicate that the overlays have debonded. More examples of test results may be found in Carlswärd 2006.
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Figure 11 – Crack width divided by debonded length as a function of steel fibre amount. Only the cracks that developed within major debonded areas have been considered.

Figure 12 – Proposed theoretical model divided into two stages: (1) initial stress analysis in the uncracked stage and (2) crack width analysis.

Figure 13 – (a) Shrinkage strain development in time and (b) corresponding stress development in a restrained structure incorporating the effect of relaxation.
Figure 14 – (a) Measured strain development at the upper face of overlay strips on slab 3 in mid-section compared to calculated strain development at the upper face of the overlays using the proposed theoretical model. (b) Calculated stress development in overlays of concrete types SCC and SRASCC.

Figure 15 – (a) Measured versus calculated widths of cracks that developed within debonded zones of the base restrained overlay strips. (b) Measured and calculated relative crack width (crack width divided by debonded length) as a function of residual strength factor $R_{10,20}$ for cracks that developed in major debonded zones (> 1 m = 40 in.) in SCC and SFRSCC overlays of the base restrained test series.
Tensile Behavior of Steel Fiber-Reinforced Self-Compacting Concrete

by V. M. C. F. Cunha, J. A. O. Barros, J. M. Sena-Cruz

Synopsis: In the present work the tensile behavior of a self-compacting concrete reinforced with two hooked ends steel fiber contents was assessed performing stable displacement control tension tests. Based on the stress-displacement curves obtained, the stress-crack width relationships were derived, as well as the energy dissipated up to distinct crack width limits and residual strengths. The number of effective fibers bridging the fracture surface was determined and was compared with the theoretical number of fibers, as well as with the stress at crack initiation, residual stresses and energy dissipation parameters. In general, a linear trend between the number of effective fibers and both the stress and energy dissipation parameters was obtained. A numerical model supported on the finite element method was developed. In this model, the fiber reinforced concrete is assumed as a two phase material: plain concrete and fibers randomly distributed. The plain concrete phase was modeled with 3D solid finite elements, while the fiber phase was modeled with discrete embedded elements. The adopted interface behavior for the discrete elements was obtained from single fiber pullout tests. The numerical simulation of the uniaxial tension tests showed a good agreement with the experimental results. Thus, this approach is able of capturing the essential aspects of the fiber reinforced composite's complex behavior.

Keywords: FEM; steel fiber-reinforced self-compacting concrete; uniaxial tensile behavior
Self-compacting concrete (SCC) can be defined as a concrete that is able to flow in the interior of the formwork, filling it in a natural manner and passing through the reinforcing bars and other obstacles, flowing and consolidating under the action of its own weight (Okamura 1997). Discrete steel fibers are added to cement based materials to increase their post-cracking residual strength, energy dissipation capacity and impact resistance (ACI 1996). The advantages associated to the addition of steel fibers to concrete mixes may be combined with the ones resulting from the self-compacting ability concept in concrete. The resulting material is designated by steel fiber-reinforced self-compacting concrete (SFRSCC) and, when compared to conventional concretes, presents clear technical advantages in terms of costs/benefits ratio and an enhanced post-cracking behavior.

To explore the potentialities of SFRSCC for structural applications, mainly for the development of innovative systems in the precasting industry (Barros et al. 2007), a research project has being carried out evolving experimental, analytical and numerical research. With the purpose of getting, as much as possible, a consistent understanding of the behavior of this composite material, and to collect data for the calibration of the analytical formulations and FEM-based numerical models, the experimental research covers the micromechanics aspects of the fiber pullout (Cunha et al. 2006, 2008), the compressive (Cunha et al. 2008b) and flexural behavior (Pereira et al. 2008) and, more recently, the uniaxial tensile behavior.

The present work is dedicated to the characterization of the tensile behavior of two SFRSCC mixes, one reinforced with 30 kg/m³ (1.873 lb/ft³) and the other with 45 kg/m³ (2.809 lb/ft³) of hooked end steel fibers. From the stress-displacement curves, the stress-crack width relationships were derived and the values of parameters that characterize the residual strength and energy absorption for certain crack width limits were determined. The relevant aspect of the fiber distribution in SCC mixtures was also assessed comparing the values of these parameters determined in specimens extracted from top and bottom zones of SFRSCC elements. The influence of the number of effective fibers crossing the fracture surface in terms of the values of the parameters that characterize the post-cracking behavior of SFRSS was also assessed.

Finally, taking into account the force-slip relationships obtained in pullout tests with steel fibers of distinct inclination and embedded bond length, a numerical approach was developed assuming SFRSCC as a two phase material: SCC and fibers. SCC is simulated by a 3D multi-fixed smeared crack model developed by Ventura et al. (2008), while fibers are modeled as discrete embedded cables, whose internal forces are obtained from the force-slip relationships recorded in pullout tests, taking into account the inclination between fiber and the neighbor crack plane formed in the surrounding SCC, as well as an adopted average value for the fiber embedded length. In this approach it is assumed that the slip is equal to the crack width, which is determined multiplying the crack strain normal to the crack by the crack band width (Ventura et al. 2008). The present work describes the experimental work concerning to the characterization of the uniaxial tensile behavior of the developed SFRSCC. A brief description of the numerical model is done and its predictive performance is assessed.
Tensile Behavior of Steel Fiber-Reinforced Self-Compacting Concrete

RESEARCH SIGNIFICANCE

Structural applications of SFRC are yet to some extent limited, if having in mind that the appearance of SFRC dates back to the early sixties (ACI 1996). The high scatter of the SFRC material behavior, in part due to non-uniform fiber distribution, contributes to the mistrust in this material. In order to overcome these doubts it is of vital importance reducing the material behavior scatter, and consequently enabling the adoption of lower material safety factors (Shah and Ferrara 2008). Self-compacting concrete is effective in guaranteeing a more uniform distribution of fibers within the specimen, as well in effectively orienting them along the casting direction. Nowadays, it is acknowledge that SFRSCC exhibits better post-cracking behavior with a lower scatter than conventional SFRC.

In this paper, the post-cracking behavior of SFRSCC obtained from uniaxial tensile tests is presented. The results are discussed based upon the micro-mechanical behavior of the steel fibers used in the composition. The fibers micro-mechanical behavior was previously assessed by means of single fiber pullout tests (Cunha et al. 2008b). The knowledge of the micro-mechanical behavior enables a deeper understanding of the composite behavior, which often lacks in the discussion of the experimental results presented in the available literature.

The numerical model developed and employed to simulate the SFRSCC’s uniaxial tensile behavior assumes the composite as a two phase material: SCC (paste and aggregates) and fibers. The random fiber distribution into the matrix is simulated with an algorithm supported on the Monte Carlo method, providing a realistic distribution of the fibers in a certain specimen. The numerical fiber distribution generated takes into account mould and fiber dimensions. This enables to consider the so-called wall effect in distinct ways depending on the mould shape and geometry. With a realistic approximation of the actual fiber distribution and with the knowledge of the micro-mechanical behavior of the fibers, it was possible to predict the macro-mechanical behavior of laboratory’s specimens. The presented numerical approach is not yet fully validated, however the results are promising and its application to other structural elements of fiber reinforced concrete with complex geometry may be, in the future, easily achieved with good results.

MATERIALS AND SPECIMENS

The materials and mixture procedures used for the SFRSCC are described elsewhere (Pereira et al. 2008). Two batches with distinct contents of fibers (Cf), 30 and 45 kg/m³ (1.873 and 2.809 lb/ft³) were used to study the SFRSCC tensile post-cracking behavior. Table 1 includes the composition that has best fitted self-compacting requirements for the adopted two fiber contents. Remark that, in Table 1, WS is the water necessary to saturate the aggregates, and W/C is the water/cement ratio. The WS portion was not used to compute the W/C ratio. Hooked-ends steel fibers (length, \( l_f = 60 \text{ mm} (1.181 \text{ in}) \), diameter, \( d_f = 0.75 \text{ mm} (0.0216 \text{ in}) \), with an aspect ratio, \( l_f/d_f = 80 \), and yield stress of 1100 MPa (159542 psi)) were used for the study.

For each batch, eight cylinders with a diameter of 150 mm (5.905 in) and 300 mm (11.810 in) height were cast. Three of them were used to assess the compressive strength of each SFRSCC batch. At the date when the tests were performed, the series with 30 kg/m³ (1.873 lb/ft³) of fibers had an average compressive strength of 71.1 MPa (10312 psi) with a coefficient of variation, CoV, of 1.9 %, while in the series with 45 kg/m³ (2.809 lb/ft³) of fibers the SFRSCC had an average compressive strength of 67.2 MPa (9747 psi) with a CoV of 1.4 %.

TEST SETUP

The RILEM TC 162-TDF (2001) recommendations for the uniaxial tension test of steel fiber-reinforced concrete were adopted in this work. According to this document, a notched cylinder of both 150 mm (5.905 in) diameter and height should be used. The specimens were swan out from standard cylinders having a height of 300 mm (11.810 in). Afterwards, a notch along the perimeter with a 15 mm (0.591 in) depth and 5 mm (0.197 in) thick was swan at mid height of the final test specimen. When the sawn operations were executed, the specimens were in its hardened-mature phase. These operations were conducted with care in order to ensure that the notch become perpendicular to the specimen’s axis.
Afterwards, each specimen was ground and carefully cleaned with both compressed air and solvent. The specimen was then directly glued, “in situ,” to the loading platens of the testing rig. The selected glue is a high strength epoxy resin, which achieves a tensile strength of about 30 MPa (4351 psi) and a bond strength between 4 to 8 MPa (580 to 1160 psi) (depending on the surface material and treatment characteristics).

A servo-hydraulic system with a 2000 kN (449618 lb) static load carrying capacity with a very stiff frame was used to perform the tensile tests, Figure 1a. A test was performed in closed-loop displacement control using the average signal of three displacement transducers mounted on two steel rings disposed at equal distances along the perimeter of the specimen, Figure 1b. The gauge length adopted was 35 mm (1.378 in), smaller than the upper limit length of 40 mm (1.575 in) suggested by RILEM TC 162-TDF (2001). The following displacement rates were used: 5 μm/min (0.000196 in/min) up to a displacement of 0.1 mm (0.00394 in); 100 μm/min (0.00394 in/min) until the completion of the test, i.e. a 2 mm (0.0787 in) displacement.

RESULTS

Stress-displacement curves

In all the uniaxial tension tests performed, cracking occurred along the notched plane, hence the desired crack localization was assured. The average curve and the envelope of the experimental uniaxial tension stress - average displacement relationships, $\sigma - \delta_{\text{avg}}$, for a fiber content of 30 and 45 kg/m$^3$ (1.873 and 2.809 lb/ft$^3$) are presented in Figures 2 and 3, respectively. Hereinafter, these series will be designated by Cf30 and Cf45, respectively. A detailed view of the initial part of the experimental response is depicted on the right side of these figures. For both tested series the $\sigma - \delta_{\text{avg}}$ response is linear almost up to peak. Only just before the peak load some non-linearity is observed. Once the peak load is attained, the load had a relatively accentuated decrease up to a displacement of about 0.10 mm (0.00394 in) (see right side of both Figures 2 and 3). Beyond this displacement value, a plateau or a hardening-plastic response has occurred. In general, the post-peak hardening is observed in the Cf45 series, but this response was also occurred in some specimens of the Cf30 series (Cunha 2009). On the other hand, after the plateau on the Cf30 series, i.e. beyond a displacement of nearby 0.8 mm (0.0315 in), the residual stress starts to decrease with sudden strength losses corresponding to the fiber fracture. In fact, during the execution of the tension tests, this was audible by the peculiar sound of fiber fracturing.

According to Stroeven and Hu (2006), the average orientation angle of the active fibers crossing a leading crack is 35° (this value was analytically derived), and a similar value (34°) was experimentally observed by Soroushian and Lee (1990). Recent fiber pullout tests with fibers at an inclination angle of 30° with the load direction have conducted to fiber rupture; the predominant failure mode occurred when the slip was in the range (0.6-1.0) mm (Cunha et al 2006). These reasons can justify the occurrence of the significant residual stress decay after the displacement of 0.8 mm (0.0315 in), in Cf30 series, where the number of active fibers is relatively reduced for the strength of the matrix.

Concerning the Cf45 series (Figure 5), the post-peak hardening observed can be ascribed to two reasons: a higher number of active fibers crossing the crack; notice that both the compressive strength and stiffness of the Cf45 series matrix are lower than the Cf30, thus the fiber-matrix bond properties are not so favorable to proportionate fiber rupture. After the peak load, the stress starts to decrease until a minimum stress is attained, roughly about 0.1 mm (0.00394 in). As micromechanics of hooked-ends fiber pullout demonstrate (Cunha 2009), above this displacement, the strengthening provided by fiber hooked-ends starts to be the predominant fiber reinforcement mechanism. Since there are more fibers intersecting the crack, and due to the lower tensile strength of the concrete, the energy released during cracking is smaller when compared to the Cf30 series. Moreover, due to the lower tensile strength and matrix stiffness, fibers did not fracture so often as in the Cf30 series. Consequently, beyond a displacement around 0.1 mm (0.00394 in) a hardening phase occurred up to a displacement value of about 1.0 mm (0.0394 in).

In general, the responses exhibit very low scatter in the pre-peak phase. On the other hand, in the post-peak branch the scatter was considerably higher, particularly in Cf30 series and for a displacement higher than 0.1 mm (0.00394 in). Up to 0.1 mm (0.00394 in) the commanding pullout reinforcement mechanism is the chemical bond (Cunha 2009), hence the influence of the fiber dispersion (implicitly
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orientation) is not so important. As the crack width increases, the fiber’s hooked-ends start to be mobilized. Hence, the scatter of the post-peak behavior increases due to the variation of the fiber dispersion between different specimens, namely, the fiber orientation. Moreover, for the Cf30 series, fiber rupture was the predominant fiber failure mode, as it was expected taking into account the results obtained in fiber pullout tests presented elsewhere (Cunha et al. 2006).

Stress-crack opening curves

Figures 4 and 5 present the envelope of the stress-crack width curves obtained in the tests for the Cf30 and Cf45 series, respectively. A stress crack width curve (\(\sigma - w\)) was derived from a stress displacement curve (\(\sigma - \delta\)) according to the recommendations of RILEM TC 162-TDF (2001). The average and the characteristic \(\sigma - w\) curves are also included in these figures. The characteristic \(\sigma - w\) curve for the lower bound (L.B.) and upper bound (U.B.) with a confidence level of \(k = 95\%\) was obtained from the average curve computed from all tests, \(\bar{\sigma}_w(w)\), according to RILEM TC 162-TDF (2001):

\[
\sigma_{\text{eff}, \text{w}}(w) = \bar{\sigma}_w(w) \frac{G_{F2\text{mm},k}}{G_{F2\text{mm}}}
\]  

(1)

where \(G_{F2\text{mm},k}\) is the average energy dissipated up to a crack width of 2 mm (0.0787 in) and \(G_{F2\text{mm}}\) is the characteristic energy dissipated for the same crack width. To compute \(G_{F2\text{mm},k}\) a t-Student distribution was assumed.

Stress and toughness parameters

The average and characteristic values of the stresses and toughness parameters, as well as the respective coefficients of variation obtained from the performed uniaxial tests, are included in Table 2. In this Table \(\sigma_{\text{peak}}\) is the maximum stress, while \(\sigma_{\text{eff}, \text{max}}\), \(\sigma_{\text{fmax}}\) and \(\sigma_{\text{2mm}}\) are the stress at a crack width values of 0.3, 1 and 2 mm, respectively (0.0118, 0.0394 and 0.0787 in). On the other hand, \(G_{F2\text{mm},k}\) and \(G_{F2\text{mm}}\) represent the dissipated energy up to a crack width of 1 and 2 mm, respectively (0.0394 and 0.0787 in). The characteristic values were obtained for a \(k=95\%\) confidence level assuming a t-Student distribution. The normality of the sample was ascertained by the Shapiro-Wilk test (Montgomery and Runger 1994). The number of total fibers, \(N_r\) and effective fibers, \(N_{\text{eff}}\), counted at the fracture surface are also included in Table 2. Effective fibers were considered all the fibers that had the end hook deformed, as well as the fibers that have ruptured. In spite of some researchers do not consider the ruptured fibers as “effective,” in the authors’ opinion they should be considered, since they are able to transfer forces between the crack surfaces up to reasonable crack width. The “fully effectiveness” of this type of fibers can be questionable, but it is feasible to admit that they are “partially effective.” For the simplicity sake, let’s assume that the fiber slip when it is being pulled out is approximately equal to the crack width. Then, as it can be observed in Cunha et al. (2006) work, in the pullout behavior of inclined fibers, depending on the inclination angle and embedded length, a fiber that fails by rupture can sustain forces up to a slip, i.e. crack width that varies approximately from 0.7 mm (0.0276 in) to 4 mm (0.157 in).

From the analysis of Table 2 it is verified that, in general, the stresses and toughness parameters increased with the fiber content, as it would be expected. The only exception was the peak stress which was nearby 10% lower for the series with a content of fibers of 45 kg/m³. Remark that this decrease is not due to the content of fiber, even though it could be indirectly appointed to it, since the aggregates, cement and additions contents for each series are distinct in order to attain self-compactability requirements. Moreover, the peak stress cannot be regarded as the actual tensile strength, but as estimation. In fact the notched forced a stress concentration at the notch tip, forcing the crack to be localized in the notched plan, which could not be the weakest section of an unnotched specimen (Van Mier and Van Vliet, 2002).

A significant increase of the post-cracking stresses was observed with the higher fiber content. An increase of the fiber content of 15 kg/m³ (0.936 lb/ft³) increased the residual strengths from 2 to 7 times, depending on the crack width value. Such increase is not simply justified by the higher number of fibers crossing the crack surface for the Cf45 series, since there are other factors that contributed to this fiber reinforcement effectiveness. One of these factors is the predominant fiber failure mode that, as already mentioned, was fiber rupture in the Cf30 series and fiber pullout in Cf45 series.
As far as the dissipated energy ($G_f$) is concerned, in the Cf45 series a significant increase (2 to 2.6 times) up to both deflections of 1 and 2 mm (0.0394 and 0.0787 in) was observed. In general, the CoV values obtained for the $G_f$ parameters were considerably smaller for the Cf45 series. At a first glance, these values may seem relatively high; the magnitude of such values is, however, within the expected values for this type of material. In fact, in general, the obtained CoV values were smaller than the ones reported by other authors with the same test procedure and specimen’s dimensions for conventional fiber reinforced concrete (Barragan 2002, Läofgren et al. 2008).

In Figure 6 is depicted the relationship between the total number of fibers, $N_f$ (Cunha 2009), and the number of effective fibers, $N_{f\text{eff}}$, at the fracture surface obtained for all the tested specimens of both Cf30 and Cf45 series. Since the uniaxial tension test specimens (with 150 mm (5.905 in) height) were obtained from distinct parts of a standard cylinder with 300 mm (11.810 in) height, additionally, the specimens obtained from the bottom and upper part of the standard cylinder are distinguished in Figure 6. This process enables an indirect assessment of an eventual influence of the material specific weight (gravity) on the fiber distribution. Figure 6 shows a linear relationship between the $N_f$ and $N_{f\text{eff}}$. Moreover, a tendency for an eventual higher concentration of fibers in the bottom part of the specimen was not found. Dupont (2003) has also found a linear relationship between the total number of fibers and the number of effective fibers for small fiber contents. However, for higher fiber contents this relationship becomes nonlinear, with an increase of $N_{f\text{eff}}$ as smaller as higher is $N_f$. In fact, due to the group effect, $N_{f\text{eff}}$ may decrease as the fiber spacing decreases due to the higher probability of mutual influence of adjacent fibers (Naaman and Shah 1976).

**Influence of the number of effective fibers on the post-cracking parameters**

Throughout Figures 7a to 7f are depicted the relationships between $N_{f\text{eff}}$ and the aforementioned post-cracking parameters obtained from the uniaxial tension test. Regarding the peak stress, $\sigma$, no significant relation was observed with the $N_{f\text{eff}}$ increase (Figure 7a). In spite of $\sigma_{\text{peak}}$ for the Cf45 series has been, in general, smaller than for Cf30 series, due to the reasons already pointed out, there was no significant relation between $\sigma_{\text{peak}}$ and the origin of specimen, i.e. from lower or upper part of a standard cylinder. This suggests that there was no significant segregation of the matrix skeleton and paste in the casted cylinders. On the other hand, for the residual stresses at a crack width of 0.3 mm (0.0118 in), $\sigma_{0.0118}$, and 1 mm (0.0394 in), $\sigma_{0.0394}$, a linear relationship between these residual stresses and the $N_{f\text{eff}}$ is quite evident (Figures 7b and 7c). This was expected, since the residual stress sustained by the crack is intimately related to number of mobilized fibers. Concerning the Cf45 series, the residual stresses for the “top specimens” exhibited a lower scatter when compared to the values from the “bottom specimens”. Due to technical problems occurred during the test program, the number of specimens from the bottom part (5) was different of the specimens obtained from the top part of the standard cylinder (3); therefore, no conclusive elations could be withdrawn in this subject.

The relation between the residual stress at a crack opening of 2 mm (0.0787 in), $\sigma_{0.0787}$, and the $N_{f\text{eff}}$ is represented in Figure 7d. When compared to the $\sigma_{0.0394}$-$N_{f\text{eff}}$ and $\sigma_{0.0118}$-$N_{f\text{eff}}$ relationships, the overall trend differed. First of all, there are two clear distinct trends for the $\sigma_{0.0118}$-$N_{f\text{eff}}$ relationship. For the Cf30 series, the increase of $\sigma_{0.0118}$ with $N_{f\text{eff}}$ is marginal and can be assumed null. On the other hand, for the Cf45 series a linear increase of $\sigma_{0.0118}$ with $N_{f\text{eff}}$ is quite visible, in spite of a higher scatter than those register for the $\sigma_{0.0394}$-$N_{f\text{eff}}$ and $\sigma_{0.0118}$-$N_{f\text{eff}}$ relationships. Moreover, a clear jump on the $\sigma_{0.0118}$ value from the series Cf30 to Cf45 is visible, even though there is a small difference of $N_{f\text{eff}}$ between the Cf30 series’ specimen with the highest $N_{f\text{eff}}$ and the Cf45 series with the lowest $N_{f\text{eff}}$. This considerable jump on $\sigma_{0.0118}$ value from the Cf30 to Cf45 specimens is not ascribed to the increase of effective fibers, hence this jump comprises differences on the fiber micro-mechanical behavior between the two series with distinct fiber content, as previously stated. In fact, for the series Cf30, in general, the fibers ruptured before a 2 mm (0.0787 in) crack width, while in case of Cf45 series, in general, the fibers were fully pulled-out enabling a higher crack bridging stress transfer effectiveness.

Finally, in both series and regardless the extracting location of the specimen, the dissipated energy has shown a linear increase with $N_{f\text{eff}}$ for both considered crack width limits. Nevertheless, there are two aspects that should be emphasized. For the energy dissipated up to a 2 mm (0.0787 in) crack width, $G_{f\text{2mm}}$, there is also a jump on the $G_{f\text{2mm}}$ value from the Cf30 to Cf45 series for the same reasons pointed
for the \( \sigma_{2\text{mm}}-N_{f}^{\text{eff}} \) relationship. The other aspect is that the increment rate of both \( G_{f1\text{mm}} \) and \( G_{f2\text{mm}} \) with \( N_{f}^{\text{eff}} \), for the CI45 series, was slightly smaller than for CI30 series.

**NUMERICAL SIMULATION**

**Numerical model**

In steel fiber reinforced cementitious composites (SFRC), steel fibers and matrix are bonded together through a weak interface, which behavior is important to understand the mechanical behavior of SFRC, since properties of the composite are greatly influenced by this interface zone (Pereira 2006). Taking this into account and that the fiber contribution for the post-cracking behavior of a composite is significantly higher than the unreinforced matrix contribution, it was settled to model the SFRC as a two phase material. In the developed model, SFRC is treated as a heterogeneous medium comprised by one homogeneous phase (aggregates and paste), and another one composed by the steel fibers. The fracture process of the cementitious matrix (unreinforced) is modeled with a 3D multi-fixed smeared crack model. The formulation of this crack model can be found elsewhere (Ventura et al. 2008). On the other hand, the stress transfer between crack planes due to the fibers bridging an active crack is modeled with discrete embedded elements. A nonlinear behavior law is assigned to these elements in order to account the fiber-matrix interface properties.

The random fiber distribution into the matrix is simulated with an algorithm supported on the Monte Carlo method, providing a realistic distribution of the fibers in a certain specimen. The geometry, location and orientation of the fibers are subsequently inserted into a three dimensional finite element mesh. This approach was adopted, mainly, due to the following reasons: 1) a homogenization of the reinforcements (fibers) crossing a certain solid element is difficult due to the random nature of the fiber distribution; 2) the discrete modeling of the reinforcements as bar elements located along the solid element nodes leads to a higher computational cost due to an unnecessary concrete mesh refinement. Moreover, this mesh refinement could lead to numerical errors caused by distorted elements for comprising the fiber distribution.

The contribution of the steel fibers crossing a solid volume is given by:

\[
K^{\text{eff}} = K^{c} + \sum_{i=1}^{n_{f}} K_{i}^{f}
\]

where \( K^{c} \) and \( K_{i}^{f} \) are, respectively, the stiffness matrix of the reinforced solid element (plain concrete + fiber reinforcement contribution), the stiffness matrix of plain concrete and the stiffness matrix of the \( i^{th} \) embedded fiber; \( n_{f} \) is the total number of fibers crossing the “mother” element.

A tri-linear stress-strain (\( \sigma \varepsilon_{\text{cable}} \)) diagram was used for modeling the fibers’ bond-slip behavior. This relationship was obtained from fiber pullout tests carried out for three distinct inclinations angles, \( \alpha \), (0°, 30° and 60°). In Figure 8 is depicted the procedure adopted to obtain the \( \sigma \varepsilon_{\text{cable}} \), where \( \varepsilon_{\text{cable}} \), \( s \) and \( l_{b} \) are, respectively, the embedded cable strain, the crack band-width and the steel fiber’s slip; \( \sigma \) is stress computed from the pullout force, \( P \), divided by the fibre’s cross sectional area, \( A_{f} \). The tension-strain law assigned to each embedded cable depends on the inclination angle, \( \theta \), between the cable and the normal vector of the active crack surface, \( \hat{n} \) (see Figure 9).

**Simulations**

The model performance is appraised by simulating the uniaxial tension tests already presented in this work. In Figures 10a and 10b are depicted, respectively, the mesh used exclusively for the concrete matrix phase and the three-dimensional mesh used for modeling the self-compacting concrete with the steel fiber contribution (CI30 series).

In the present mesh, Lagrangian 8-noded solid elements are used for modeling the plain concrete contribution. Since the specimen had a notch at its midheight, all the non-linear behavior was localized at the notch region, thus a 2×2×1 Gauss-Legendre integration scheme is used (1 integration point in the loading direction). The remaining solid elements are modeled with linear elastic behavior, and a 2×2×2 Gauss-Legendre integration scheme is adopted. The Cornelissen et al. (1986) softening law was used for
modeling the nonlinear behavior of SCC. The material properties of the plain concrete matrix used in the simulations are included in Table 3. These values were obtained taking into account the strength class registered for the Cf30 and Cf45 series.

On the other hand, the steel fibers are modeled with 3D embedded elements with two integration points (Gauss-Legendre). A nonlinear behavior is assumed for all the embedded elements. From the fiber pullout tests were ascertained three distinct \( \sigma-\varepsilon \) cable laws corresponding, respectively, to the studied fiber pullout inclination angles, \( \alpha \) (0°, 30° and 60°). Due to the impossibility of having a \( \sigma-\varepsilon \) cable law for every possible inclination angle, the \( \sigma-\varepsilon \) cable laws obtained from the pullout tests with an angle \( \alpha \) of 0°, 30°, and 60° was assigned, respectively, to the embedded cables with an orientation towards the active crack surface (\( \theta \)) ranging from (0°, 15°(, (15°, 45°( and (45°, 75°).

Figures 11 and 12 present the numerical simulation of the uniaxial tension tests of the Cf30 and Cf45 series, respectively. For the Cf30 series the results of the numerical simulations were in agreement with the experimental results. The predicted numerical tensile strength is nearby the upper bound limit of the experimental envelope. This is feasible since, during testing, is almost impossible to completely exclude eccentricities, thus the experimental post-cracking average strength is smaller than the correspondent numerical one. For the Cf45 series, two numerical simulations were carried out. The first one adopting the \( \sigma-\varepsilon \) cable laws obtained from the fiber pullout tests (in which fiber rupture was observed for a \( \alpha=30° \) and 60°). Up to a crack width of approximately 1.0 mm (0.0394 in), a fairly good agreement with the experimental response is observed. Significant stress decay is, however, observed after this crack width limit due, mainly, to the rupture of the embedded cables with an inclination \( \theta \) between 15° and 45°. It should be noticed that, the embedded cables with an inclination \( \theta \) between 45° and 75° have also rupture, but only for crack widths higher than 2 mm. Since for the uniaxial tension tests of Cf45 series, fiber rupture did not occur so often, due to lower matrix strength and a higher number of fibers crossing the crack's fracture surface, another simulation was carried out assuming that the embedded cables with an inclination \( \theta \) between 15° and 45° did not rupture. Assuming that fibers did not rupture, for Cf45 series; the quality of the simulation was improved. (Figure 12).

CONCLUSIONS

In the present work the tensile behavior of self-compacting concrete (SCC) reinforced with two distinct hooked ends steel fiber contents (30 and 45 kg/m³ (1.873 and 2.809 lb/ft³)) was characterized by performing displacement controlled tensile tests.

The stress-crack opening relationships were derived from the stress-displacement curves. Additionally, the residual stress and energy dissipation parameters able of indicating the effectiveness of fiber reinforcement mechanisms in these two composite materials were determined. The relatively high compactness of the matrix system of these SFRSCC, the number of effective fibers bridging the fracture surface and the results obtained in a previous research program dealing with fiber pullout behavior of this type of steel fibers were all taken into account to interpret the post-cracking tensile behavior of the tested composites. In general, a linear relationship between the post-cracking parameters and the number of effective fibers was observed. A strong dependency on the type of fiber failure mode and the stiffness of the matrix and number of effective fibers was detected, which justified the occurrence of a pseudo-hardening branch in the softening phase of the SFRSCC with the highest fiber content (Cf45 series), as well as the significant residual strength decay occurred in the SFRSCC with the lowest fiber content (Cf30). In fact, in Cf45 series the predominant failure mode was fiber pullout, while in Cf30 series fiber rupture was the main failure mode.

The SFRSCC tensile behavior was numerically modeled as a two phase material. The matrix was simulated with a 3D multi-fixed smeared-crack model, while fibers were modeled as discrete embedded short cables distributed into the concrete matrix FE-mesh according to a Monte Carlo method. A good agreement between the numerical and experimental results was obtained.
ACKNOWLEDGEMENTS

The first author acknowledges the support provided by the grant SFRH/BD/18002/2004. The study reported in this paper forms a part of the research program “PONTALUMIS - Development of a prototype of a pedestrian bridge in GFRP-ECC concept,” Project n° 3456, QREN. The Authors also acknowledge the support of Civitest Company on the production of the SFRSCC specimens.

REFERENCES


In the analysis of Table 2 it is verified that, in general, the stresses and toughness parameters increased with the fiber content, as it would be expected. The only exception was the peak stress which was nearby 10% lower for the Cf45 series, since there are other factors that contributed to this fiber reinforcement effectiveness. One of these factors is the crack width value. Such increase is not simply justified by the higher number of fibers crossing the crack surface for the Cf45 series.

As far as the dissipated energy, an important parameter in fiber reinforced composite materials, is concerned, a significant increase of the post-cracking stresses was observed with the higher fiber content. An increase of the work, in the pullout behavior of inclined fibers, depending on the inclination angle and embedded length, a fiber that is being pulled out is approximately equal to the crack width. Then, as it can be observed in Cunha et al. (2009), and the number of effective fibers, as well as the fibers that have ruptured. In spite of some researchers do not consider the ruptured fibers as "effective", in the authors' opinion they should be considered, since they are able to transfer forces between the concrete matrix and steel fibers.

Influence of the number of effective fibers on the post-cracking parameters

<table>
<thead>
<tr>
<th>Cf (kg/m³)</th>
<th>Nf</th>
<th>Nf&lt;sub&gt;eff&lt;/sub&gt;</th>
<th>σ&lt;sub&gt;fol&lt;/sub&gt; (MPa)</th>
<th>σ&lt;sub&gt;1,3mm&lt;/sub&gt; (MPa)</th>
<th>σ&lt;sub&gt;1,1mm&lt;/sub&gt; (MPa)</th>
<th>σ&lt;sub&gt;2,mm&lt;/sub&gt; (MPa)</th>
<th>σ&lt;sub&gt;1,mm&lt;/sub&gt; (N/mm)</th>
<th>σ&lt;sub&gt;2,mm&lt;/sub&gt; (N/mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Avg. 30</td>
<td>27</td>
<td>19</td>
<td>3.392</td>
<td>0.672</td>
<td>0.500</td>
<td>0.186</td>
<td>0.658</td>
<td>1.007</td>
</tr>
<tr>
<td>CoV</td>
<td>30.8%</td>
<td>28.8%</td>
<td>13.6%</td>
<td>45.1%</td>
<td>32.3%</td>
<td>33.7%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Avg. 45</td>
<td>67</td>
<td>39</td>
<td>3.019</td>
<td>0.719</td>
<td>0.520</td>
<td>0.250</td>
<td>0.500</td>
<td>0.724</td>
</tr>
<tr>
<td>CoV</td>
<td>16.1%</td>
<td>16.5%</td>
<td>9.9%</td>
<td>18.0%</td>
<td>13.6%</td>
<td>13.1%</td>
<td></td>
<td></td>
</tr>
<tr>
<td>k&lt;sub&gt;5%&lt;/sub&gt;</td>
<td>20</td>
<td>14</td>
<td>3.024</td>
<td>0.450</td>
<td>0.324</td>
<td>0.250</td>
<td>0.500</td>
<td>0.724</td>
</tr>
<tr>
<td>k&lt;sub&gt;5%&lt;/sub&gt;</td>
<td>58</td>
<td>34</td>
<td>2.768</td>
<td>1.036</td>
<td>0.972</td>
<td>1.189</td>
<td>2.356</td>
<td></td>
</tr>
</tbody>
</table>

Table 3 — Plain concrete properties used in the simulations

(1 MPa = 146 psi; 1 N = 0.2248 lb; 1 mm = 0.0394 in.)

<table>
<thead>
<tr>
<th>Property</th>
<th>Series</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>ρ = 2.4×10⁶ N/mm³</td>
</tr>
<tr>
<td>Poisson ratio</td>
<td>ν = 0.2</td>
</tr>
<tr>
<td>Initial Young modulus</td>
<td>41.3×10⁹ N/mm²</td>
</tr>
<tr>
<td>Compressive strength</td>
<td>71.1 N/mm²</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>4.6 N/mm²</td>
</tr>
<tr>
<td>Fracture energy</td>
<td>0.117 N/mm</td>
</tr>
<tr>
<td>Crack band-width</td>
<td>l&lt;sub&gt;c&lt;/sub&gt; = 5 mm (equal to the element height at the notch)</td>
</tr>
<tr>
<td>Threshold angle</td>
<td>30°</td>
</tr>
</tbody>
</table>
Figure 1 — Uniaxial tensile test setup: (a) general view and (b) location of displacement transducers (not scaled)

Figure 2 — Uniaxial tensile stress - displacement relationship for the Cf30 series
Figure 3 — Uniaxial tensile stress - displacement relationship for the Cf45 series

Figure 4 — Uniaxial stress - crack width relationship for the Cf30 series
Figure 5 — Uniaxial stress-crack width relationship for the Cf45 series

Figure 6 — Relationship between the total number of fibers and the number of effective fibers on the crack surface
Figure 7 — Relationships between the number of effective fibers and the post-cracking parameters: (a) peak stress, (b), (c) and (d) stress at 0.3, 1 and 2 mm crack width, respectively; (e) and (f) dissipated energy up to 1 and 2 mm crack width, respectively

(a) $\sigma_{\text{peak}}$ [MPa] vs. $N_{\text{eff}}$ [-]

(b) $\sigma_{0.3\text{mm}}$ [MPa] vs. $N_{\text{eff}}$ [-]

(c) $\sigma_{1\text{mm}}$ [MPa] vs. $N_{\text{eff}}$ [-]

(d) $\sigma_{2\text{mm}}$ [MPa] vs. $N_{\text{eff}}$ [-]

(e) $G_{\text{F1\text{mm}}}$ [N/mm] vs. $N_{\text{eff}}$ [-]

(f) $G_{\text{F2\text{mm}}}$ [N/mm] vs. $N_{\text{eff}}$ [-]
Figure 8 — Determination of the embedded cable’s stress-strain diagram based on the experimental pullout force-slip relationship

\[ \varepsilon_{\text{cable}} = \frac{s}{l_b} \]

Figure 9 — Three-dimensional scheme of the embedded cable intersecting an active crack

\( n \) is the vector normal to the crack plane
Figure 10 — Three-dimensional finite element mesh: (a) concrete phase and (b) concrete + fibers phases (Cf30 series; red lines represent the fibers)

Figure 11 — Numerical simulation of the Cf30 series’ uniaxial tension tests
Figure 12 — Numerical simulation of the Cf45 series’ uniaxial tension tests
Investigation of Steel and Polymer Fiber-Reinforced Self-Consolidating Concrete

by M. C. Brown, H. C. Ozyildirim, and W. L. Duke

Synopsis: Self-consolidating concrete (SCC) promises to shorten construction time while reducing the need for skilled labor. However, experience has shown that SCC may be prone to shrinkage cracking, which may compromise durability. In conventional concrete, fiber reinforcement has been used to control cracking and increase post-cracking tensile strength and flexural toughness. These benefits could be achieved in SCC without compromising the workability or stability, provided that the amount of fiber reinforcement is optimized.

This project sought to evaluate the feasibility of fiber reinforced self-consolidating concrete (FR-SCC) for structural applications. Tests were conducted in the laboratory to assess the fresh and hardened properties of FR-SCC containing various types and concentrations of fiber. The results indicate that SCC with high flowability and some residual strength beneficial for crack control can be prepared for use in transportation facilities. The results of the experiments further show that, at optimal fiber additions, FR-SCC mixtures can have the same fresh concrete properties as traditional SCC mixtures. FR-SCC also demonstrates a considerable improvement in the residual strength and toughness of a cracked section. Though not specifically measured, increase in residual strength and toughness is expected to lead to control of crack width and length (ACI 544.1R, 1996). The increase in the FR-SCCs’ cracked section performance indicates that it can be expected to have better durability in service conditions than an identical SCC without fibers. In transportation structures FR-SCC can be used in link slabs, closure pours, formed concrete substructure repairs; or prestressed beams where end zone cracking has been an issue.

Keywords: concrete; crack control; fiber-reinforced concrete; residual strength; self-consolidating concrete; strength; toughness
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William L. Duke is a senior pavement engineer with the Virginia Department of Transportation. He received his civil engineering degree from the University of Virginia and is an Engineer in Training.

INTRODUCTION

Self-Consolidating Concrete

Self-consolidating concrete (SCC) was first developed in Japan in 1986 by Professor Hajime Okamura of the Kochi University of Technology (Ozawa et al, 1989). SCC mixtures can have a high workability and stability allowing them to be placed without the necessity of mechanical vibration, even in situations which require large amounts of reinforcing steel or complicated formwork (Vachon, 2002). The technology also assists in cutting costs by speeding up construction while also reducing the amount of skilled labor necessary to complete the job (Koehler, 2005).

SCC can provide smooth surfaces making it desirable in architectural applications. In transportation structures SCC can simplify casting in precast, prestressed concrete beams designed for bridge structures where long spans between supports are ideal. These beams are characterized by very heavy reinforcement that is densely packed, which make the consolidation of conventional concrete mixtures very difficult, leading to unwanted large voids in concrete. A properly designed SCC mixture eliminates the need for additional consolidation energy (vibration and compaction) and both the economy and the durability of the beams can be improved (ACI 237R, 2007).

Fiber-Reinforced Concrete

Concrete is by nature a heterogeneous and brittle material. Concrete does not perform well in tension because it fractures easily. The resulting cracks, once induced, can propagate quickly under structural or thermal loading or under restraint in cases of concrete shrinkage or other dimensional instability. Cracks allow water and other substances, such as deicing salts, to readily enter the concrete and corrode the steel reinforcement or react detrimentally with the concrete, reducing the durability of concrete.

Fiber Reinforced Concrete (FRC) may mitigate this problem in two ways; fibers restrain cracks from opening and act as a substitute or supplement for conventional steel reinforcement within the concrete (Maingay, 2004; ACI 544.1R, 1996). Fiber reinforcement for concrete is typically made of either steel or synthetic filaments. In most concrete applications, these fibers are generally 1 to 2 inches (25 to 50 mm) in length and designed to be added during the mixing process. Because the fiber reinforcement is added during mixing, it provides uniform tensile reinforcement throughout the concrete member. Even if this reinforcement is not relied upon for structural strength, it serves to hold a cracked concrete section tightly together during loading. By keeping the cracks from opening as wide, the cracks have less effect on the concrete permeability, slowing the corrosion process. Also, because the fibers can, in some cases, be used as a replacement to traditional reinforcement, corrosion problems can be reduced or eliminated through the use of synthetic fibers which do not corrode in water (Concrete Paving Assoc., 2003). Steel fibers in cast-in-place concrete slabs and pavements, or various shotcrete applications, can improve flexural toughness, impact resistance and flexural fatigue endurance (ACI 544.1R, 1996). Synthetic fibers have been incorporated into concrete slabs on grade, floor slabs and stay-in-place forms or precast elements for multi-story buildings (ACI 544.1R, 1996).

Fiber Reinforcement in Self-Consolidating Concrete

Merging the benefits of fiber reinforcement with self-consolidating concrete technology may pay dividends in construction efficiency and durability of concrete elements. A problem with SCC is that it may have large amounts of fine material and small maximum coarse aggregate size. The resulting
comparatively large cumulative surface area onto which mix water will adhere may lead to greater water
demand and, subsequently, excessive drying shrinkage. However, SCC mixtures are often designed with low
water content for stability, which can serve to reduce potential for drying shrinkage. Thus, the shrinkage
characteristics of the SCC mixtures will depend upon mixture proportions and the intended application.
If shrinkage occurs, restrained stresses may result, which exceed the concrete tensile strength, resulting
in cracks. This cracking will increase the permeability and may lead to durability problems. Through the
addition of reinforcing fibers, cracking may be controlled (Slag Cement Assoc., 2005).

Construction with Fiber Reinforced Self-Consolidating Concrete (FR-SCC) may even be faster than with
SCC because the need for traditional steel reinforcement could potentially be reduced or eliminated.
In pavements and bridge decks designed with FR-SCC, the elimination of secondary reinforcing steel
can lead to sections that can be built faster than those built with conventional construction techniques
(Concrete Paving Association, 2003). These benefits have positive implications for reducing costs, through
reduction of labor and materials for reinforcement placement, and possible public and worker safety
enhancement through the reduction of overall construction times and required maintenance of traffic.

PURPOSE AND SCOPE
The purpose of this study was to investigate the properties of self-consolidating concrete containing
fibers and to determine the feasibility of using such concretes in transportation facilities. Synthetic
polymer and steel fibers were used at varying concentration in SCC.

METHODOLOGY
This section describes the mixture compositions and test methods. SCC mixtures with fibers were
prepared in the laboratory in two phases. In the first phase, four different mixtures of fiber reinforced
self-consolidating concrete (FR-SCC) were tested against a control mixture of regular SCC. Although
American Concrete Institute (ACI) suggests 22 in slump flow as adequate for most structural applications,
for this research, slump flow values exceeding 20 in were sought, which is sufficient for many typical
transportation applications (ACI 237R, 2007). In the second phase, mixture proportions and fiber content
were refined, and some alternate fibers were also investigated.

Mixtures
Concrete mixture proportions are given in Table 1 for Phases 1 and 2. They are expected to provide
a minimum compressive strength of 4,000 psi and meet the requirements of Virginia Department of
Transportation (VDOT) Class A4 concrete, common for bridge decks.

<table>
<thead>
<tr>
<th>Ingredient</th>
<th>Source</th>
<th>Phase 1</th>
<th>Phase 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Type I/II</td>
<td>439 (260)</td>
<td>439 (260)</td>
</tr>
<tr>
<td>Slag Cement</td>
<td>Grade 120</td>
<td>236 (140)</td>
<td>236</td>
</tr>
<tr>
<td>Coarse1</td>
<td>Lime stone (SG=2.81)</td>
<td>1535 (911)</td>
<td></td>
</tr>
<tr>
<td>Coarse2</td>
<td>Granite (SG=2.80)</td>
<td>1436 (852)</td>
<td></td>
</tr>
<tr>
<td>Fine1a</td>
<td>Natural sand (SG=2.60)</td>
<td>704 (418)</td>
<td></td>
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<tr>
<td>Fine1b</td>
<td>Natural sand (SG=2.60)</td>
<td>704 (418)</td>
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</tr>
<tr>
<td>Fine2</td>
<td>Natural sand (SG=2.61)</td>
<td>1436 (852)</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td></td>
<td>290 (172)</td>
<td>287 (170)</td>
</tr>
<tr>
<td>Air (%)</td>
<td></td>
<td>6%</td>
<td>6%</td>
</tr>
</tbody>
</table>

The mixture for phase 1 was based on a concrete mixture used at a nearby precast plant for precast
concrete pipe and manhole structures. It includes two types of fine aggregate with the same specific
gravity, but from two different sources for a better grade distribution. The ingredients were obtained
from the precast plant. The scope of the first phase was limited to the study of fibrillated structural
synthetic fiber (PF1), a 2-in-long monofilament fiber with an aspect ratio of 70 and specific gravity of 0.92,
manufactured from a synthetic blend of polypropylene and polyethylene resins. The monofilament fiber partially fibrillates during mixing, increasing the fiber surface area and strengthening the bond between the fiber and the concrete matrix. In the first phase, PF1 was used at 0.2%, 0.3%, 0.4% and 0.6% by volume (3.0, 4.5, 6.0 and 9.0 lb/yd³ or 1.8, 2.7, 3.6 and 5.3 kg/m³). It is recognized that, “Higher volume percentages of fibers have been found to offer significant property enhancements to the [synthetic] FRC, mainly increased toughness after cracking and better crack distribution with reductions in crack width.” (ACI 544.1R-96) Thus, researchers posited that mixing large quantities of this fiber may be possible, resulting in enhanced toughness, impact and fatigue resistance, and control of plastic shrinkage cracking with minimal effect on concrete workability.

In the second phase, the concrete mixture incorporated ingredients available in the laboratory. To achieve the high flow rates, a polycarboxylate-based high range water-reducing (HRWR) admixture was used. Mixtures also contained a vinsol resin for air entrainment to achieve an adequate air void system to resist freezing and thawing, as suggested by ACI (ACI 544.1R, 1996). An alternate synthetic polymer structural fiber (PF2) and two coated steel fibers were evaluated. The second synthetic fiber (PF2), employed in phase 2, is a monofilament fiber made of a polypropylene/polyethylene blend with a specific gravity of 0.92, fiber length of 1.6 inches, and aspect ratio of 90. It does not fibrillate. The polymer fiber (PF2) was added at 0.2% to 0.3% by volume (3.0 to 4.5 lb/yd³ or 1.8 to 2.7 kg/m³). The two coated steel fibers, one with twisted, triangular cross-section and one round with hooked ends, were used at concentrations of 0.3% to 0.5% by volume (40 to 66 lb/yd³ or 24 to 39 kg/m³).

Tests

Fresh concrete properties determined included standard slump or flow tests, mixture temperature, air content and unit weight (density). In the slump flow (ASTM C 1611) an inverted cone was used. Once a mixture met the target values for the fresh concrete properties, cylinders and beam specimens were cast for hardened concrete tests. These specimens included 4”×4”×16” (102 mm×102 mm×406 mm) prisms for flexural and residual strength and toughness (4 samples per mixture) (ASTM C 1609) and 4”×8” (102mm×204mm) cylinders for compressive strength (2 samples per mixture) (ASTM C 39) and splitting tensile strength (3 samples per mixture) (ASTM C 496), and 4”×4” (102mm×102mm) in cylinders for permeability (2 samples per mixture) (ASTM C 1202). Specimens were cured in the moist (100% humidity) room until the tests were carried out. Flexural strength and toughness were tested on a closed loop servo controlled loading frame with 4-point loading (third point loading) apparatus and “Japanese yoke” affixing 2 linear-variable displacement transducers (LVDTs) to measure deflection at midspan of the beam specimen. The toughness can be calculated from the load-deflection curves by integrating the area under the curve up to the specified endpoint, in this case L/150 (0.08 in or 2 mm).

RESULTS AND DISCUSSION

The results for each phase are explained under separate headings.

Phase 1

_Fresh Concrete Properties_ – The fresh concrete properties of the mixtures are summarized in Table 2. At fiber concentrations of 6 and 9 pounds per cubic yard (3.6 and 5.3 kg/m³), stable self-consolidating concretes were not possible. To retain the stability of the mixtures, workability was compromised, and the resulting concrete was treated as normal fiber-reinforced concrete (not SCC). SCC mixtures at these levels of fiber content were deemed infeasible.

<table>
<thead>
<tr>
<th>Fiber Concentration</th>
<th>Unit Weight</th>
<th>Air Content</th>
<th>Slump Flow Diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>% by Vol. (lb/yd³)</td>
<td>[kg/m³]</td>
<td>(% by vol.)</td>
<td>in (mm)</td>
</tr>
<tr>
<td>Control 0%</td>
<td>(0) [0]</td>
<td>5.6</td>
<td>22.0 (559)</td>
</tr>
<tr>
<td>0.2% (3)</td>
<td>144.8 (3.18)</td>
<td>4.0</td>
<td>21.5 (546)</td>
</tr>
<tr>
<td>0.3% (4.5)</td>
<td>146.0 (3.21)</td>
<td>3.5</td>
<td>22.0 (559)</td>
</tr>
<tr>
<td>0.4% (6)</td>
<td>144.5 (3.18)</td>
<td>-</td>
<td>*</td>
</tr>
<tr>
<td>0.6% (9)</td>
<td>145.2 (3.19)</td>
<td>3.6</td>
<td>*</td>
</tr>
</tbody>
</table>

*Concrete was not sufficiently workable to report flow values
Hardened Concrete Properties – The averaged results of the 28-day strengths, elastic modulus, and toughness are presented in Table 3. The compressive strengths were not affected by the fiber addition. Some observed differences were attributed to the inherent material variability of concrete mixtures. Similar behavior was observed with elastic modulus.

Table 3 Hardened Concrete Properties of Phase 1 Mixtures

<table>
<thead>
<tr>
<th>Strength Property</th>
<th>0 (0)</th>
<th>3 (1.8)</th>
<th>4.5 (2.7)</th>
<th>6 (3.6)</th>
<th>9 (5.3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression, psi (MPa)</td>
<td>8430 (58.1)</td>
<td>8963 (61.8)</td>
<td>8640 (59.8)</td>
<td>8560 (59.0)</td>
<td>8153 (56.2)</td>
</tr>
<tr>
<td>Elastic Modulus, ×10^6 psi (GPa)</td>
<td>4.48 (30.9)</td>
<td>4.66 (32.1)</td>
<td>4.96 (34.2)</td>
<td>5.07 (35.0)</td>
<td>4.49 (31.0)</td>
</tr>
<tr>
<td>Splitting tensile, psi (MPa)</td>
<td>722 (4.98)</td>
<td>863 (5.95)</td>
<td>872 (6.01)</td>
<td>867 (5.98)</td>
<td>785 (5.41)</td>
</tr>
<tr>
<td>Flexural at peak, psi (MPa)</td>
<td>1028 (7.1)</td>
<td>1167 (8.0)</td>
<td>1078 (7.4)</td>
<td>1295 (8.9)</td>
<td>1055 (7.3)</td>
</tr>
<tr>
<td>Residual at δ=0.02 in (0.5mm), psi (MPa)</td>
<td>0 (0)</td>
<td>113 (0.78)</td>
<td>77 (0.53)</td>
<td>150 (1.03)</td>
<td>310 (2.14)</td>
</tr>
<tr>
<td>Residual at δ=0.08 in (2mm), psi (MPa)</td>
<td>0 (0)</td>
<td>144 (0.99)</td>
<td>173 (1.19)</td>
<td>246 (1.70)</td>
<td>135 (0.93)</td>
</tr>
<tr>
<td>Toughness at δ=0.08 in (2mm), in-lb (N-m)</td>
<td>3 (0.34)</td>
<td>60 (4.68)</td>
<td>73 (5.25)</td>
<td>97 (7.10)</td>
<td>123 (8.39)</td>
</tr>
</tbody>
</table>

In concretes containing fiber reinforcement, a limited increase in the splitting tensile strength of the concrete was observed, though no clear relation to fiber concentration could be identified. At high concentrations of fiber reinforcement, fibers often exhibit a tendency to clump together during mixing, which can influence concrete workability, fiber distribution and bond between fibers and cement paste. Also the reduction in workability could result in increased amounts of entrapped air.

The flexural strength results, represented by individual sample points and the average as a line, show that the addition of fiber reinforcement in the tested range of concentration had no clear effect on the peak flexural strength of concrete.
The addition of fiber reinforcement has a tremendous impact on the residual strength. Data in Table 3 show a clear trend for deflection of 0.02 in and 0.08 in (0.5 mm and 2.0 mm) wherein the residual strength increased with fiber concentration, except at the L/150 (0.08 in or 2.0 mm) deflection (where L is the unsupported length) with the highest fiber addition which is attributed to poor consolidation (exemplified by greater than 2% entrapped air voids by volume) (Walker et al., 2004).

The flexural toughness is closely related to the residual strength of concrete. The residual strength plays the dominant role in the toughness of the test specimen, since the fibers absorb considerable energy prior to the loss of stress capacity.

**Phase 2**

*Fresh Concrete Properties* – Table 4 summarizes the fresh concrete properties for Phase 2 mixtures.

<table>
<thead>
<tr>
<th>Fiber Type</th>
<th>Fiber Concentration % by Vol. (lb/yd³) [kg/m³]</th>
<th>Fresh Unit Weight lb/ft³ (kg/m³)</th>
<th>Air Content (% by vol.)</th>
<th>Slump Flow Diameter in (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Control</td>
<td>0%</td>
<td>144.4 (85.7)</td>
<td>6.2</td>
<td>21.5 (546)</td>
</tr>
<tr>
<td>PF2</td>
<td>0.2%</td>
<td>140.8 (83.5)</td>
<td>8.0</td>
<td>20.5 (521)</td>
</tr>
<tr>
<td>PF2</td>
<td>0.3%</td>
<td>137.2 (81.4)</td>
<td>10.5</td>
<td>20.5 (521)</td>
</tr>
<tr>
<td>Twisted Steel</td>
<td>0.3%</td>
<td>142.4 (84.5)</td>
<td>6.0</td>
<td>23.5 (597)</td>
</tr>
<tr>
<td>Hooked Steel</td>
<td>0.5%</td>
<td>141.6 (84.0)</td>
<td>9.5</td>
<td>20.0 (508)</td>
</tr>
<tr>
<td>Hooked Steel</td>
<td>0.5%</td>
<td>143.6 (85.2)</td>
<td>6.8</td>
<td>21.0 (533)</td>
</tr>
</tbody>
</table>

Very small dosages of air entraining admixture were required for the relatively small laboratory batches, and a few of the mixtures exhibited higher air content than anticipated. All of the Phase 2 mixtures were able to attain the minimum slump required for SCC concrete without segregation or excessive bleeding.

*Hardened Concrete Properties* – The average compressive strength and modulus of elasticity results are presented in Table 5. The compressive strength was comparable for the tested mixtures, except for the batch containing 0.3% by volume polymer fibers (4.5 lb/yd³ or 2.7 kg/m³), which exhibited lower average compressive strength which could be attributed to lack of consolidation.
Steel and Polymer Fiber-Reinforced Self-Consolidating Concrete

Table 5 Hardened Concrete Properties of Phase 1 Mixtures

<table>
<thead>
<tr>
<th>Strength Property</th>
<th>Control at 0 %</th>
<th>PF2 at 0.2%</th>
<th>PF2 at 0.3%</th>
<th>Twisted Steel at 0.3%</th>
<th>Twisted Steel at 0.5%</th>
<th>Hooked Steel at 0.3%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compression, psi (MPa)</td>
<td>7553 (52.1)</td>
<td>7240 (49.9)</td>
<td>6247 (43.1)</td>
<td>7880 (54.3)</td>
<td>7213 (49.7)</td>
<td>8030 (55.4)</td>
</tr>
<tr>
<td>Elastic Modulus, ×10⁶ psi (GPa)</td>
<td>3.76 (25.9)</td>
<td>3.52 (24.3)</td>
<td>3.37 (23.2)</td>
<td>3.74 (25.8)</td>
<td>3.48 (24.0)</td>
<td>3.8 (6.2)</td>
</tr>
<tr>
<td>Splitting tensile, psi (MPa)</td>
<td>330 (2.28)</td>
<td>342 (2.36)</td>
<td>698 (4.81)</td>
<td>477 (3.29)</td>
<td>857 (5.91)</td>
<td>505 (3.48)</td>
</tr>
<tr>
<td>Flexural at peak, psi (MPa)</td>
<td>855 (5.90)</td>
<td>802 (5.53)</td>
<td>873 (6.02)</td>
<td>872 (6.01)</td>
<td>932 (6.43)</td>
<td>880 (6.07)</td>
</tr>
<tr>
<td>Residual at δ=0.02 in (0.5mm), psi (MPa)</td>
<td>0 (0)</td>
<td>108 (0.74)</td>
<td>261 (1.80)</td>
<td>481 (3.32)</td>
<td>665 (4.59)</td>
<td>481 (3.32)</td>
</tr>
<tr>
<td>Residual at δ=0.08 in (2mm), psi (MPa)</td>
<td>0 (0)</td>
<td>117 (0.81)</td>
<td>248 (1.71)</td>
<td>183 (1.26)</td>
<td>332 (2.29)</td>
<td>135 (0.93)</td>
</tr>
<tr>
<td>Toughness at δ=0.08 in (2mm), in-lb (N-m)</td>
<td>30 (3.4)</td>
<td>70 (7.9)</td>
<td>130 (14.7)</td>
<td>147 (16.6)</td>
<td>220 (24.9)</td>
<td>145 (16.4)</td>
</tr>
</tbody>
</table>

Elastic modulus values were similar, with the lowest modulus values corresponding to the two mixtures with highest fiber concentrations (0.3% = 4.5 lb/yd³ or 2.7 kg/m³ polymer and 0.5% = 66 lb/yd³ or 39 kg/m³ twisted steel fibers). The observed reductions in values are attributed to lack of consolidation and high fresh air content in the case of 0.5% twisted steel fiber.

Contrary to compressive strength and elastic modulus results, the splitting tensile strength tests clearly showed the benefit of the fibers. The trends illustrate a clear increase in tensile capacity attributable to higher fiber concentrations.

Typical flexural toughness plot of load versus mid-span deflection for 4-point bending of a beam containing no fibers would exhibit a sudden drop in load after the first crack. By contrast, the load-deflection curve for a concrete beam reinforced with 0.3% polymer fiber by volume exhibits the behavior shown in Figure 3, wherein the load drops significantly after initial concrete rupture; not completely to zero, but to a load level corresponding to the residual fiber strength. Continued loading at a constant strain rate results in the long strain plateau, which is continued under the test protocol past the target deflection of L/150, or 0.08 in.
A similar test of a concrete beam containing 0.5% twisted steel fibers exhibits behavior shown in Figure 4, wherein considerable residual strength is retained by the fibers, and the load capacity gradually tapers off under constant strain-rate loading until the target deflection is attained.

Flexural strengths were generally comparable, though the polymer fiber at 0.2% (3 lb/yd³ or 1.8 kg/m³) concentration averaged lower than Control with no fiber and the polymer mixture at 0.3% (4.5 lb/yd³ or 2.7 kg/m³). The air content of PF2 was higher compared to the control mixture, leading to the lower strength. The steel fibers appeared to provide an increasing strength above that of control as the fiber concentration increased, with mixtures containing both steel fiber types attaining similar flexural strengths at the matching concentration of 40 lb/yd³ (24 kg/m³).
Steel and Polymer Fiber-Reinforced Self-Consolidating Concrete

In plain concrete with no fibers, there is no residual strength. The concrete with polymer fibers exhibits modest residual flexural strength, which increases with fiber dosage. The steel fibers of both types show comparable and considerable residual flexural strength at 0.3% by volume (40 lb/yd³ or 24 kg/m³), and an even higher strength at 0.5% (66 lb/yd³ or 39.2) for the twisted fiber.

The residual flexural strengths at a deflection of L/150 (0.08 in or 2 mm) reveal comparable load carrying capacities of polymer fibers at 0.2% by volume and steel fibers of both types at 0.3% by volume. Higher concentrations of both polymer and steel fibers provided a modest increase in residual flexural strength at this level of deflection. Residual strengths indicate that there is some crack control capability after the first crack. However, a more useful measure of the benefit of fibers to control cracking is toughness, which reflects the total energy necessary to strain or elongate the fibers after the concrete has cracked. The twisted and the hooked steel fibers exhibit substantial toughness. Polymer fibers also provide benefit over non-fiber concrete. The toughness values estimated for the Control concrete represent the relatively small deflection capacity of the concrete under flexural load.

CONCLUSIONS

The results of the experiments show that, for an optimal fiber addition, FR-SCC mixtures can have comparable fresh concrete properties as traditional SCC mixtures. FR-SCC also demonstrates a considerable improvement in the residual strength and toughness of a cracked section that is expected to lead to control of crack width and length. The increase in the FR-SCCs’ cracked section performance indicates that it can be expected to have better durability in service conditions than an identical SCC without fibers.

The results indicate that FR-SCC can be easily produced at fiber reinforcement concentrations at or below 0.3% by volume (4.5 lb/yd³ or 2.7 kg/m³) of synthetic fiber or 0.5% by volume (66 lb/yd³ or 39 kg/m³) of steel fiber per cubic yard. With this level of fiber reinforcement, FR-SCC could be expected to have a residual flexural strength 25 to 30 percent of the flexural strength at first crack with no negative effect on either the workability or stability of the plastic concrete. The residual strength of the FR-SCC should help control the cracking problems typically associated with non-fiber reinforced concrete mixtures including SCC. Through crack control, FR-SCC is a much more viable construction material, particularly for applications such as bridge structures, where a 75-year service life is expected. To employ the technology in transportation structures, further study in field applications may be necessary to determine the residual strengths, and associated fiber concentrations, required to effectively limit crack widths.

FR-SCC mixtures may find application in a variety of structural and transportation applications where crack control and ease of placement are of concern. One potential application may be crack control in pavement slabs. Another may be to limit the extent of cracking in the ends of prestressed beams that occur due to thermal stresses and time-dependent behavior of the mixtures during curing. Further application may be use in closure pours on continuous bridge decks. Other uses may include precast concrete components where fibers may reduce or replace secondary reinforcement for temperature and crack control.

REFERENCES


Experience with Self-Consolidating High-Performance Fiber-Reinforced Mortar and Concrete

by W.-C. Liao, S.-H. Chao, and A. E. Naaman

Synopsis: Self-consolidating high performance fiber reinforced cementitious composites (SC-HPFRCC) combine the self-consolidating property of self-consolidating concrete (SCC) in their fresh state, with the strain-hardening and multiple cracking characteristics of high-performance fiber-reinforced cement composites (HPFRCC) in their hardened state. Two different classes of SC-HPFRCC are briefly introduced in this paper: concrete based and mortar based. They all contain 30 mm long steel fibers in volume fractions of 1.5% and 2%, and exhibit strain-hardening behavior in tension. These mixtures are highly flowable, non-segregating and can spread into place, fill the formwork, and encapsulate the reinforcing steel in typical concrete structures. Six concrete based SC-HPFRCC mixtures, with compressive strengths ranging from 35 to 66 MPa (5.1 to 9.6 ksi), were successfully developed by modifying SCC mixtures recommended in previous studies and using the available local materials. Spread diameter of the fresh concrete based SC-HPFRCC mixtures measured from the standard slump flow test was approximately 600 mm (23.6 in.). Strain-hardening characteristics of the hardened composites were ascertained from direct tensile tests. Three mortar based SC-HPFRCC mixtures with 1.5% steel fiber content were also developed and exhibited average compressive strengths of 38, 50 and 106 MPa (5.5, 7.2 and 15.3 ksi), respectively. Recent structural large scale laboratory applications (structural wall, coupling beams, panels etc.) made of SC-HPFRCC have demonstrated the applicability of these mixtures.

Keywords: fiber-reinforced concrete; high performance; HPFRCC; life cycle; self-compacting; self-consolidating, SCC; strain hardening
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Antoine E. Naaman, FACI, is Professor Emeritus in the Department of Civil and Environmental Engineering at the University of Michigan. He is a member of ACI Committee 544, Fiber Reinforced Concrete; 549, Thin Reinforced Cementitious Products and Ferrocement; and Joint ACI-ASC Committee 423, Prestressed Concrete. His research interests include high-performance fiber-reinforced cement composites and prestressed concrete.

INTRODUCTION

Self-consolidating high performance fiber reinforced cementitious composites (SC-HPFRCC) combine the self-consolidating property of self-consolidating concrete (SCC) in their fresh state, with the strain-hardening and multiple cracking characteristics of high performance fiber reinforced cement composites (HPFRCC) in their hardened state. Two different classes of SC-HPFRCC are briefly introduced in this paper: concrete based and mortar based. The mortar is defined as containing sand with maximum particle size of about 3 mm (1/8 in.) while the concrete has aggregates with a maximum size of 12.7 mm (0.5 in.). Both are highly flowable, non-segregating cementitious composites that can spread into place, fill the formwork, and encapsulate the reinforcing steel in typical concrete structures.

In this paper, the authors provide a summary of findings based on numerous laboratory trials. Six concrete based SC-HPFRCC mixtures, with compressive strengths ranging from 35 to 66 MPa (5.1 to 9.6 ksi), were successfully developed by modifying SCC mixtures recommended in previous studies and using the available local materials, including 30 mm (1.2 in.) long hooked steel fibers in volume fractions of 1.5% and 2%. Spread diameter of the fresh concrete based SC-HPFRCC mixtures measured from the standard slump flow test was approximately 600 mm (23.6 in.). Strain-hardening characteristics of the hardened composites were ascertained from direct tensile tests. Mortar based SC-HPFRCC with 1.5% steel fiber content and similar target characteristics, that are, self-consolidating in the fresh state and strain-hardening in the hardened state were also developed. They showed average compressive strengths of 38 and 50 MPa (5.5 and 7.2ksi), of 106 MPa (15.3 ksi), respectively.

SC-HPFRCC is being addressed as part of a project for the U.S. Network for Earthquake Engineering Simulation (NEES) with the objective to develop concrete based SC-HPFRCC mixtures that can be easily manufactured and delivered by ready-mix trucks for use on the job site, with particular application in seismic resistant structures. Recent structural large scale laboratory applications (structural wall, coupling beams, panels etc.) using SC-HFPRCC at several universities in the US and Canada, including University of Michigan, Stanford University, and University of Toronto, have demonstrated the applicability of these mixtures.

RESEARCH SIGNIFICANCE

It is generally agreed that adding fibers can expand the applications of SCC; however, a reduction in workability due to fiber addition may become a barrier for its application in practice. By modifying SCC mixtures recommended in previous studies and using locally available materials, several concrete and mortar based SC-HPFRCC mixtures for different strength demands can be easily manufactured and delivered by ready-mix trucks for cast-in-place applications. Structural large scale test results from several research programs also showed that the SC-HPFRCCs proposed in this paper are effective in increasing shear strength, displacement capacity and damage tolerance in members subjected to large inelastic deformations, with substantial reductions or even total elimination of transverse steel reinforcement (Parra-Montesinos 2005, Canbolat et al. 2005).
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SCC AND HPFRCC

Self-consolidating concrete (SCC) has a high flowability and a moderate viscosity, and has no blocking by the reinforcement during flow. In addition, SCC will de-air by itself during casting. The concept of SCC was proposed first by Okamura in 1986 (Okamura et al 1994). There are three key aspects of workability which should be carefully controlled to ensure satisfactory performance of SCC during its wet phase and for its successful classification as SCC (Ouchi et al 2003), namely: filling ability, resistance to segregation and passing facility. Significant water reduction ability of superplasticizer (SP) is essential to provide the necessary workability; high fluidity, however, can increase the tendency of a mix to segregate. Therefore maintaining homogeneity is an important issue for the quality control of SCC. Polycarboxylate Ether (PCE) based SPs represent a major breakthrough in concrete technology as they can reduce the water requirement by as much as 40% and impart very high workability without the undesirable effects of postponement and segregation (EFNARC 2002, Shah 2005). In addition, with proper use of viscosity modifying agents (VMA), SCC could achieve higher flowability and higher slump without segregation, and also maintain better slump retention, thus making concrete more durable due to its lower water/cementitious ratio, and consequently reducing sulfate attack and salt penetration (Massicotte et al 2000; Nowak et al 2005). Several international conferences have addressed various aspects of self-consolidating concrete over the past few years; we refer in particular to the proceedings of three conferences organized by the ACBM (Advanced Cement Based Materials) Center at Northwestern University [SCC 2002, 2005 and 2008] which contain a large number of useful studies with some addressing especially the use of fibers.

In recent years, considerable attention has been paid to fiber reinforced concrete/cement composites (FRCCs). In order to classify FRCCs based on their tensile performance, Naaman (1987, 1996, 2003, 2006) proposed a new class of FRCCs, referred to high performance fiber reinforced cement composites (HPFRCCs). The idea behind this new classification of FRCCs was to distinguish between the typical tensile performance obtained with traditional FRCCs, characterized by a softened response after first cracking, and the tensile strain-hardening response with multiple cracking exhibited by HPFRCCs. Typical tensile stress-strain responses of FRCC and HPFRCCs are illustrated in Fig. 1. In spite of similar initial ascending portions (from point 0 to point A), it should be noted that, after first cracking, HPFRCCs show a hardening portion (from point A to point B) up to relatively high strains, typically higher than 0.5%, while conventional FRCCs decay in strength. This unique portion (from point A to point B) of the stress-strain response for HPFRCCs characterizes the material’s “strain hardening” behavior, which leads to a high material toughness. Moreover, multiple cracks can be observed after first cracking throughout HPFRCC structural elements, as opposed to the localized cracks in conventional FRCCs.

Despite the development of equations to optimize packing density, yielding stress and viscosity of SCC without fibers by several researchers (Grünewald 2006; Markovic 2006; Petit et al 2007), a rational approach for optimizing fluidity as a function of fiber characteristics is still needed. While adding fibers can significantly enhance mechanical properties, particularly tensile behavior, of SCC, a reduction in workability due to fiber addition may become a handicap in practice. The mix design of SCC should be further adjusted if fibers are added (Markovic 2006). The addition of steel fibers into SCC mixtures has been studied by a number of researchers (Bui et al 2003, 2005; Busterud et al 2005; Chan et al 2004; Chern et al 2006; Corinaldesi et al 2004; Johnston 1996; Khayat 1999; Groth 1999; Grünewald 2006; Kuder et al 2007; Markovic 2006; Miao et al 2003; Ozyurt el al 2007). In addition, numerous commercial laboratories have been involved in the development of SCC with fibers and continuously improving their performance. Following is a summary of key findings based on their studies (Bui el al. 2003):

1. The coarse-to-fine aggregate ratio in the mix needs to be reduced so that individual coarse aggregate particles are fully surrounded by a layer of mortar. Furthermore, it is recommended by Johnston (1996) to reduce the volume of coarse aggregates at least 10 % compared with plain concrete to facilitate pumping.
2. Before addition of fibers, slump flow of SCC must be relatively high. The slump flow criterion of qualified SCC is 600mm (23.6 in) for a 300 mm high cone.
3. Everything else being equal, addition of fibers reduces slump flow of SCC; higher fiber volume and higher aspect ratio of fibers reduce slump flow of SCC as well, thereby leading to higher possibility of blocking and segregation.
Based on an extensive review of prior studies on SCC mixtures with fibers, six mixtures of concrete-based and three mixtures of mortar-based SC-HPF RCC (initially taken from prior studies and modified), were developed and are described next (Table 1). All mixtures contain either 1.5 % or 2.0 % steel fibers by volume and achieve a compressive strength (28-days) ranging from 35 to 106 MPa (5 to 15.3 ksi). Two types of hooked steel fiber with circular cross-section were used, one with normal tensile strength and one with higher tensile strength and higher aspect ratio. Their main properties are shown in Table 2. According to the criteria mentioned earlier, SCC in the fresh state should satisfy simultaneously the filling ability, segregation resistance and passing ability. These can be examined by various test methods, such as slump flow test, V-funnel test, L-box test, and J-ring test, etc. Only slump flow tests were carried out in this study to simulate simple field conditions. The slump flow test provides the most fundamental information regarding the flowability of SC-HPF RCC. A diameter of slump flow of 600 mm (23.6 in.) is generally accepted as a minimum requirement for SCC; since there is no accepted standard for fiber-reinforced SCC, the same value of 600 mm was targeted in this research. Visual inspection was used to observe if segregation occurs, particularly in relation to the fibers. The concrete based SC-HFPRCC mixtures were successfully used in large scale specimens, including coupling beams, shear walls, and wall panels at the University of Michigan. No significant problems were observed in flowability, segregation resistance, filling ability, and passing ability of the SC-HPF RCCs in these specimens even though a large amount of reinforcing steel bars, such as coupling beams, was present as shown in Figure 2. It is noted that due to the very close reinforcement spacing at the ends of the coupling beam (Figure 2a) which was smaller than the fiber length, a slight hand vibration with a rod was needed for full compaction. In addition to the commonly accepted compression tests, direct tensile tests on dog-bone shaped specimens were carried out in order to verify if these developed SC-HPF RCC mixtures could achieve the desired strain-hardening response.

Materials for the matrix

The cementitious materials used in this study were ASTM Type I and III Portland cement and class C fly ash. A silica fume (more than 97.5% SiO₂) was added only for the very high strength mortar based SC-HPF RCC. The coarse aggregate for the concrete mixtures had a maximum size of 12.7 mm (0.5 in) and consisted of either solid crushed limestone or pea gravel from a local source, with a density of about 2.70 g/cm³ (0.098 lb/in³). The fine aggregate was a #16 flint silica sand (ASTM 50-70). For the very high strength mortar based mixture a #100 flint silica sand and a silica powder were also used with proper gradation to improve the packing density of the resulting material. Three polycarboxylate-based superplasticizers (SP1, SP2 and SP3) were used in various mixtures as shown in Table 1, and indicate that several superplasticizers are compatible for the practical production of SC-HPF RCC. In addition to the superplasticizer, a viscosity modifying admixture (VMA) was used in some mixtures to enhance the viscosity and reduce fiber segregation in the presence of higher water to cementitious ratios. It is noted that the mixtures described in Table 1 were arrived at after numerous preliminary trials for each mixture.

Mix proportions

Six different concrete based (C1-C6) and three mortar based (M1, M2 and VH) SC-HPF RCC mixtures, were developed to cover a broad range of strength requirements. Mixture C1 was essentially taken, as a starting reference point, from Bui et al. (2005). The mix proportions recommended, after several preliminary trials for each mixture, are summarized in Table 1. In this study, the amount of superplasticizer (SP) and the ratio of water-to-cementitious materials were selected as primary means to modify the compressive strengths. Water to cementitious material ratios are also provided in Table 1. The binder comprised the cement, fly ash and silica fume (if present). The water portion in SP and VMA was not included in the calculation of water to cementitious material ratios because the exact content was not known. However, to be more accurate, the water should be included. In addition, the fiber volume fraction for all mixtures was larger than or equal to 1.5%. It is noted that all these proportions were developed based on the following philosophy, that is, to reduce the coarse-to-fine aggregate ratio in order to provide a well-developed paste layer which can fully surround individual coarse aggregates. This is because the amount of paste must be sufficient not only to fill the voids between aggregates, but also to fully cover the aggregate particles and the fibers.
Mixing procedure

According to the literature review of various mixing procedures for SCC, the sequence of placing the various materials in the mixer plays an important role, especially when higher volume fraction of fibers are added. The described advantages of two procedures found in prior studies (Grünewald 2006; Markovic 2006; Sahmaran et al. 2005 and 2007) were incorporated in this study, namely: 1) Premix all dry components for the paste, that is cement, fly ash, fine sand, silica or glass powder (if present); and 2) Pre-mix water, SP, and VMA (if needed), then pour the resulting fluid in several steps in order to develop a homogenous matrix without paste lumps before adding the coarse aggregates and fibers.

In this study the premixed liquid (Water + SP + VMA if needed) was added in several steps as described below. This allowed supervision of the status of the mixtures in order to limit paste lumps. The following steps were used (see also Figure 3):

1. Dry-mix the cement, fly ash, silica sand, silica powder and silica fume (if present) for 30 seconds.
2. Pour 1/2 of liquid (Water+SP+VMA) in the mixer. After mixing for about 1 minute, pour 1/4 of the remaining liquid (Water+SP+VMA).
3. After mixing for about 1 minute, pour 1/8 of liquid (Water+SP+VMA).
4. After mixing for about 1 minute, pour 1/16 of liquid (Water+SP+VMA).
5. After mixing for about 1 minute, pour all of the remaining liquid (Water+SP+VMA).
6. After mixing for about 1 minute, add all coarse aggregates in the mixer.
7. After mixing for about 2 minutes, slowly add all steel fibers in the mixer.
8. Continue mixing for about 2 minutes after all the fibers have been added. The mixture is then ready for pouring.

The status of a typical mixture at each step for concrete base SC-HPFRCC is illustrated in Figure 3. Note that if other types of SP and VMA are used, it is important to ascertain from the manufacturer’s guidelines that they could be mixed together with the water in the same container, in order for the above procedure to succeed. Moreover, it is known that fiber orientation can affect the mechanical performance of the composite (Naaman 2003). In this experimental program, all casting flows were kept the same and the resulting fiber orientation can be considered somewhere in between 2D and 3D.

Testing

The slump flow test (EFNARC 2002) on the fresh mixture, and compression and direct tensile tests on the hardened composite were carried out to estimate flowability, compressive strength, and tensile response, respectively.

The slump flow test was the easiest and most familiar way to evaluate the horizontal free flow (deformability) of SCC in the absence of obstructions. The diameter of the spread concrete is measured in two perpendicular directions and recorded as slump flow. In general, the average of diameters in two perpendicular directions should be larger than 600 mm (23.6 in) for qualified SCC. The slump flow test results are shown in Table 3.

The compressive strength of hardened concrete was determined from compression tests on standard cylindrical specimens 102 × 204 mm (4 × 8 in.) according to ASTM C39 (ASTM 2005). The direct tensile tests were needed to ascertain that the developed SC-HPFRCCs give a strain-hardening response in tension after first cracking. Dog-bone shaped tensile specimens were prepared and tested for each SC-HPFRCC mixture (Naaman et al. 2007). These specimens have a cross-sectional dimension of 25.4 × 50.8 mm (1 × 2 in.) as shown in Figure 4(a). The applied load was monitored by the load cell of the testing machine and the elongation was recorded by a pair of LVDTs attached to the specimen (Figure 4(b)), with a gauge length of about 178 mm (7 in).

Experimental results

As described earlier, slump flow of SC-HFPRCC can be recorded as the average diameter of the concrete in two perpendicular directions. It was measured for each mixture, and its value is shown in Table 3. Table 3 also gives the average compressive strength of each SC-HPFRCC mixture. Figure 5(a) to 5(c) show typical stress-strain curves in compression of the concrete-based mixtures, mortar-based mixtures, and the very-high strength mortar-based mixture, respectively. It can be observed that, except
for VH, SC-HPFRCCs behave as well-confined reinforced concrete in compression. For the VH mixture, a higher fiber content may be needed to achieve the same result.

In terms of tensile tests, the stress-strain curves were recorded from the dog-bone specimens tested. Multiple cracks developed up to peak stress (post-cracking strength) at which crack localization occurred (Figure 7). Typical stress-strain curves are shown in Figure 6. It can be observed that the tensile stress increases with an increase in strain after the first crack. Thus these mixtures all satisfy the requirement of strain-hardening behavior of HPFRCC. Beyond the peak stress, the tensile stress dropped gradually due to fiber kinematic pullout from the matrix. Some key results are summarized in Table 4.

ADVANTAGES IN STRUCTURAL APPLICATIONS

Structural tests using HPFRCC members under reversed cyclic loading showed that these members develop significantly higher shear deformation capacity and superior damage tolerance compared with RC members (Wight et al. 2003; Canbolat 2004; Canbolat 2005; Parra-Montesinos 2005). The excellent mechanical properties of SC-HFPRCC also offer the opportunity to significantly simplify the design and construction of coupling beams, while ensuring adequate ductility and damage tolerance (Lequesne et al. 2009). Moreover, even with less reinforcement, finer and denser multiple cracks were observed in the specimens made of SC-HPFRCC compared with those made of conventional concrete, which exhibited only a few cracks before final failure. In terms of sustainability, not only the use of fly ash and silica fume in significant proportion, but also the lower cracking potential and excellent crack width control (Chao et al. 2007) points to the potential of SC-HFPRCC as a green construction material. Other studies (Li 2004; Lepech et al 2005) also showed that the life cycle cost and primary energy consumption of HFPRCC were substantially lower than conventional materials. While the above advantages have served rather well compared to the conventional concrete, it is believed that SC-HPFRCC could push the practice achieving higher levels of performance, safety, and economy including life cycle costs.

SUMMARY AND CONCLUSIONS

Based on the experimental studies and analyses, the following conclusions can be drawn:

1. **Flowability**: only the slump flow test was used in this study to observe workability. Flow diameters about 600 mm were achieved in most cases. While the flowability of SC-HPFRCCs was not as high as the conventional SCC without fibers, it was deemed sufficient for practical implementation, with slight vibration when needed. In addition, larger scale specimens have been cast by using these mixtures described here, and the flowability was quite satisfactory with no segregation observed.

2. **Segregation**: segregation of fibers was greatly reduced by using viscosity modifying agent, VMA. The mixtures became viscous enough to bring fibers to the edge of the slump base plate during slump flow test. No segregation was observed either in this experimental program or other large scale specimens which were carried out by other researchers using the same mixtures.

3. **Mixing Procedure**: a strict mixing procedure was followed in this study, in terms of time of mixing, addition of components, and sequence of material addition. In order to achieve similarly good quality of fresh SC-HPFRCC mixtures, it is essential to strictly follow the recommended mixing procedure.

4. **Mechanical Properties**: specimens made from the hardened composites were tested for compressive strength and tensile stress-strain response. The SCHPFRCCs developed have compressive strengths ranging from about 35 to 106 MPa (5 to 15.3 ksi) and a tensile strengths ranging from 3.5 to 8.9 MPa (500 to 1300 psi). They also showed strain-hardening response in tension, accompanied by multiple cracking. The peak strain capacity after first cracking in tension ranged from 0.25% to 0.6%.

ACKNOWLEDGEMENTS

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Experience with Self-Consolidating Mortar and Concrete

of Damage Tolerant Fiber-Reinforced Cementitious Materials for New Earthquake-Resistant Structural Systems and Retrofit of Existing Structures”, with James K. Wight, University of Michigan, Ann Arbor, as Principal Investigator and Project Director. Other Principal Investigators include Gustavo Parra-Montessinos, University of Michigan, Sherif El-Tawil, University of Michigan, Sarah Billington, Stanford University, and James Lafave, University of Illinois, Urbana-Champaign. Also we acknowledge with thanks Sang-Yeol Park from Cheju National University in Korea for his contribution.

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### Table 1(a) – Proportions by weight of cement for SC-HPFRCC mixtures used

<table>
<thead>
<tr>
<th>Mix Proportions by weight of cement</th>
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<th>Mortar based</th>
</tr>
</thead>
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</tr>
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</tr>
<tr>
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</tr>
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<td>Silica Fume</td>
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</tr>
<tr>
<td>Fine Aggregates</td>
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</tr>
<tr>
<td></td>
<td>Silica Sand #100</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Silica Powder</td>
<td></td>
</tr>
<tr>
<td>Coarse Aggregates</td>
<td>Crushed Limestone</td>
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</tr>
<tr>
<td></td>
<td>Pea gravel</td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>Water</td>
<td>0.45</td>
</tr>
<tr>
<td>Water to Cementitious ratio</td>
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<td>Total Weight</td>
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</tr>
<tr>
<td>$V_f$ (%)</td>
<td></td>
<td>1.96</td>
</tr>
</tbody>
</table>

Cement: Type I 1, Type III 1 1 1 1 1 1 1
Mineral Admixture: Fly Ash 0.48 0.5 0.5 0.875 0.67 0.875 0.5 0.875
Silica Fume 0.25
Fine Aggregates: Silica Sand #16 1.7 1.7 1.7 2.5 2.1 2.2 1.7 2.2
Silica Sand #100 1
Silica Powder 0.27
Coarse Aggregates: Crushed Limestone 1.1 1 1 1.25 1.2
Pea gravel 1.1
Water to Cementitious ratio [W / (C+SF+FA)] 0.30 0.4 0.4 0.45 0.4 0.43 0.3 0.32 0.26
Total Weight 5.082 5.152 5.0635 6.8455 5.871 6.433 3.856 4.947 3.054
$V_f$ (%) 1.96 1.92 1.50 1.50 1.50 1.50 1.50 1.50 1.50
**Table 1(b) – Proportions in kg/m³ for SC-HPFRC mixtures use**

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<tr>
<th>Series ID</th>
<th>C1</th>
<th>C2</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
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<th>M1</th>
<th>M2</th>
<th>VH</th>
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<td>274.6</td>
<td>326.4</td>
<td>311.2</td>
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<td>1.96</td>
<td>1.92</td>
<td>1.50</td>
<td>1.50</td>
<td>1.50</td>
<td>1.50</td>
<td>1.50</td>
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*1 kg/m³=0.0624 lb/ft³= 1.6856 lb/yd³*

**Table 2 – Properties of fibers used in this study**

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<tr>
<th>Fiber ID</th>
<th>Type</th>
<th>Diameter, (mm)</th>
<th>Length, (mm)</th>
<th>Density, (g/cc)</th>
<th>Aspect ratio</th>
<th>Tensile Strength, (MPa)</th>
<th>Elastic Modulus, (GPa)</th>
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<td>Hooked</td>
<td>0.38</td>
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<td>79</td>
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<td>200</td>
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<td>Hooked</td>
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<td>30</td>
<td>7.85</td>
<td>55</td>
<td>1100</td>
<td>200</td>
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*1 mm=0.0393 in.; 1 g/cc=0.03617 lb/in³; 1 MPa = 0.145 ksi; 1 GPa=145 ksi*
<table>
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<tr>
<th>Fiber Type</th>
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<th>Mortar based</th>
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</tbody>
</table>

*1 mm = 0.0393 in.; 1 MPa = 0.145 ksi

Figure 1 – Stress-strain response of conventional FRC and HPFRCC (Naaman, 2003)
Figure 2 – Application of SC-HFPRCC to coupling beams for seismic resistance
(a) reinforcement layout (b) pouring (c) finishing
Figure 3 – Typical mixing procedure for concrete based SC-HPFRCC

Step 1. Dry mix with Cement, Fly Ash, and Sand

Step 2. Add 1/2 of (Water+SP+VMA)

Step 3. Add 1/2 of the rest of (Water+SP+VMA)

Step 4. Add 1/2 of the rest of (Water+SP+VMA)

Step 5. Add 1/2 of the rest of (Water+SP+VMA)

Step 6. Add the all rest of (Water+SP+VMA)

Step 7. Add Coarse Aggregates

Step 8. Add Steel Fibers

Step 9. Finish Mixing

Uniform fibers and coarse aggregate can be observed on the surface.
Figure 4 – (a) Geometry and dimensions of tensile test (dog-bone) specimen; (b) Tensile test set up

Figure 5 – Compressive stress-strain curves of SC-HPFRCCs: (a) concrete based (b) mortar based (c) very high strength mortar based
Figure 6 – Tensile stress-strain curves of SC-HPFRCCs: (a) concrete based (b) mortar based (c) very high strength mortar based
Figure 7 – Typical multiple cracks observed in SC-HPFRCC specimens
Design and Construction Aspects of Steel Fiber-Reinforced Concrete Elevated Slabs

by B. Mobasher and X. Destrée

Synopsis: Applications of slabs supported on piles are quite common for areas where soil-structure interaction may create differential settlement or long term tolerance issues. An application for the use of steel fiber reinforced slabs that are continuous and supported on piles is discussed in this paper. The experience and design methodology for slabs on piles is further extended to floor slabs of multi-story buildings, where a high dosage of steel fibers (50-100 kg/m³, 84-168 lbs/ft³) is used as the sole method of reinforcement.

Suspended ground slabs are generally subjected to high concentrated point loading (150 kN, or 33.7 kips) intensities as well as high uniformly distributed loadings (50 kN/m² or 1000 lb/ft²) and wheel loads. The span to depth ratios of the SFRSS is between 8 and 20 and depends on the loading intensity and the pile/column capacity. Standard procedures for obtaining material properties and finite element models for structural analysis of the slabs are discussed. Methods of construction, curing, and full scale testing of slabs are also presented.

Keywords: elevated slabs; ground slabs; point loading; soil structure; steel fiber-reinforced slabs; steel fibers
INTRODUCTION

This paper addresses the use of steel fiber-reinforced suspended slabs (SFRSS) in applications addressing pile supported continuous and elevated slabs. Pile supported continuous slabs are used for factories, warehouses, and basements where the areas underneath the slab are not accounted as a useful part of the structure. Elevated slabs are used as floors of multistory buildings, and both systems use steel fiber reinforced concrete as the main source of flexural reinforcement. In applications such as multi-story buildings, a high dosage of steel fibers (50-100 kg/m³, 84-168 lbs/ft³) is used as the sole method of reinforcement.

Pile supported slabs are quite common in areas where soil-structure interaction may create differential settlement or long term tolerance problems. It is now estimated that 20% of industrial concrete floors are supported on piles. The piles are installed since the ground does not offer the required bearing capacity and the structural slab is designed without any ground support. Furthermore, some soils are so weak that within a period of time, their plastic settlement under the slab would lead to loss of all contact. In other cases where the soil is expansive such as swelling clays, SFRSS slab has to be installed above the top level of the ground. The span to depth ratios of the SFRSS is between 8 and 20 and depends on the loading intensity and pile capacity. Suspended ground slabs are generally subjected to high concentrated point loading (150 kN, 33.7 kips) intensities as well as uniformly distributed loadings (50 kN/m² or 1000 psf) and wheel loads.

The first set of guidelines for slabs on grade that addresses SFRC use were based on methods in TR 34 developed in 1992 in UK. This design methodology has been used on several million square meters. The success is evident as SFRC slabs resist high moment intensities as well as high shear and punching shear stresses.

Using the same approach, one can expand the application for SFRSS slabs as cast in place elevated slabs E-SFRSS for elevated floors. Typical applications are for condominium buildings, office towers, and schools when loading intensities are light to moderate from 4-8 kN/m² (80-160 psf). A typical example is shown in Figure 1 for a multi-story high rise condominium building. The E-SFRSS sections are flat plates with a span to depth ratio of up to 30. Shrinkage cracking control is directly obtained by the use of fiber reinforcement. Large bay areas are possible due to shrinkage control and directly result in a reduced number of construction joints. Additional savings are also obtained in the preparation of the drawings as rebars do not need to be detailed or inspected.
Fig. 1—Suspended elevated flat slab (SFRSS) using steel fibers

The design procedure is based on full-scale testing and/or large scale round panel tests to verify the calculations. Full scale experiments were initially conducted in Belgium and Holland where many industrial slabs are built on pile supports due to low bearing capacity of the shallow ground. A combination of results from small scale flexural beams, round panel tests, finite element modeling, and full scale testing of above-ground suspended slabs leads to better understanding of the mechanisms involved, and a comprehensive design methodology for elevated slabs based on post-cracking response and ductility of slabs subjected to flexure.

**CONSTITUTIVE PROPERTIES OF MATERIALS AND STRUCTURAL MECHANISMS**

Construction materials require standards for measuring material properties and guidelines for structural design. In order to develop standardized procedures for design with FRC materials, test methods have been developed for quality control and calibration of material properties. Efforts are under way by various research groups to develop design guides.

Slabs with two lateral dimensions that are much larger than the thickness are quite appropriate for fiber reinforcing since the fiber orientation tends to become preferential in the plane. Stress concentrations in slabs are far less than in beams, so that SFRC can more adequately reinforce plates than the beams. The structural slab behavior results from the two way action, thus moments $m_x$, $m_y$, and $m_{xy}$ interact together with membrane and arch actions.

There are two main ductility aspects in the design of fiber reinforced concrete structures; defined in terms of materials level, and structural level ductility. To differentiate between these, two competing mechanisms operating at the material level are considered. For plain concrete, or in cases with low fiber contents, initiation of the tensile crack may result in localization of deformations such that as soon as cracks are formed, they tend to open, and propagate. Under these circumstances, the material is viewed as brittle. As the fiber content increases, propagation of newly formed cracks requires a much higher level of energy than what was used to create them. Under these circumstances, more new cracks are formed, whereas the existing cracks which require additional energy to propagate behave as stationary and remain inactive. This results in multiple crack formation as opposed to the propagation of a dominant single crack. Under this situation, the cracking diffuses and distributes within a band of high tensile stress regions in the elevated slab.

The second mode of ductility is due to the structural aspect. The geometry and the degree of indeterminacy of the structure play a dominant role in its response. If the structure is statically determinate as in the case of a simply supported beam, the first crack that localizes into a yield line
forms a collapse mechanism. However, if the degree of structural indeterminacy increases due to support fixity, continuous beams, or cases such as joint free slabs, then a minimum number of yield lines are needed to initiate a collapse mechanism. The contribution of the fibers is thus enhanced by the degree of indeterminacy of the structure and increases the load carrying capacity. As far as testing is concerned, small beam specimens, whether notched or un-notched, may not appropriately represent the potential load carrying capacity of a SRFC slab since multiple cracking mechanisms and load redistributions cannot be replicated in a determinate beam specimen. These mechanisms must be addressed by incorporating the post cracking ductility in the design process.

Several types of panel tests\textsuperscript{17,18,19} have been developed to simulate the stress state in plate type specimen. These tests have been used to evaluate the flexural capacity of fiber reinforced concrete. There is however a lack of theoretical models to predict the flexural behavior. Test results produce a load deflection response in addition to the total energy absorption as a material index. As the size, thickness, and fixity of the panel changes, the load, deflection, and energy absorption also change. Therefore, the quality of fiber reinforced concrete produced in various sizes, fiber dosage rates, and shapes cannot be compared due to the ambiguity of test methods. Back-calculation procedures based on the finite element method work to obtain the material properties form these test results.\textsuperscript{20} The same finite element procedures can be extended to accomplish the design of slabs as well.

**Moment capacity based on backcalculation**

The design of SFRSS up to now has been addressed by experimental validation with full scale testing. Large size specimens and full scale slab testing however are costly and make standardization of the design methodology impractical. Test results of round slabs supported continuously along the edge in flexure can provide the resisting moment intensities for design. Analytical solutions for predicting the flexural behavior of a round panel test in the elastic and cracked stages are based on elastic solutions and limit analysis approaches. Alternatively, inverse analysis using the finite element method can be employed to obtain the material properties from the response of the round panel tests.\textsuperscript{20} The proposed design methodology follows a three tier approach:

1. Experimental testing using three-point bend specimens, round panel specimens, and/or continuous supported slab to yield experimental load-deflection of the SFRC material.
2. Back-calculation procedure to obtain stress-crack width relationship, or simplified stress strain response to compute the experimental moment curvature response can be directly incorporated into the design procedures.
3. Use of finite element method or plastic analysis procedures for the calculation of capacity of a structure given the boundary conditions or simplified patterns of failure using yield line analysis.\textsuperscript{20}

Fig. 2—Concrete model in ABAQUS (a) tensile stress strain model; (b) tensile stress crack width model
The experimental load deflection response of round panel configurations has been modeled using finite element software such as ABAQUS. The material parameters for concrete model as shown in Figures 2(a) and 2(b) are obtained. In order to reduce the computational time, the symmetry of the loading condition was used to model a wedge of 22.5° of the section representing a 1/8 model, or a slab with 8 discrete supports. During the preliminary stages of the modeling, this assumption was checked against loading conditions representing: three point supported slab of ASTM C1550, three-point bending flexural beam, and a slab continuously supported along its edges. There are three main tasks as follows:

1. Model the load-deflection response of a fiber reinforced concrete round panel on continuous support and/or 8 discrete supports subjected to a point load at the center. This task yields the stress crack width relationship or moment curvature response from the load deflection response. The moment curvature relationship can be used in the round panel limit analysis.

2. Model fiber reinforced concrete three point bending test as an alternative approach to obtain a stress-crack width or moment-curvature relationship for the round panel rigid crack model.

3. Develop procedures for rigid crack models to compare the flexural behavior using limit analysis with the finite element result.

Use of round panel tests to obtain material parameter for composite using the finite element model of a slab continuously supported along its edges is discussed elsewhere. The experiments were based on six panels tested at the University of Brussels in Belgium. Specimens with a thickness of 150 mm (5.9"), radius of 790 mm (31") and simply supported at a radius of 750 mm (29.5") were tested. The results of slabs containing 80 kg/m³ (134 lbs/yd³) steel fibers are shown in Figure 3(a). The stress-crack width inverse analysis fits the load deflection results. The tensile stress-crack width model from inverse analysis is shown in Fig. 3(b) and used in the structural design or calculation of moment curvature response.

![Fig. 3—Inverse analysis of round panel test to obtain material parameters for concrete model in ABAQUS; (a) load deflection response of mix (volume fraction Vf=80 kg/m³, 134 lbs/yd³) and (b) Backcalculated tensile stress crack width model from inverse analysis.](image-url)
Moment Capacity Based On Direct Calculation

Comprehensive sets of procedures for design of slabs based on plasticity approach are available. Recent tools which enable derivation of the allowable moment capacities can also be used directly in the design of elevated slabs. The procedures use closed form equations that relate the tensile and compressive response of fiber reinforced concrete to its flexural response using small notched specimens in flexure. Plastic analysis based on Johansen and more recent publications addressing yield line analysis of slabs are then used in order to relate the section properties to the load carrying capacity of the slabs.

Strain softening materials such as SFRC and synthetic macro fiber reinforced concrete contain fibers below the critical level, hence the contribution of fibers is apparent in the post peak region. A simplified model for the uniaxial tensile and compressive stress strain results in closed-form solutions for the moment curvature relationship. A linear elastic response up to a point of tensile cracking is followed by a constant stress level at the post cracking region.

The tension model as shown in Figure 4.a, consists of a linear stress-strain response up to cracking strain \( \varepsilon_{cy} \), followed by a constant stress level post crack \( \sigma_p = \mu \varepsilon_{cr} E \) with parameter \( \mu (0 \leq \mu \leq 1) \) representing the residual strength. Young’s modulus for compression and tension are equal and the compressive strength is represented as a multiple of tensile strength defined using the compressive strain at yield \( \varepsilon_{cy} = \omega \varepsilon_{cr} \) with \( \omega \geq 1 \) and elastic perfectly plastic beyond this level as shown in Figure 4(b). Design applications utilize the full-scale moment-curvature relationship based on estimating the moment capacity at each compressive strain \( (\lambda_{cu}) \) level. The neutral axis depth, curvature, and moment capacity at an infinite strain \( (\lambda_{cu} = \infty) \) is derived as:

\[
k_u = \frac{\mu}{\omega + \mu}, \quad m_u = \frac{3 \omega \mu}{\omega + \mu}, \quad \phi_u = \infty
\]  

In accordance to LRFD approach, the ultimate strength of the section must be greater than the factored moment \( M_u \) calculated from loads. The internal forces are obtained by structural analysis of factored loads based on coefficients according to ACI Sec 9.2. The equation for nominal moment capacity can be derived by using \( m_u \) from Eq. (1) and the cracking moment capacity defined in Equation 3. A material reduction factor \( \phi_p \) introduced in equation 2 is used for the ultimate moment capacity. Thus, the deign equation for nominal moment capacity \( M_p \) along a yield line of the cross section can be obtained as:

\[
M_p = \phi_p M_n = \phi_p m_u M_{cr} = \phi_p \frac{3 \omega \mu}{\omega + \mu} M_{cr} \geq M_u
\]
Design and Construction Aspects of Steel Fiber-Reinforced Concrete

Where the first cracking moment $M_{cr}$ is defined as:

$$M_{cr} = \frac{\sigma_{cr} R d^2}{6}$$

With $R$ and $d$ the radius and the thickness of the round panel. In the case of a three point bend specimen parameter $R$ is replaced with $b$, the width of the specimen.

Anti-progressive collapse reinforcement

North American standards (USA and Canada), require that all free suspended elevated slabs regardless of their reinforcing details, be provided with minimum positive moment reinforcing in order to prevent the catastrophic collapse in case of a column support rupture by accident or terrorist attack. Reinforcing bars are installed at the bottom of slab from column to column in each direction, following the column grid and crossing each other above the column foot print. Generally, with a span to depth ratio of 30, this requirement consists of a set of two or three rebars for each direction. The guidelines for design of cross sectional steel area are in accordance to CSA, article 13.4.9. The contribution of positive moment rebars to the static resistance under service loadings conditions is neglected.

Mix design recommendations

The mix design procedure consists of the following ingredients:

1. Cements: Type I, II, IS, in 350 kg/m³ (583 lbs/yd³) concentration in order to secure a strength of C35-C40 class (minimum $f'_c = 35$ MPa, 5000 psi).

2. Water content and Slump: Water to cement ratio (W/C) of less than 0.50 and a slump in the range of 50-75 mm (2-3 in) prior to addition of fibers and admixtures at the jobsite. Prior to mixing the steel fibers in the truck mixer, the slump shall be a minimum of 225 mm (8.85 in). A trial mix at the laboratory of the concrete plant shall be organized to check water demand.

3. Aggregate gradation: The aggregate grading curve shall be continuous with at least 6% passing 125 microns (5 mils), 9% 250 microns (10 mils), 30% 1 mm (#30), 50% 4 mm (#8), and 20 mm (3/4") as the maximum size. In the case of pumping, the amount of fibers passing at 125 microns (#200 sieve) may need to be increased to 8%. The minimum hose size for pumping is 125 mm (5") and no reducers on the pump lines are recommended. In case of pumping, it is essential that the aggregate grading curve is held to the upper limit of the grading envelope as much as possible.

4. Admixtures: Addition of superplasticizer at a rate of 0.7%-1.5% of cement in order to increase the plain concrete slump from 75 mm to 225 mm (3-9 in) prior to adding the fibers into the truck mixer is recommended. Admixture concentration must be compatible with the installation method.

5. Steel fibers: Various fiber types are used as the sole means of reinforcement including:

- TABIX PLUS 1/60: an undulating fiber of 1 mm (0.04 in) dia wire, 60 mm (2.36 in) length, 8 mm (0.31 in) undulation length, 0.65 mm (25 mil) depth of undulation at axis, minimum tensile strength wire of 1,475 MPa (214 ksi).
- TWINCONE : a straight wire provided with anchoring by cone ends of 1 mm (0.04 in) diameter, 54 mm (2.16 in) length, 60° angle at summit, minimum tensile strength of 1,000 MPa (145 ksi).
- HE-1/60: a hooked end fiber of 1 mm (0.04 in) dia. wire with a min tensile strength of 1,475 MPa (214 ksi).
- Other deformed fibers are possible meeting 1 mm (0.04 in) diameter, 60 mm (2.36 in) long and made of high strength steel wire. Thinner fibers of lower strength or of a higher slenderness are not recommended since their performance might be lower and that the finishing shall not enable a fiber free surface.
- Fiber dosage rates of 40-100 kg/m³ (67-168 lbs/ft³) according to design procedure.

The fibers can be introduced into concrete at the batch plant, within the central mixer, truck mixer, conveyor belt, or feeders. Introduction and mixing of the fibers must be such that no balling takes place.
Casting and finishing

Once the concrete has been poured, the surface is levelled using a straight edge, followed by bull-floating in order to improve flatness and hide visible fibers. Neither poker vibration, nor screed vibration are necessary. When rebars are present in the slab, a minimum of 150 mm (6") clearance is needed to ensure proper compaction of concrete. The slab shall be cured as soon as possible after bull-floating. Figure 5(a) shows the slab lay out with the anti-progressive collapse reinforcement as the only continuous reinforcement system. Figure 5(b) shows the anchorage requirement for the walls and slabs. Note that the Anti-Progressive collapse reinforcement is the only continuous reinforcement system used in E-SFRSS.

Construction joints

Construction joints are typically located at ¼ of the span between piles and reinforced by a 5 mm (1/4 inch) thick steel nose to protect the areas under traffic as well as protection against hard wheel impact. They provide full shear transfer. Double Omega, Delta, or Diamond dowel types are used. The construction joints form bay sizes smaller than 2,500 m² (26,900 ft²) in general such that the length/width ratio is less than 1.5. Dock levellers areas are separated from the rest of the slab by a construction joint. At the re-entrant corners, a set of three 12 mm (#4) rebars are recommended on the top of slab at a 45° angle. Figure 6 shows the details of an elevated SFRC slab next to a large opening resulting in loss of two way action.
Long-term shrinkage and creep
The overall restraint against drying shrinkage movement, causes the slab to crack with a closely spaced pattern of minute cracks. The contraction restraint is generated by the contact with pile heads, the large bay size (2000-3000 m², or 21,500-32,300 ft²) and then, the length of the bay up to 60 m (197 ft) or even more in some cases. The random pattern of closely spaced cracks is quite helpful as it relieves the overall shrinkage stresses. At this stage, the slab is no longer elastic. The observed bulk shrinkage cracks are within 0.1-0.5 mm (3.93-19.7 mils) range of opening with a maximum value of 0.75 mm (29.7 mils). These crack widths are comparable to free and restrained shrinkage tests of steel fiber reinforced concrete. Under restrained conditions, a 0.5% or 40 kg/m³ (67.2 lbs/yd³) steel fiber concrete shows a reduction in the total crack opening by a factor of 4 with reference to a plain mix [33]. Under free shrinkage conditions, a 0.5% steel fiber reinforcing reduces the free contraction by 10% with reference to a plain mix as shown by Mangat and Azari [34].

Case study 1—steel fiber reinforced elevated suspended slabs (E-SFRSS)
A typical application of a E-SFRSS slab is presented in Figure 7(a) with a 5 m x 5 m grid (16.4 x 16.4 ft); and designed for a uniformly distributed loading of 5 kN/m² (100 psf). The design parameters include a thickness of 180 mm (7 in), and C40 concrete type. The slab is installed in sections of 800 m² (8800 ft²) bays between construction joints. Steel Fiber reinforcing of TABIX 1.3 mm dia x 50 mm length (0.052 in x 2 in), undulating shape, and 900 MPa (130 ksi) tensile strength at 100 kg/m³ (156 lbs/yd³) dosage rate was used.
Case study 2—Industrial floor on piles

A typical example of application of a 250 mm (10 in) thick suspended steel fiber reinforced concrete slab is presented in Figure 7(b). The slab is resting on a pile grid = 3.6m x 3.6m (12 ft x 12 ft) using a pile flared head diameter of 0.6 m (2 ft). The design is based on using a uniformly distributed loading of 50 KN/m² (1000 psf); bay size of 2500 m² (27,500 ft²); 45kg/m³ dosage rate of 1mm dia x 54 mm (0.040 in x 2.1 in) Twincone steel fibers. The slab is laid in 3000 m² (33000 ft²) joint free bay sizes of 50 m x 60 m (170 ft x 200 ft), using a 350kg/m³ (583 lbs/yd³) OPC Type I mix with w/c ratio below 0.50.

FULL SCALE TESTING OF ELEVATED SLABS

To evaluate the performance of SFRSS and E-SFRSS elements under different loading conditions, full scale experiments were conducted. The design methodology was based on full scale testing as outlined in TR6325, derived from the analysis of full scale flat round indeterminate slab tests. These test results were further evaluated by means of finite element analysis.20 Parameters addressed in the experimental program include the initial stiffness and cracking stress, multiple cracking mechanisms, and the failure patterns. Subsequent loading up to the ultimate point resulted in measurement of ductility and safety factors at service and ultimate loads. Table 1 shows a summary of the two full-scale testing procedures of E-SFRSS slabs (referred to as Bissen, and Tallinn).

The slabs were loaded by using hydraulic rams reacted against the column supports. The deflection and the strain measurements under the slabs were collected. The following observations were made during the testing of the center slab 4 (Tallin) as shown on Figure 7(a) and (b). The first crack occurred at P = 125 kN (28,100 lbs), under a deflection of δ = 3.55 mm (0.14 in) and a crack width of 0.05–0.1 mm (2 – 4 mils) within a center span of 5 m x 5 m (49.2 ft x 49.2 ft). As the load increased to 180 kN (40,465 lbs), the deflection increased to δ = 7 mm (276 mils), and the crack width to 0.15 mm (6 mils). Further observations on this full scale test include the following:

- Maximum loading intensity: P_max = 595 kN (133,700 lbs).
- Permanent deflection after unloading at 595 kN (133,700 lbs) was 22 mm (0.87 in).
- Maximum bottom crack opening under the point loading of 480 kN (107,900 lbs) was 3.5 mm (0.14 in).
- Unloading at P = 340 kN (76,435 lbs) resulted in a permanent deflection of 5.44 mm (0.21 in).
- Extremely dense minute fan pattern cracking on the tensile face of the slab.
- Circular multiple cracking passing by the 4 footprints of columns at the top of slab.
- Minute cracking at 595 kN (133,700 lbs) along the perimeter edges of the slab.

Figure 9 represents the comparison between two full-scale tests of the center span of Tallin and Bissen. The only difference in between the slabs is the column to column spacing. As the span length changes form 5 to 6 m (16.4 - 19.7 ft) in the Bissen case, the initial stiffness and cracking point do not change significantly, however due to the larger span, the moment on the Bissen test increases much faster, leading to failure at much lower loads. The plastic response and the significant ductility of both cases are notable. Full scale testing of the Bissen Central span indicate a First crack load of 230 kN (51.7 kips), First crack deflection of 7 mm (0.275 in), or L/857, and a maximum load of 470 kN (105.6 kips).

<table>
<thead>
<tr>
<th>#</th>
<th>Year</th>
<th>Location</th>
<th># x &amp; y Spans # of columns</th>
<th>Span length, mm (ft)</th>
<th>Thickness mm (in)</th>
<th>Column size mm x mm</th>
<th>Steel fiber dosage, kg/m³ (lbs/ft³)</th>
<th>Span/depth ratio</th>
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<td>Bissen, Luxembourg</td>
<td>3 / 3 / 16</td>
<td>6000 (19.7)</td>
<td>200 (7.9)</td>
<td>300 x 300</td>
<td>100 (168)</td>
<td>30</td>
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<tr>
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<td>2007</td>
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<td>5000 (16.4)</td>
<td>180 (7)</td>
<td>300 x 300</td>
<td>100 (168)</td>
<td>28</td>
</tr>
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</table>
CONCLUSIONS

Steps for the development of pile supported continuous slabs addressing factory floors, warehouse and basement floors, and elevated slabs of multistory buildings are addressed. Design methodologies consist of using moment capacity based on simplified material models or back-calculation of round panel tests. Typical mix designs, fiber loadings, finishing, and detailing aspects of these slabs are discussed. The design methodology is further confirmed by the full-scale structural testing of multi column slabs.
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Self-Consolidating High-Performance SFRC: An Example of Structural Application in Italy

by L. Ferrara, M. di Prisco, and N. Ozyurt

Synopsis: The addition of fibers into a self-consolidating concrete (SCC) matrix can take advantage of the superior fresh state performance to achieve homogeneous dispersion of the discontinuous wirelike reinforcement. Such a positive synergy between SCC and FRC technologies is of paramount importance to promote reliable structural applications. It has been furthermore shown that, through a well balanced set of fresh state properties of the mix, fibers can be effectively oriented along the direction of the fresh concrete flow. Superior mechanical performance of the material hence is obtained in the same direction. A “tailored” orientation of the fibers may be pursued to obtain a deflection-, or even a strain-hardening, behavior, which may be required by the specific application to be designed.

With reference to a project on going in Italy, this paper details the steps of a “holistic” approach to the design of Self Consolidating High Performance Fiber Reinforced Concrete (SCHPFRC) elements. In this framework both the mix composition and the casting process are designed to the anticipated performance of the structural element, in the sight of an optimized material and structural efficiency. This would allow to pursue, in the design process, a desirable closer correspondence between the shape of an element and the function it performs in a structure assembly. A suitably balanced fresh-state performance of the fiber reinforced cementitious composite would allow to “mold” the shape of an element and, thanks to a tailored casting process, to orient the fibers along the direction of the principal tensile stresses resulting from its structural function.

Keywords: deflection hardening behavior; fiber orientation; self-consolidating fiber-reinforced concrete; structural design
INTRODUCTION

The ability of randomly dispersed discontinuous fibers to enhance the fracture toughness of a brittle matrix, such as a cement based material, has been known for a long time, dating back to mud bricks reinforced with straw and horsehair mud bricks in use already in ancient Mediterranean civilizations.

Fiber reinforced concrete and cementitious composites actually represent the “up-to-date state of the art application” of the above mentioned concept. Along an almost half-a-century odyssey, initiated by the pioneer studies by Romualdi and Batson,1-2 the increase of knowledge about fiber reinforcing mechanisms, both in the field of theoretical modeling and experimental evidence, has led to continuous developments in new materials, processing techniques, standards and high-end products for building structures and other civil engineering applications.

Nowadays a wide variety of fiber reinforced cement-based composites is available, which are characterized by a range of engineering properties in terms of workability, mechanical strength and stiffness, fracture toughness and ductility, durability, fire and impact resistance, pricing and constructability, which may be selected depending on the specific application to be designed and built. As recognized by Naaman and Reinhardt3, among the most urgent issues to be tackled to promote increased structural applications of fiber reinforced cementitious composites, there is the need to specify different performance levels for their key engineering properties, together with the need to show how a particular fiber and fiber concrete mixture can lead to a prescribed level of performance.

As for the former issue, they have proposed to use as a key distinguishing material characteristic, whether the mechanical response of the composite to tensile loading is strain-hardening or strain-softening, and whether the structural behavior in bending is deflection-hardening or deflection-softening. “Conventional” deflection softening FRCs are well suitable for a wide range of applications, from the lower end, represented by the control of plastic shrinkage cracking,4,5 to the reduction/substitution of conventional reinforcement in concrete pavements and slabs on grade,6 to the higher end when fibers are used, e.g., to replace conventional shear reinforcement in precast prestressed elements.7 Deflection hardening composites may be useful, or even necessary, when structural elements mainly subjected to bending have to be designed. This may lead to optimized shape and size of the structural elements, with reduced self weights, also resulting into a more rational design of the supporting structure and consequently yields to a more time- and cost-effective construction process (including costs for transportation, erection, capacity of lifting equipments etc.). As further bonuses, improvements e.g. in durability related properties are likely to be achieved, such as imperviousness, diffusion resistance etc., from which the structural performance all along the anticipated life cycle may surely benefit.8

Within this cutting-edge framework, precast construction industry may be regarded as the most straightforward recipient of related technology transfer, able and willing to put forward on the construction market elements and components for high end structural applications.
Self-Consolidating High-Performance SFRC

With reference to the second need addressed above, i.e. to understand and show how a given fiber and fiber reinforced concrete mixture can lead to a prescribed level of performance, Naaman\(^9\) has demonstrated, by applying classical composite and fracture mechanics concepts, that a critical volume fraction of fibers is needed in order to obtain a strain-hardening behavior in tension or a deflection-hardening behavior in bending. Such critical volumes depend, obviously, on fiber dosage and aspect ratio (fiber factor \(V_f = \frac{l_f}{d_f}\)), fiber-matrix bond and matrix cracking strength, but also on the dispersion of the fibers and their orientation with respect to the direction of the applied tensile stress. The possibility of still keeping a deflection hardening behavior with a relatively low amount of fibers (e.g. around 1\% by volume), besides being attractive from the cost point of view, may also simplify the material and structural manufacturing processes, thus opening the way to more widespread and larger scale practical applications.

Very recent research\(^10\)\(^-\)\(^11\) has shown that the dispersion and orientation of fibers in concrete structural elements can be effectively governed through a suitably balanced set of fresh state properties and a carefully designed casting procedure. The key idea of the whole process is that the fluid mixture has to be first of all self compacting, in order to fill the formwork without any vibration or manual compaction. As a matter of fact, these operations may hinder homogeneous fiber dispersion and hence affect the final structural performance due to flaw effect of fiber-free or poorly reinforced zones. In details, the fresh state performance of the mixture must be characterized by an adequate viscosity, to drive the fibers along the direction of the flow and orient them according to the fluid velocity field lines. The value of the yield stress must be also carefully calibrated to control, within a reasonable range, the downward settlement of fibers due to their higher specific gravity with respect to the suspending fluid mortar. Furthermore, in order to transfer all these benefits to the final structural performance, the casting process must be carefully designed so to make the flow direction of fresh concrete, along which fibers may be aligned, to match as close as possible with the direction of the principal tensile stress within the structural element when in service.

In order to pave the way for spreading such an approach into the design and manufacturing practice of high-performance fiber-reinforced concrete (HPFRC) structural elements, the effect of fiber orientation, as governed through the fresh state performance and the casting process, has to be assessed through a suitable experimental procedure. This should form an integral part of the design approach in order to identify material parameters which are meant as relevant to it.

This paper will present an example of the above said “holistic” design approach applied to precast thin (25 mm–1 in.) roof slabs which have to be employed over a 2.5 m (8.3 ft.) span. Roof slabs are anticipated to be simply supported between precast prestressed roof elements and hence under prevalent bending action (Figure 1).\(^12\) The mix-design methodology of the self consolidating steel fiber reinforced cementitious composite (SCSFRC) will be first of all illustrated and the efficacy of the mix composition to guarantee random dispersion of fibers and orient them along the flow direction will also be checked. Through a “specimen casting” procedure conceived ad hoc, the dependence will be assessed of the mechanical behaviour in bending on the orientation of the fibers, as driven through the tailored casting process by the optimized fresh state properties. The mechanical performance of the tested full scale prototypes will be then analyzed.

The non-secondary aim of the study is also to show, with reference to a case study, how to exploit the correlation among fresh state performance, fiber dispersion and hardened state properties of SCSFRC to achieve a prescribed level of performance tailored to the specific structural application. This will be also prodromal to pave the way to further applications.
RESEARCH SIGNIFICANCE

In the framework of several research projects dealing with structural applications of SCSFRC currently ongoing in Italy, the focus in this paper will be on the manufacturing of precast slabs, 25 mm (1 in.) thick and with no conventional reinforcement. These slabs have to be employed as secondary elements in roof decks, laid out between precast prestressed single or double tees. A schematic layout is shown in Figure 1, which also compares the traditional with the high-end proposed solution, with self evident advantages in terms of reduced self weight, which turn into faster and more cost-effective construction. Improvements in the overall energetic performance of the building as well as with reference to the behavior under fire can also be addressed. Deflection hardening behavior of the FRC is crucial to guarantee the structural performance of these elements which are anticipated to work as simply supported beams over a 2.5 m span (8.3 ft.). Adaptation of rheology is believed to be the tool to obtain the required material and structural performance via a random dispersion and tailored orientation of fiber reinforcement. This also provides an example of exploitation, at an industrial scale, of the correlation between fresh and hardened state performance and fiber dispersion, in the framework of a “holistic” design approach of SCSFRC structural elements.

MIX-DESIGN AND FRESH STATE PERFORMANCE: FIBER DISPERSING AND ORIENTING ABILITY

The framework for mix-design optimization was provided by a methodology recently proposed: self consolidating fiber reinforced concrete in its fresh state is regarded as a suspension of solid particles (aggregates and fibers) in the fluid cement paste. The rheological properties of the paste (yield stress and viscosity) have to be balanced as a function of the paste and aggregate volume fractions as well as of the granular characteristics of the solid particle skeleton (e.g. the void ratio or the average particle size). Fibers are incorporated into the solid skeleton as a set of fictitious monosize spheres having the same surface area as the fibers. In the case of a denser suspension (lower paste ratio – higher volume fraction of solid particles) a less viscous and more deformable (lower yield stress/higher mini-cone flow diameter) paste is needed to obtain a good self consolidating concrete. On the other hand, for a looser suspension (higher paste ratio and comparatively lower volume fraction of solid particles) the same self compactability can be achieved through a paste which has to have a higher yield stress and a higher viscosity to prevent looseness of the fluid mixture and particle segregation.

Dealing with the proposed application, the anticipated thickness of the slabs (25 mm-1 in. nominal) actually represents a major constraint to be considered when selecting raw materials, with main reference to the fiber length and the maximum aggregate size. Short steel fibers, 13 mm (± 0.5 in.) long
and with a diameter equal to 0.16 mm (0.006 in.) were chosen, the higher aspect ratio (lf/df = 80) being a prerequisite for the desired high mechanical performance of the composite. Maximum aggregate size was consequently limited to 2 mm (0.08 in.); the grading curve of the adopted sand is summarized in Table 1. Due to the higher required cement content, the possibility of replacing it with slag was also evaluated; an optimum volume replacement ratio of 50% was found after a series of preliminary tests. The optimization of the actual mix-design composition was performed through a multiscale investigation (paste, mortar, fiber reinforced composite) aiming at adapting the fresh state performance to achieve the required mechanical properties in the hardened state.

Cement pastes were prepared in a planetary mixer (2 lt capacity), adopting the following mixing protocol: first raw materials were mixed for one minute; after water and superplasticizer (SP) were added in one minute and mixed for two minutes at low speed, left at rest for one minute and further mixed for three minutes at high speed.

The fresh state properties of the cement paste, as a function of its composition (w/b ratio, SP dosage) were first of all assessed through mini-cone and Marsh cone flow tests (results in Table 2). Mini-cone flow diameters was measured from the first test, whereas time employed for a given amount of fluid to flow through the nozzle orifice was garnered from the second (nozzle diameter φ = 11 mm - 0.41 in. was employed). This was deemed to provide enough information to robustly assess the fundamental rheological properties of the paste (i.e. yield stress and viscosity) even without rheometer tests.

After this investigation at the paste level, a set of mortar was prepared combining, in different volume ratios, pastes with different compositions and sand. The mixing protocol was the same adopted for cement paste; at the end of the high mixing step sand was added while continuously mixing at high speed along a two minutes interval and the mortar was finally mixed at high speed for three further minutes. This was deemed to provide a homogeneous mixing of the mortar. The fresh state performance was measured as detailed above for the paste. Results of the fresh state characterization of mortars are also summarized in Table 2. The role of paste and mortar mix-design variables (w/b ratio, SP dosage, paste volume) is coherent with well assessed knowledge on the topic; furthermore, saturation levels of the superplasticizer are evident when no further change in the mini-cone flow diameter and even increasing Marsh cone flow times (e.g. due to sedimentation of solid particles which may obstacle the free flow) are observed at increasing SP dosage.

Finally fiber reinforced mixes were prepared with selected mortar composition: fiber content was set, as a starting point at 100 kg/m³ (5.94 lb/ft³ - fiber volume fraction Vf = 1.27%), as a compromise between superior performance in the fresh and hardened state and cost effectiveness. The mixing protocol followed that detailed for mortar. At the end of the mortar mixing step fibers were added while continuously mixing at high speed along a three minutes interval and, after all fibers were added, the fiber reinforced composite was further mixed at high speed for five minutes: this was deemed to provide a good dispersion of the fibers in the mixing bowl. Details of fiber reinforced mix compositions are given in Table 2. The investigation then focused on the role of the rheology of the suspending fluid phase (either paste or mortar) on the flowability of the fiber reinforced mixture as well as on its ability to hold fibers in suspension and driving them along the flow, furthermore guaranteeing their randomly uniform dispersion as well as an orientation tailored to the specific foreseen application. In order to evaluate these properties, four sample methods were used, as illustrated in Figure 2. Besides the mini-cone slump test, the other ones are especially designed to reproduce as close as possible, at a smaller scale, the elementary flow conditions during the casting of precast slabs.

### Table 1 - Grading curve of the employed sand (crushed)

<table>
<thead>
<tr>
<th>sieve diameter – mm (in.)</th>
<th>% passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.0 (0.08)</td>
<td>3.7</td>
</tr>
<tr>
<td>1.0 (0.04)</td>
<td>16.9</td>
</tr>
<tr>
<td>0.50 (0.02)</td>
<td>43.9</td>
</tr>
<tr>
<td>0.25 (0.01)</td>
<td>70.5</td>
</tr>
<tr>
<td>0.125 (0.005)</td>
<td>86.8</td>
</tr>
<tr>
<td>0.075 (0.003)</td>
<td>92.9</td>
</tr>
<tr>
<td>0.063 (0.0025)</td>
<td>96.2</td>
</tr>
<tr>
<td>0</td>
<td>100</td>
</tr>
</tbody>
</table>
Table 2 - Fresh state characterization of cement pastes, mortars and fiber reinforced mortars

<table>
<thead>
<tr>
<th>Mix-ID</th>
<th>Paste composition</th>
<th>Mini cone flow diameter mm (in.)</th>
<th>Marsh cone flow time-sec (≥ 11 mm) (1lt over 1.5 lt)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>P1</td>
<td>0.18 2</td>
<td>460 (18.1)</td>
<td>6’18”</td>
</tr>
<tr>
<td>P2</td>
<td>0.18 2.5</td>
<td>470 (18.5)</td>
<td>6’05”</td>
</tr>
<tr>
<td>P3</td>
<td>0.18 3</td>
<td>510 (20.1)</td>
<td>4’18”</td>
</tr>
<tr>
<td>P4</td>
<td>0.20 2</td>
<td>515 (20.3)</td>
<td>4’06”</td>
</tr>
<tr>
<td>P5</td>
<td>0.20 2.5</td>
<td>520 (20.5)</td>
<td>4’17”</td>
</tr>
<tr>
<td>P6</td>
<td>0.22 2</td>
<td>535 (21.1)</td>
<td>3’38”</td>
</tr>
<tr>
<td>P7</td>
<td>0.22 2.5</td>
<td>550 (21.6)</td>
<td>3’20”</td>
</tr>
<tr>
<td>P8</td>
<td>0.22 3</td>
<td>560 (22.0)</td>
<td>3’22”</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mortar composition</th>
</tr>
</thead>
<tbody>
<tr>
<td>w/b</td>
</tr>
<tr>
<td>Vpaste</td>
</tr>
<tr>
<td>0.55</td>
</tr>
<tr>
<td>0.60</td>
</tr>
<tr>
<td>0.65</td>
</tr>
<tr>
<td>0.60</td>
</tr>
<tr>
<td>0.65</td>
</tr>
<tr>
<td>0.55</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fiber reinforced mortar composition</th>
</tr>
</thead>
<tbody>
<tr>
<td>V_fiber</td>
</tr>
<tr>
<td>---------</td>
</tr>
<tr>
<td>F1</td>
</tr>
<tr>
<td>F2</td>
</tr>
<tr>
<td>F3</td>
</tr>
<tr>
<td>F4</td>
</tr>
<tr>
<td>F5</td>
</tr>
<tr>
<td>F6</td>
</tr>
</tbody>
</table>

The effects of the rheology of the suspending paste and mortar on the resistance to static and dynamic segregation of fibers can be clearly appreciated from Table 3, which summarizes the results of tests detailed in Figure 2. It clearly appears that while all the mixes were characterized by a similar, quite good, fiber driving ability (resistance to dynamic segregation), resistance to static segregation dramatically varied. Due to the foreseen application, results from bending tests on thin slabs were regarded as more significant than those from the cylinder test and mix labeled as F3 was selected for the on site application, as featured by good resistance to dynamic segregation and acceptable resistance to static segregation of fibers for the intended application. Furthermore its higher easiness of mixing was regarded of the utmost importance when high quantities (1 m³ – 37 cubic feet per batch) had to be mixed in the precast plant. Figure 3 and Table 4 detail the results of the fresh state characterization performed on the site, where three batches were cast to manufacture specimens for characterization of hardened state behavior and structural elements prototypes. Some higher values of T50 and V-funnel flow time (TV) were measured, indicating a higher viscosity: this, even if may somewhat counteract speed of casting, is necessary to guarantee tailored orientation of fibers. All other tests were plainly successful: questions
about meaningful representativeness of standard tests conceived for plain SCC may be raised when HPFRCC with small maximum aggregate size and short fibers are employed. Further investigation is highly recommended also aimed at drafting “dedicated” standards.

![Figure 2 - Test methods used to assess dynamic and static segregation of fibers](image)

Table 3 - Results of tests for dynamic and static segregation of fibers

<table>
<thead>
<tr>
<th>Fiber reinforced mix</th>
<th>Mini-cone test fiber content kg/m³ (lb/ft³)</th>
<th>Channel flow test fiber content kg/m³ (lb/ft³)</th>
<th>Bending test MOR diff (%)</th>
<th>Cylinder test FDG</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
<td>b</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>F5</td>
<td>0.65</td>
<td>0.18</td>
<td>3</td>
<td>81.3</td>
</tr>
<tr>
<td>F3</td>
<td>0.60</td>
<td>0.19</td>
<td>2.5</td>
<td>95.0</td>
</tr>
<tr>
<td>F1</td>
<td>0.55</td>
<td>0.20</td>
<td>2</td>
<td>Difficult mixing and casting</td>
</tr>
<tr>
<td>F6</td>
<td>0.55</td>
<td>0.22</td>
<td>2</td>
<td>91.1</td>
</tr>
</tbody>
</table>
CASTING OF SPECIMENS AND TESTING FOR MECHANICAL PERFORMANCE

As a matter of fact, moving from the concept and production of a construction material towards realizing the intended structural application, a crucial step stands in defining a sound and reliable procedure for the identification of relevant material properties which have to be employed in design calculations.

![Image of fresh state characterization](image)

Figure 3 – Fresh state characterization of the SCHPFRC mix performed at the precast plant

Table 4 – Results of the fresh state characterization at the precast plant

<table>
<thead>
<tr>
<th>Batch</th>
<th>Slump flow</th>
<th>V-funnel</th>
<th>L-box</th>
<th>U-box</th>
<th>J-ring</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>diameter mm (in.)</td>
<td>T50 (sec)</td>
<td>flow time TV (sec)</td>
<td>height ratio</td>
<td>t200 (sec)</td>
</tr>
<tr>
<td>1</td>
<td>770 (30.3)</td>
<td>7</td>
<td>23</td>
<td>Not performed</td>
<td>735 (28.9)</td>
</tr>
<tr>
<td>2</td>
<td>730 (28.7)</td>
<td>7</td>
<td>=</td>
<td>Not performed</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>775 (30.5)</td>
<td>4</td>
<td>20</td>
<td>1</td>
<td>2.5</td>
</tr>
</tbody>
</table>

For structural applications of fiber reinforced cementitious composites, the identification of a “constitutive” stress crack-opening (or stress-strain) relationship is required, to be employed in the framework of suitable design approach and procedures. Currently available guidelines recommend prismatic notched specimens to be tested either under three or four point bending to obtain a (nominal) stress vs. crack (mouth or tip) opening displacement curve, from which the constitutive relationship has to be identified. In the case of HPFRCs, which due to multi-cracking are featured by a deflection hardening behavior, bending tests on unnotched prisms are recommended, whereas direct tensile tests on dog-bone or dumbbell specimens remain the most suitable for strain hardening materials.

The application of the above said concepts and methodologies to self consolidating fiber reinforced concrete, has to suitably take into account the peculiarities of this kind of material which is the achievement, among the other properties, of a random non-spotty dispersion and a tailored orientation of fibers. The fulfillment of this goal actually not only depends on the rheological properties of the material but also on the casting procedure as well as on the geometry of the structural elements. In order to effectively take all what above said into account, even at the level of material property identification, the specimen geometry and manufacturing procedure must resemble and mimic as close as possible that of the structural element to be. In this way it would be possible to obtain fiber dispersion and orientation features truly representative of those which will occur within the structure when in service, as also in relation with the pattern of stresses.

In the framework of this study the aim was to characterize the mechanical performance of the composite and identify the stress-crack opening relationships as a function of the orientation of the fibers. Slabs 1m x 0.5 m x 30 mm (40 x 20 x 1.2 in.) were cast, from which smaller beams, 150 mm (6 in.) wide and 500 mm (20 in.) long, had to be sawn and further tested in 4-point bending (4pb). In order to have quite a broad range of fiber orientations, two different casting procedures were adopted, as shown...
in Figure 4, either pouring the material at one short edge of the moulds, and allowing it to flow parallel to the long sides, or, alternatively, pouring the material centrally along one long edge and achieving formwork filling through an almost radial spread of the fresh concrete. Schematics in Figure 4 also show how beams for 4pb were sawn from the larger slabs, thus providing information about the combination of the flow driven fiber orientation (most likely parallel to the flow lines) and direction of the principal tensile stress applied during tests (in the direction of the beam axis), performed, in displacement control, according to the set up shown in Figure 5.

Results of 4pb tests are shown in Figure 6 in terms of nominal stress \( \sigma = \frac{P/2}{bh^2/6} \) with \( P \) total applied load, \( \ell \) loading span, \( b \) and \( h \) beam width and depth) vs. Crack Opening Displacement (COD). This was measured by means of LVDTs across the constant bending moment region over a 200 mm gauge length as shown in Figure 5. The results in Figure 6 clearly show to what extent the fibers, as oriented by the fresh concrete flow, were able to affect the mechanical performance of the material. Excellent repeatability between pairs of nominally identical tests can also be appreciated.

---

**Table 4 -- Results of the fresh state characterization at the precast plant**

<table>
<thead>
<tr>
<th>Slump flow diameter (in.)</th>
<th>N. = 1</th>
<th>T. (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200</td>
</tr>
<tr>
<td></td>
<td></td>
<td>400</td>
</tr>
</tbody>
</table>

---

**Figure 4 --** Schematic of slab-specimen casting and beam cutting (dashed specimens were further used for fiber orientation analysis) [measures in mm – 1 in. = 25.4 mm]

**Figure 5 --** test set up for 4pb tests
IDENTIFICATION OF TENSILE STRESS-CRACK OPENING RELATIONSHIP

The identification of the “constitutive” stress vs. crack opening behavior from bending tests has to be performed by taking into account the different length scales involved in the fracture process. In the pre-peak regime, where multiple cracking has been observed in the central part of the specimen subjected to constant bending moment, the length-scale governing the mechanical response can be assumed equal to the COD-measuring gauge-length ($l_{COD} = 200 \text{ mm} - 8 \text{ in.}$). On the other hand, once the localization into a single crack has occurred, the length scale governing the fracture process can be assumed equal to the specimen depth $h$.

This distinction obviously does not apply to the case of beams cut from slab A with the axis perpendicular to the flow, for which a conventional SFRC deflection-softening behavior was measured, with no multiple cracking. In this case the governing length scale can be assumed equal to $h$ all along the fracture process.

In the case of deflection hardening behavior it is proposed herein to consider:

- a pre-peak stress-strain law (Figure 7a), up to the peak strain, in which the first cracking strength $f_i$ is identified as the maximum nominal stress measured in the COD range 0-0.1 mm;
- a post-peak stress vs. crack opening law (Figure 7b); its relevant points are identified by inverse analysis through force and moment equilibrium equations, by assuming the stress distributions over the cross section also shown in Figure 7b. The crack opening ranges to which equivalent post-cracking strengths $f_{eq,2}$ and $f_{eq,0.1h}$ correspond are defined as follows:

$$\text{COD} = (0.02h \pm 20\% - \varepsilon_{peak} h) + \varepsilon_{peak} l_{COD} \quad [1a]$$

$$\text{COD} = (0.10h \pm 20\% - \varepsilon_{peak} h) + \varepsilon_{peak} l_{COD} \quad [1b]$$

---

Figure 6 - Nominal stress $\sigma_N$ vs. Crack Opening Displacement (COD) experimental curves
Figure 7 – Pre-peak stress-strain (a) and post-peak stress crack-opening (b) laws for deflection-hardening materials (coefficients $\beta_1$, $\kappa_1$ and $\kappa_2$ computed through inverse analysis)

For deflection softening behavior, as detected in beams T1/2 from slab A, the pre-peak stress-strain law is meaningless and only the post-peak survives; the crack opening ranges to which the post-cracking equivalent strength $f_{eq,2}$ and $f_{eq,0.1h}$ correspond are defined as:

- for $f_{eq,2}$: $COD = 0.02h \pm 20\%$ [2a]
- for $f_{eq,0.1h}$: $COD = 0.10h \pm 20\%$ [2b]

Table 5 summarizes the values of the above defined material parameters with reference to the set of performed identification tests.
Table 5 - First cracking and post-cracking equivalent strengths at different COD calculated from load vs. COD curves in Figure 6; peak stresses and corresponding CODs and strains

<table>
<thead>
<tr>
<th>Spec.</th>
<th>$f_{If}$ [wI = 0.1 mm] N/mm$^2$ (psi)</th>
<th>$f_{eq,2}$ [0.02h±20%] N/mm$^2$ (psi)</th>
<th>$f_{eq}$ [0.1h±20%] N/mm$^2$ (psi)</th>
<th>$\sigma_{N peak}$ [CODpeak] N/mm$^2$ (psi)</th>
<th>CODpeak (mm – in.)</th>
<th>$\varepsilon_{peak}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-L1</td>
<td>12.6 (1827) 23.5 (3408) 16.6 (2407) 26.0 (3770)</td>
<td>1.08 (0.042)</td>
<td>5.4e-3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-L2</td>
<td>15.4 (2233) 24.6 (3567) 12.2 (1769) 27.6 (4002)</td>
<td>1.36 (0.054)</td>
<td>6.8e-3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-T1</td>
<td>9.2 (1334) 8.9 (1290) 7.0 (1015) 9.2 (1334)</td>
<td>0.08 (0.003)</td>
<td>=</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A-T2</td>
<td>8.9 (1290) 8.8 (1276) 6.8 (986) 8.9 (1290)</td>
<td>0.10 (0.004)</td>
<td>=</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-L1</td>
<td>12.4 (1798) 23.5 (3408) 13.7 (1986) 24.1 (3494)</td>
<td>1.12 (0.044)</td>
<td>5.6e-3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-L2</td>
<td>12.7 (1842) 21.4 (3103) 15.2 (2204) 23.5 (3408)</td>
<td>1.07 (0.042)</td>
<td>5.4e-3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-T1</td>
<td>12.2 (1769) 19.2 (2784) 12.2 (1769) 20.3 (2944)</td>
<td>1.7 (0.067)</td>
<td>8.5e-3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-T2</td>
<td>11.7 (1697) 18.9 (2740) 13.5 (1958) 19.4 (2813)</td>
<td>1.04 (0.041)</td>
<td>5.2e-3</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 8 – Tensile stress-strain and stress vs. crack opening laws identified from 4pb tests
Figure 9 – identification 4pb tests - numerical vs. experimental comparison
Figure 10 – Influence of fiber orientation on stresses at different crack openings calculated from 4pb tests

Equations [1a-b] and [2a-b] hence results in equivalent openings of the localized crack for either deflection hardening and deflection softening materials. The proposed identification procedure, which results in the “constitutive” laws shown in Figure 8a-d (average values between pairs of nominal identical specimens), hence allows to consistently handle the same fiber reinforced cementitious composite exhibiting even strongly different mechanical behavior as a function of the orientation of fibers with respect to the applied tensile stress. It has to be furthermore remarked that what has been herein proposed can be regarded as an extension of the procedure currently recommended by Italian guidelines on the design of SFRC structures\textsuperscript{14} to the case of deflection hardening fiber reinforced cementitious composites for thin structural elements, which exhibit significant toughness up to quite high crack openings\textsuperscript{17-18}. The reliability of the proposed procedure can also be appreciated from back analysis fitting of experimental results, performed by means of a plane section approach (Figure 9 a-d), where the different length scales involved in the fracture process, as detailed above, have been consistently taken into account.

The correlation between values of relevant stresses characterizing the identified constitutive behavior and the fiber orientation is shown in Figure 10. Fiber orientation factors\textsuperscript{19-20} vertical to the fracture cross section have been evaluated for specimens L2 and T2 cut from both slabs by fibers counting, performed through the aid of an image analysis technique, detailed by Ozyurt et al. (2006)\textsuperscript{21}; details are also given in Table 6.

Table 6 - Details of fiber counting from micrograph analysis and fiber orientation factors on fracture cross section of selected specimens

<table>
<thead>
<tr>
<th>Spec.</th>
<th>cross section area cm(^2) (in(^2))</th>
<th>(n_{\text{fibers}})</th>
<th>fibres/cm(^2) (fibers/in(^2))</th>
<th>(\alpha = \frac{n_{\text{fibers}}}{V_f})</th>
</tr>
</thead>
<tbody>
<tr>
<td>A-L2</td>
<td>45.7 (7.3)</td>
<td>2463</td>
<td>53.9 (337.4)</td>
<td>0.850</td>
</tr>
<tr>
<td>A-T2</td>
<td>44.1 (7.0)</td>
<td>1236</td>
<td>28.0 (176.6)</td>
<td>0.442</td>
</tr>
<tr>
<td>B-L2</td>
<td>45 (7.1)</td>
<td>2159</td>
<td>48.0 (300)</td>
<td>0.757</td>
</tr>
<tr>
<td>B-T2</td>
<td>46.7 (7.5)</td>
<td>1593</td>
<td>34.1 (212.4)</td>
<td>0.538</td>
</tr>
</tbody>
</table>

**STRUCTURAL APPLICATION: FULL SCALE TESTS AND PAVING THE WAY TOWARDS “DESIGN WITH FIBER ORIENTATION”**

Paving the way for widespread application of SCSFRC to full load-bearing structures, some true scale prototypes of the designed roof slab were cast at a factory. Slabs were 1.2 m (4 ft) wide and 2.5 (8.3 ft.) long and their average thickness resulted equal to 26 mm (\(\equiv 1\) in.). It is worth here remarking once again that no conventional reinforcement was provided in these elements, which were designed to work as simply supported along their short edges spanning over their length. Furthermore no special care was taken in casting these elements, with fresh concrete poured at the center of each mould and allowed to freely fill them randomly spreading.

Full scale prototypes were subsequently tested as shown in Figure 11. Tests were performed by controlling the displacements of the actuator, which allowed to follow the structure behavior also in...
the post-peak branch up to 50% of the maximum load bearing capacity, also considering the quite high deflection values attained. Vertical deflections at mid-span, in correspondence of loading points as well as astride the supports were measured; crack opening at the beam intrados was also measured in the central constant bending moment region (800 mm - 31.5 in. wide), both over a 950 mm (3 ft 2 in.) gauge length as well as over a shorter gauge length equal to 100 mm (4 in.) either at midspan and at 500 mm (20 in.) from the center on both sides. In this way information about the local behavior was captured. In exact correspondence with COD transducers, longitudinal deformations at the beam extrados, e.g. on the compression side, were measured in order to evaluate moment-curvature behavior in the regions of interest. Further details can be found in di Prisco et al. (2008). \(^\text{12}\)

Figure 11 - Full scale SCHPFRC roof plate tested in 4 point bending

The fiber orientation dependant stress crack-opening relationships identified in the previous section can be employed to provide a deeper insight into the performance of structural elements, as measured in full scale tests, and, in case, to evaluate about the possibility of improving it by suitable casting process designed to obtain tailored orientation of fibers. The comparison between experimental results and numerical predictions, shown in Figure 12, provides interesting information to the above said purpose.

The experimental value of the moment at first cracking can be quite well predicted from the values of the first cracking nominal flexural strength (Table 5), as \(M_{cr} = f_{cr} \text{average} bh^2/6\), resulting in \(M_{cr} = 1.61 \text{ kNm}\) or \(M_{cr} = 1.75 \text{ kNm}\), respectively if the worst results of beams a T1/2 are included or not in the calculation of the average value of the first cracking flexural strength \(f_{cr, \text{average}}\). This clearly shows, as exspectable, limited influence of fiber orientation on the first cracking moment.

On the other hand, the maximum load bearing capacity (or moment capacity of the cross section) is significantly affected by the orientation of the fibers, as shown in Figures 12-13a. The benefits which would come from a casting tailored to obtain a most favorable fiber orientation are evident also in terms of post-cracking strength resources (ratio between the maximum bending moment \(M_u\) and the cracking moment \(M_{cr}\) – Figure 13b) and ductility (ratio between the ultimate and the first-cracking curvature \(\chi_u/\chi_{cr}\) - Figure 13c). Interestingly, the actual orientation obtained in the true scale prototypes can be estimated from prediction fitting. The reliability of the procedure herein employed paves the way to address a design approach which explicitly takes into account the effects of fiber orientation on the material and structural performance\(^\text{22}\) as well as towards the study of further optimized geometries which may take profit at the best of the enhanced performance of the material.
CONCLUDING REMARKS

This work has presented in detail the results of a pilot study dealing with structural applications of Steel Fiber Reinforced Self Consolidating Concrete, in which the main focus was on the possibility of governing the orientation of steel fibres within a structural element through a well balanced fresh state performance, as obtainable by virtue of an appropriate mix composition, and a suitably designed casting process. The orientation of the fibres was shown to significantly affect, as expectable, the mechanical performance of the fibre reinforced cementitious composites, as also witnessed by the remarkable correlation between fracture toughness parameters and fibre orientation factors. This is a discriminating factor to obtain a high mechanical performance of the material, such as for example deflection hardening or even reliable strain hardening behaviour, besides suitable selection of fibre type, fibre volume fraction, fibre-matrix bond and matrix characteristics. The influence on the load bearing capacity of the structural element has also been addressed.
Figure 13 – Effect of fiber orientation on cracking moment $M_{cr}$ and ultimate moment $M_u$ (a), on post-cracking strength resources $M_u/M_{cr}$ (b) and on ductility $\chi/\chi_{cr}$ (c): numerical predictions from identification tests and experimental results from true scale tests.
The procedure for the identification of the stress-strain and/or stress-crack opening laws to be used in design should be coherent for both deflection softening and deflection hardening behaviour, which may “pertain” to the same material as a function of fibre orientation. In this paper the procedure recommended by Italian guidelines for conventional SFRCs has been adapted to deflection hardening behaviour through suitable “offset” with respect to the peak of the relevant crack opening ranges for the calculation of post-cracking equivalent strengths.

All what above said converges towards the definition of an integrated design approach, which can be achieved thanks to the synergy of the SCC and SFRC technologies, which tailors both the material composition and the casting process to the anticipated structural performance. This aims to achieve a tailored orientation of fibres, matching as close as possible with the direction of the principal tensile stress within the structural element when in service, and which may yield to a more efficient structural use of the material. In the design process a desirable closer correspondence between the shape of an element and the function it performs in a structure assembly could hence be pursued. A suitably balanced fresh-state performance of the fiber reinforced cementitious composite would allow to mould the shape of an element and, thanks to a tailored casting process, to orient the fibers along the direction of the principal tensile stresses resulting from its structural function.

**REFERENCES**

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