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Shear Response of SFRC Beams Constructed with SCC and Steel Fibers

Hassan Aoude* & Michael Cohen

Department of Civil Engineering, University of Ottawa, Ottawa, Canada

*haoude@uottawa.ca

ABSTRACT: This paper presents the experimental results from eleven slender beams constructed with highly workable SFRC tested under four-point loading. In the study self-consolidating concrete (SCC) was combined with steel fibers to improve workability and concrete placement. The response of the beams in terms of shear versus deflection response, crack control and damage tolerance is reported. The results demonstrate that the combined use of SCC and steel fibers in shear-deficient beams results in significant improvements in shear resistance and enhancements in flexural ductility. In the second part of the paper a model is proposed for predicting the shear capacity of SFRC beams. The proposed model and various equations proposed in the literature are used to predict the shear capacity of the beams tested in the experimental program. The results demonstrate the need for developing accurate and reliable equations for predicting the shear capacity of SFRC beams constructed with SCC and steel fibers.

1 INTRODUCTION

To prevent shear failure beams are traditionally reinforced with transverse shear reinforcement. One alternative to this reinforcement lies is in the use of steel fiber reinforced concrete (SFRC). The addition of fibers improves the diagonal tension capacity of concrete resulting in an increase in shear resistance which can promote flexural failure and ductility. The structural use of SFRC typically requires the addition of high fiber contents which can result in reduced workability and difficulty in concrete placement. The combined use of self-consolidating concrete (SCC) and steel fibers has been proposed as a solution to this problem and can result in highly workable SFRC that can allow for the higher fiber contents required for structural applications.

This paper reports the results of an experimental study that investigates the performance enhancements that can be gained from the use of highly workable SFRC in beams tested under four-point loading. The response of the beams in terms of shear versus deflection response, crack control and damage tolerance is reported. In the second part of the paper a model for predicting the shear resistance of SFRC beams is proposed. Equations proposed in the literature and by the authors are then used to predict the shear capacity of the SFRC beams tested in the experimental program.

2 LITERATURE REVIEW

2.1 Research on shear behavior of SCC beams

There is limited published data in the literature on the shear behavior of plain SCC beams. Lachemi et al. (2005) tested a series of 18 normal concrete (NC) and SCC beams to examine the influence of aggregate size and SCC on the shear behavior of beams. The authors found that a decrease in aggregate size from 19 mm to 12 mm reduces the ultimate shear capacity of reinforced concrete beams. This reduction in capacity was linked to reduced aggregate interlock when smaller aggregates are used. Similarly a comparison between the behavior of the NC and SCC series showed the beams constructed with SCC had reduced shear capacity when compared to the companion NC beams. This reduction in capacity was linked to reductions in aggregate interlock due to the lesser coarse aggregate content that is typical of SCC. In another recent study, Lin and Chen (2012) tested 24 beams, which included 8 NC beams and 16 beams constructed with two types of SCC. The "Type I" SCC beams contained a greater amount of coarse aggregate, while the "Type II" SCC beams had lesser aggregate content. The variables included shear-span-to-depth ratio (a/d), spacing and strength of transverse reinforcement. The results demonstrated that the Type I SCC beams had higher shear capacity when compared to the companion NC beams, however in the case of the SCC beams with reduced aggregate content (Type II), structural performance was inferior with reduced cracking and ultimate strengths. The results from these two studies demonstrate the need for further research on the shear behavior of SCC beams and the development of SCC-specific design equations that can modify existing "traditional concrete" shear-strength requirements in codes.

2.2 Research on shear behavior of SFRC beams

Research over the past three decades has shown that the provision of steel fibers can be used to enhance shear resistance in reinforced concrete beams. Parra-Montesinos (2006) presented a large database of SFRC beam test results which included 147 beams from 16 studies. Analysis showed that all beams in the database having fiber content, $V_f \geq 0.5\%$ exhibited shear stress at failure greater than $0.17\sqrt{f'_c}$ [MPa], equivalent to V_c as defined in the ACI-318 code (ACI, 2011), with beams having $V_f \geq 0.75\%$ failing at shear stresses not less than $0.3\sqrt{f'_c}$ [MPa]. Based on the analysis of the database, it was recommended that if added in sufficient quantity, steel fibers can be used as a replacement to minimum shear reinforcement in beams subjected to shear forces equivalent to 0.5 V_c and V_c.

2.3 Research on shear behavior of SCFRC beams

The addition of steel fibers in traditional concrete mixes can cause problems in workability and placement, particularly when high fiber contents are used. The use of self-consolidating concrete (SCC) has been proposed to reduce these problems and facilitate placement (Khayat and Roussell, 2000). Studies have shown that by utilizing a highly flowable SCC mix, self-consolidating properties can be maintained at low or moderate fiber contents, while adequate workability can be maintained at higher fiber contents (with some loss of self-consolidating properties). Liao et al. (2006) presented an excellent review of self-consolidating fiber reinforced concrete (SCFRC) mix designs studied in the literature.

While there has been several studies focusing on the development of SCFRC mix designs, there is limited data in the literature related to the structural use of SCFRC in beams. Greenough and Nehdi (2008) performed a study on 13 slender SCFRC beams having a/d ratios equal to or greater than 3 and having fiber contents ranging from 0.5-1.0%. The authors noted that SCC is particularly well suited for fiber addition due to its improved rheological properties. The authors found that the addition of steel fibers improves the shear behavior of SCC slender beams, with results showing that addition of 1.0% fiber content results in a 128% increase in shear capacity. You et al. (2010) carried out another investigation on two series of SCC beams reinforced with steel fibers. The first series included 8 beams with an a/d ratio of 3.0. Two of the beams were constructed without stirrups while 6 of the beams had transverse reinforcement ratios ranging from 0.13%-0.22%. The beams in this second series had an a/d ratio of 3.2, and transverse reinforcement ratio ranging from 0.35-0.53%.The results showed that the addition of steel fibers increases the shear capacity of SCC beams and that a sufficient quantity of fibers can alter the failure mode from a brittle shear collapse into a ductile flexural failure. The authors of this study also demonstrated that combining steel fibers with traditional shear reinforcement in the form of stirrups can produce a positive hybrid effect and allow for a partial reduction of shear reinforcement without negatively affecting performance. It is noted that only a moderate fiber content of 0-50 kg/m^3 was used in this study, with the authors noting that 50 kg/m^3 may be an upper limit for a fully self-consolidating fiber-reinforced SCC.

3 RESEARCH NEED

While there is important research in the literature on the behavior of SFRC beams, there is limited research data on the response of beams constructed with SCC and steel fibers. Given the recent adoption of provisions related to use of SFRC in beams in the ACI 318 code and increased interest in the structural use of SCFRC and highly-workable SFRC, it is of importance to enrich the experimental database related to the shear behavior of beams constructed with these materials. There is also a need to assess the ability of existing models proposed for SFRC to predict the shear resistance of beams constructed with SCC and steel fibers.

4 EXPERIMENTAL INVESTIGATION

4.1 Description of test specimens

A total of eleven slender SFRC beams constructed with SCC and steel fibers were tested under four-point loading. **Table 1** summarizes the properties of the beams tested in the experimental program. The beams had cross-sectional dimensions of 125 mm x 250 mm, with all specimens having a clear cover of 30 mm, with two ratios of longitudinal reinforcement. Two 15M bars (d_b=16 mm and A_s=200 mm^2) were placed at the bottom of the seven beams in the M15-series, while two 20M bars (d_b=19.5 mm and A_s=300 mm^2) were used in the four beams in the M20-series. No transverse reinforcement was provided in any of the specimens. All the specimens were simply supported over a span of 2400 mm and were subjected to four-point loading with a shear span of 800 mm from the face of the support (shear span-to-depth ratio *a/d* was approximately 3.8), and a constant moment region of 800 mm. The beams rested on roller and pin supports while a load transfer beam was used to apply the two point loads.

Table 1. Summary of specimen details

Beams	Concrete Type	Longitudinal Reinforcement	Fiber Type	Fiber content
M15-0.0%	KING SCC	2 - 15M (ρ=1.55%)	ZP-305	0.00%
M15-0.5%			ZP-305	0.50%
M15-1.0%			ZP-305	1.00%
M15-1.5%			ZP-305	1.50%
M15-0.5%H			BP80/30	0.5%
M15-0.75%H			BP80/30	0.75%
M15-1.5%5D			5D	1.50%
M 20-0.75%	KING SCC	2 - 20M (ρ =2.33%)	ZP-305	0.75%
M 20-1.0%			ZP-305	1.00%
M 20-1.0%A	Custom SCC		ZP-305	1.00%
M 20-1.5%A			ZP-305	1.50%

Figure 1 shows a typical beam prior to testing. The specimens in the M15 series were cast using self-consolidating concrete (KING-SCC mix) having different fiber contents. Beam *M15-0.0%* contained no fibers, while Beams *M15-0.5%*, *M15-1.0%* and *M15-1.5%* were constructed using SCC containing Dramix ZP-305 fibers in a quantity of 0.5%, 1.0% and 1.5% by volume of concrete, respectively. Beams *M15-0.5%H* and *M15-0.75%H*, were built using the same SCC mix but with 0.5% and 0.75% of BP 80/30 high-strength steel fibers, respectively. Beam *M15-1.5%-5D* was built using 1.5% of high-strength 5D fibers (see next section for fiber properties). The final two specimens, *M20-1.0%A* and *M20-1.5%A*, were constructed in a similar fashion, but a different SCC mixture (Custom-SCC) was used, combined with 1.0% and 1.5% of Dramix ZP-305 fibers, respectively.

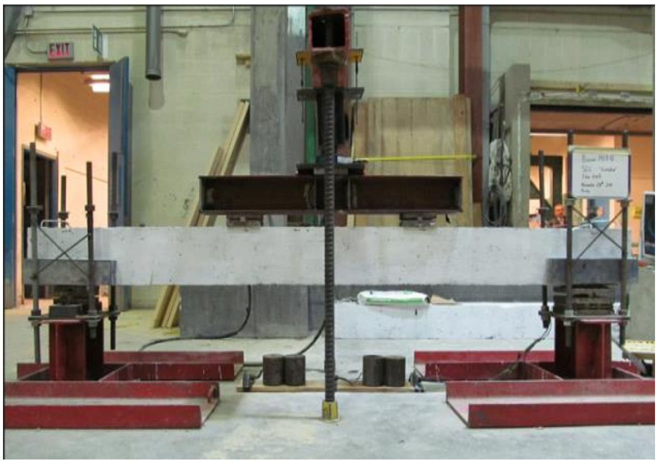


Figure 1. Typical beam specimen prior to testing

4.2 Materials Properties

In this test program the combined use of SCC and fibers resulted in highly workable SFRC. Two base SCC mixtures were considered. The KING-SCC mix consisted of a pre-packaged, self-consolidating concrete mixture with a strength of 50 MPa (MS Self-Consolidating Concrete, KING Packaged Materials). The mix contained a maximum aggregate size of 10 mm with a sand-to-aggregate ratio of approximately 0.55 and a water-cement ratio of approximately 0.42. An air-entraining admixture, a superplasticizer and a viscosity modifying admixture (VMA) are incorporated into the mix in the form of dry powder. **Table 2** lists the composition of this concrete as specified by the manufacturer. This mix was utilized in nine of the specimens used in the experimental program. The first "control" batch contained no fibers, while the fiber contents of the remaining batches ranged from 0.0% to 1.5% and included three fiber types (Dramix ZP305, Dramix BP80/30 and Dramix 5D fibers). The last two beams in this experimental program were constructed using a customized SCC mixture (Custom-SCC). The objective was to produce a regular-strength SFRC mixture with high workability at high fiber contents (1.0% and above). This was accomplished by limiting coarse aggregate size, using cement with Fly Ash and the use of chemical admixtures. The mixture contained Type 10 cement and class C fly ash. Coarse aggregate size was limited to 12 mm while silica sand was used as fine aggregate. The mix incorporated hooked-end steel fibers at fiber contents of 1.0% to 1.5%. A superplastizer (ADVA CAST 575) and viscosity modifying admixture (V-MAR 3) were used in order to improve workability and prevent segregation. **Table 2** summarizes the final mix proportions which were determined using a series of trial batches and based on a review of mixtures proposed in the literature (Liao et al. 2006).

Table 2. Mix design properties

Component	KING-SCC	Custom-SCC
Cement	500 (kg/m^3)	473 kg/m^3)
Fly Ash	---	238 (kg/m^3)
WC ratio	0.42	0.33
Coarse Agg.	765 (kg/m^3)	439 (kg/m^3)
Fine Agg.	915 (kg/m^3)	806 (kg/m^3)
Mass Density	2300 (kg/m^3)	2400 (kg/m^3)
Superplastizer	In form of dry powder	292 (ml/100 kg)*
VMA		52 (ml/100 kg)**
Steel Fibers	0.5-1.5 (%)	1.0-1.5 (%)

* ADVA CAST 575, in ml/100 kg of cementitious materials
** V-MAR 3, in ml/100 kg of cementitious materials

Three types of steel fibers were used in this experimental program (see **Figure 2**). All fibers are manufactured by Bekaert. The ZP305 hooked-end fibers have 30 mm length, aspect-ratio (L_f/d_f) of 55 and are made of normal strength steel wire with a tensile strength of 1100 MPa. The hooked-end BP80/30 fibers have 30 mm length, aspect-ratio of 80 and a higher tensile strength of approximately 2300 MPa. Finally, the 5D fibers have length of 60 mm, aspect-ratio of 65, tensile strength of 2300 MPa and have a more optimized hook shape (two bends in the end hooks to increase pullout strength).

The 15M bars in the first series of beams had a yield strength (f_y) of 441 MPa, while the 20M bars which were utilized to reinforce the remaining beams had a yield strength of 474 MPa.

The workability of the concrete mixtures was measured by performing slump flow tests on each batch of SCC and SFRC. **Table 3** summarizes the average results from the slump flow test. As noted previously the goal of combining SCC and steel fibers in this study was to produce highly workable SFRC. As such, self-consolidating properties were expected to be lost at the higher fiber contents. Nonetheless, all mixtures remained sufficiently workable. During casting of the specimens constructed with the KING-SCC mix and normal-strength fibers, no vibration was required at a fiber content of 0.5% and 1.0%, while slight vibration was required when casting the specimens with 1.5% fibers. The use of 0.75% of high-strength fibers significantly reduced workability owing to the higher aspect ratio of this fiber, and required significant vibration during casting. A sufficiently workable mix was obtained when using 1.5% of the 5D fibers and only slight vibration was required during casting, although some slight segregation was observed during visual inspection after the slump flow test. In terms of the custom-SCC mix, the SFRC showed highly workability allowing for easy concrete placement during construction of the beams.

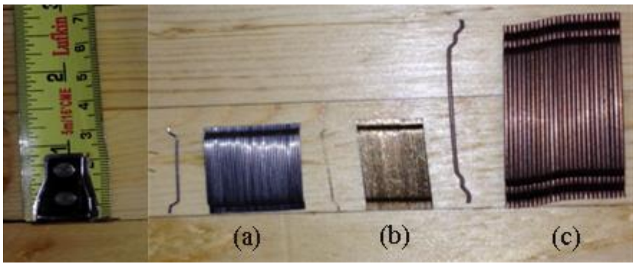
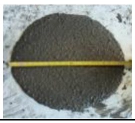
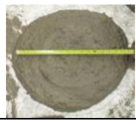


Figure 2. Hooked-end steel fibers considered in this study: (a) ZP305; (b) BP80/30; and (c) 5D fibers.

A series of lab cured cylinders and flexural beams were prepared during the mixing of the concrete (three cylinders and one flexural beam for each batch). The specimens were cured with burlap and covered with plastic sheets for seven days, and were then demolded and air-cured in the laboratory until testing. The control samples were tested on the day of the corresponding beam tests (the age of the specimens on the day of testing varied between 95 and 97 days, with the exception of the beam with the 5D fibers which had an age of 45 days at the day of testing). Compressive strengths were obtained by testing standard cylinders having a diameter of 100 mm and a height of 200 mm (average strengths at the day of testing, f_c are summarized in **Table 3**). In addition, bending tests performed according to the ASTM C1609 Standard (ASTM, 2007) were used to examine toughness. The tests were conducted on samples that were 100 mm by 100 mm by 400 mm in size which were loaded over a span of 300 mm . Average modulus of rupture values, f_r are summarized in **Table 3** and toughness parameters are reported in **Table 4** for the various SCC and SFRC mixtures.

Table 3. Average concrete properties: compressive strength, modulus of rupture, slump flow, and sample slump flow photos

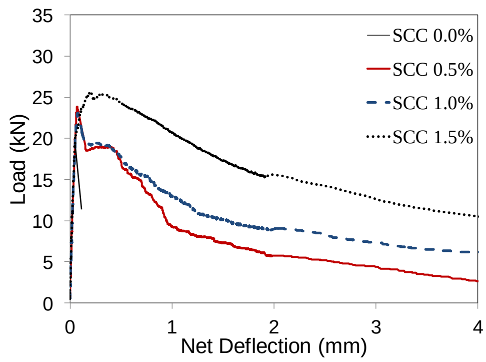
Mix	SCC-0.0%	SCC-0.5%	SCC-0.75%	SCC-1.0%	SCC-1.5%
f_c , MPa	51.9	59.4	49.7	51.5	55.8
f_r , MPa	6.0	7.0	--*	6.7	7.7
Slump, mm	600	530	520	485	440
Sample Slump Photo					
Mix	SCC-0.5H%	SCC-0.75H%	SCC-1.0%A	SCC-1.5%A	SCC-1.5%5D
f_c , MPa	49.6	46.0	54.5	50.5	52.8
f_r , MPa	6.5	5.8	7.3	8.5	--*
Slump, mm	440	410	440	400	450
Sample Slump Photo					

* No flexural beams were tested for the mixtures with 0.75% ZP305 fibers and 1.5% 5D fibers

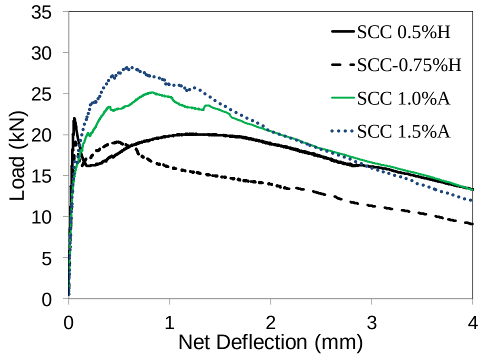
Table 4. Results from flexural toughness tests and comparison to ACI performance requirements

Mixture Name	ASTM C1609									Meets ACI requirements ?		
	$f_{r,ACI}$	f_1	f_p	f_{300}	f_{600}	f_{150}	90% f_1	75% f_1	T_{150}	$v_f \geq 0.75\%$	$f_{300} \geq 90\% f_1$	$f_{150} \geq 75\% f_1$
M15-0.0%	5.04	5.93	5.93	-	-	-	5.34	4.45	-	×	×	×
M15-0.5%	5.39	7.02	7.02	2.91	5.06	1.68	6.32	5.26	23.40	×	×	×
M15-1.0%	5.02	6.72	6.72	3.95	5.26	2.59	6.05	5.04	27.23	×	×	×
M15-1.5%	5.23	6.11	7.70	6.38	0.15	4.67	5.49	4.58	40.44	✓	✓	✓
M15-0.5%H	4.93	6.46	6.46	6.05	5.24	5.57	5.80	4.85	37.68	×	✓	✓
M15-0.75%H	4.75	5.58	5.72	5.00	5.72	4.20	5.02	4.19	32.18	✓	✓	✓
M 20-1.0%A	4.97	5.77	7.25	7.54	6.66	5.92	5.19	4.33	44.47	✓	✓	✓
M 20-1.5%A	5.17	4.91	8.45	8.00	8.21	6.14	4.41	3.88	48.43	✓	✓	✓

Load-deflection responses are presented in **Figure 3** (it is noted that no flexural beam data is available for the specimens with 0.75% ZP305 fibers and 1.5% 5D fibers). As expected increasing fiber content results in improved post-cracking resistance and toughness. For the KING-SCC mix, examination of the results in **Table 4** shows that the T_{150} toughness parameter, which corresponds to the area under the load-deflection curves from 0 to L/150, increased by 49% for the mix with 1.5% ZP305 fibers when comparing to the mix with 1% ZP305 fibers. Similarly, the use of 0.5% high-strength BP80/30 fibers resulted in a 61% increase in toughness when comparing to the mix with 0.5% normal-strength ZP305 fibers. When comparing to the KING-SCC mix, the use of 1% ZP305 fibers in the Custom-SCC mix showed a 63% increase in toughness. The ACI-318 code recently introduced provisions permitting the use of SFRC in flexural members (ACI, 2011). In order to qualify as an alternative to minimum shear reinforcement in beams material, dimensional and loading limits must be met. At the material level, the SFRC must have a minimum fiber content of 60 kg/m^3 and meet minimum performance requirements. The residual strength obtained from the ASTM C1609 test at midspan deflections of L/300 and L/150 must be greater or equal to 90% and 75% of the first-peak strength, respectively (where the first-peak strength, f_p, is obtained from a flexural test but not lesser than the value specified in Eq. 9-10 of the ACI 318 code).



(a) 0%, 0.5%, 1%, 1.5%;



(b) 0.5%H, 0.75%H, 1%A, 1.5%A

Figure 3. Sample flexural beam load-deflection responses.

Examining the residual strength values in **Table 4** it is seen that for the KING-SCC mix, the performance requirements specified by the code are met for the mixtures containing 1.5% normal-strength fibers and 0.75% high-strength fibers, whereas the requirements are not met for the mixtures with 0.5% or 1% fibers. For the Custom-SCC mix, the ACI requirements are satisfied at fiber contents of 1.0% & 1.5%.

5 RESULTS AND DISCUSSION

5.1 M15 Series-Effect of fiber content

To recall, the seven beams in the M15-KING series had identical cross-sectional properties and were cast using a traditional SCC mix. The series included one control beam (Beam *M15-0.0%)* and six beams having varying fiber contents (ranging from 0.5% to 1.5%) and fiber types. **Table 5** and **Figure 4** compare the shear and deflections values (at maximum and at failure) and the load-deflection responses of the various beams tested in this series.

In terms of shear resistance, as has been reported for traditional SFRC beams, one can see that the addition of steel fibers in SFRC beams constructed with SCC and steel fibers leads to improvements in shear capacity. In addition, as the fiber content is increased so does the relative increase in shear strength. For example, as shown in **Table 5** and **Figure 4(a)**, the addition of 0.5% fibers in *Beam M15-0.5%* increased the shear capacity of the beam by 47% when comparing to the strength of the control beam. A further increase in fiber content to 1.0% has led to 63% improvement in shear strength when compared to the control beam. The use of 1.5% fibers did not lead to a further increase in capacity when compared to the companion beam with 1.0% fibers. It is noted that these last two beams failed in flexure and the result shows that addition of steel fibers does not have a significant influence in increasing moment capacity in beams failing in flexure.

5.2 M15 Series-Effect of fiber type

In addition to the above specimens, two of the beams in the M15-series were constructed with high-strength BP80/30 steel fibers. A comparison of the results from **Table 5** and **Figure 4(b)** shows the beneficial impact of using high-strength fibers. Besides the improvement in shear capacity, the use of high-strength fibers has contributed to an enhanced post-peak response and ductility. Just as in the case of the normal strength fibers, the use of high-strength fibers was shown to improve ultimate shear capacity and member response. When compared to the control beam, increases in strength of 54% and 63% for Beams *M15-0.5%H* and *M15-0.75%H* were observed. The response of Beam *M15-0.5%H* shows that the use of high-strength fibers allowed the beam to sustain significant deflections before failure when compared to the control specimen. Similarly, although both Beams *M15-0.5%* and *M15-0.5%H* failed in shear at similar failure loads, a comparison between the deflections at failure (see **Table 5**) and the member responses (see **Figure 4(b)**) demonstrates an improved level of ductility for the beam reinforced with high-strength fibers (with Beam *M15-0.5%H* experiencing 200% increase in deflections before failure). The results also show that an increase in high-strength fiber content to 0.75% resulted in a further increase in strength and was sufficient to produce a ductile flexural failure in Beam *M15-0.75%H*. As noted previously, the use of high-strength fibers resulted in reduced workability and some reduction in compressive strength (possibly due to increased entrapped air). Nonetheless, it is noted that the structural response of Beam *M15-0.75%H* is very similar to the response of Beam *M15-1.0%* in terms of flexural ductility. The results from specimens *M15-0.5%H* and *M15-0.75%H* indicate the potential benefits from the use of higher strength steel fibers in beams, however further research is recommended. Owing to the higher aspect-ratio of the BP80/30, careful SCC mix proportioning is recommended in order to ensure adequate workability and reliable structural performance. **Figure 4(c)** shows the response of Beams *M15-1.5% and M15-1.5%5D*. Both beams failed in flexure, however the use of the 5D fibers, which have optimized strength and anchorage properties, resulted in an increase in beam flexural capacity.

5.3 M20 Series-Effect of fiber content

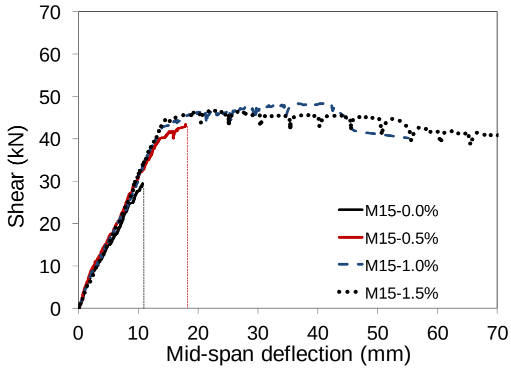
Beams *M20-0.75%* and *M20-1.0%* were constructed with a traditional SCC mix, normal strength hooked-end steel fibers and fiber contents of 0.75% and 1.0% by volume of concrete, respectively. It is noted that no control beam was tested in the M20-series. To allow for a comparison of shear capacities, the nominal shear strength of a reinforced concrete beam having identical properties but without fibers was predicted using the general shear design method of the CSA A23.3-04 Standard (CSA, 2004); assuming compressive strength, f_c of 50 MPa and yield strength, f_y of 474 MPa, a nominal capacity of 34 kN was predicated. **Table 5** compares the maximum shear capacities of the two SFRC beams with the calculated nominal capacity and indicates a relative increase in resistance due to addition of steel fibers. The results also show that increasing the fiber content from 0.75% to 1.0% is accompanied by a relative increase in shear resistance. As shown in **Figure 5**, despite the enhancement in shear capacity, the failure pattern for both beams in this series was brittle and the beams failed in shear with a sudden loss in load-carrying capacity at failure.

5.4 M20 Series-Effect of longitudinal reinforcement

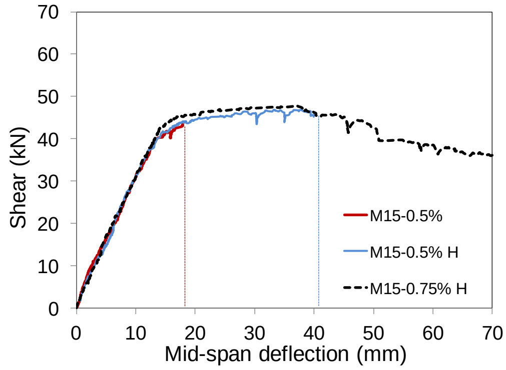
The beams discussed in the previous section were reinforced with two 20M bars, whereas all other properties including concrete type remained unchanged from the M15 test series. A comparison of the responses of Beams *M15-1.0%* and *M20-1.0%* in **Figures 4(a)** & **5** show that while 1.0% fibers was sufficient to transform the brittle shear response in the M15 series beam, the use of 1.0% fibers in the beam with 20M reinforcement was not sufficient to avoid a brittle shear failure. The reason returns to the higher flexural capacity in the beams with larger reinforcement ratio. That is to say that the additional shear capacity provided by 1.0% steel fibers was not sufficient to “make up” the shear required to reach the beam’s flexural capacity in the beam with larger reinforcement ratio. The result puts into evidence the importance of developing accurate equations for predicting the shear resistance of SFRC beams.

5.5 M20 Series-Effect of concrete type

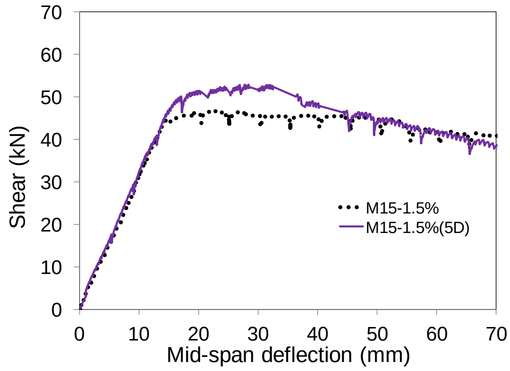
In addition to the two beams constructed with traditional SCC, the M20 series also included two other beams constructed with a customized concrete mixture (SCC-Custom Mix). In comparing the results in **Table 5** Beams *M20-1.0%A* and *M20-1.5%A* have shown a 74% and 82% higher shear capacity respectively when compared to the expected nominal shear strength capacity for a beam without fibers. A comparison of the responses of Beam M20-1.0% from the M20-KING series and Beam M20-1.0%A allows for an investigation into the effect of concrete type on response. Both beams were reinforced with 1.0% normal-strength hooked-end fibers and had identical properties with the exception of concrete mix type. In comparing the results in **Figure 5** one can see that while the capacities of the beams was similar and failure in both cases was in shear, the beam constructed with the customized SCC mix (Beam *M20-1.0%A*) showed an improved level of ductility when compared to the companion beam. This may be linked to the improved concrete tensile properties of this SFRC mixture and the fact that yielding was initiated in the longitudinal reinforcement before the brittle failure in this specimen. As shown in **Figure 5**, while 1.0% fibers was not sufficient to prevent a brittle shear failure in the previous two beams in this series, the use of 1.5% fibers in Beam *M20-1.5%A* was adequate to transform the failure into a ductile flexural response.



(a) 0%, 0.5%, 1%, 1.5%



(b) 0.5%, 0.5%H, 0.75%H



(c) 1.5%, 1.5%(5D)

Figure 4. Results for the M15 series

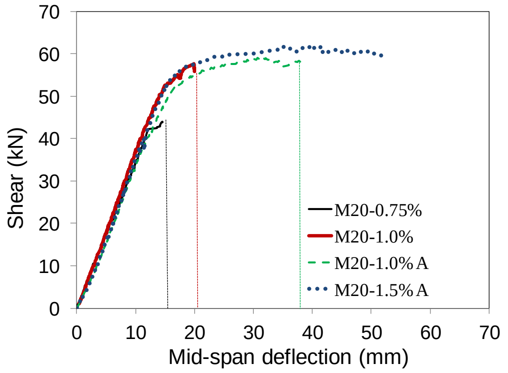


Figure 5. Results for the M20 series: 0.75%, 1%, 1%A, 1.5%A.

Table 5. Results for beams tested in the experimental program

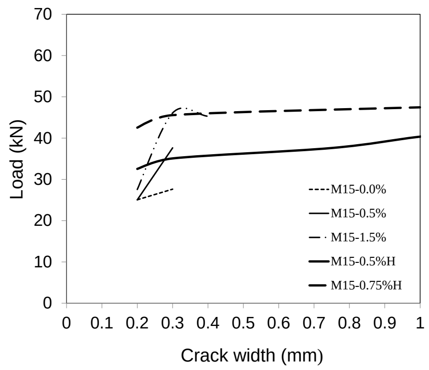
Beam specimen	Max Load (kN)		Increase in Shear (%)	Deflection (mm)	
	Total Load	Shear Load		at Peak	at Failure
M15-0.0%	60	30	---	11	11
M15-0.5%	87	43	47%	18	18
M15-1.0%	96	48*	≥ 63%	41	55
M15-1.5%	93	46*	≥ 60%	24	71
M15-0.5%H	90	45	54%	40	40
M15-0.75%H	95	48*	≥ 63%	37	51
M15-1.5% 5D	104	52*	≥ 73%	33	54
M 20-0.75%	88	44	30%**	15	15
M 20-1.0%	116	58	69%**	20	20
M 20-1.0%A	118	59	74%**	31	31
M 20-1.5%A*	124	62*	≥ 82%**	39	57

*Beam failed in flexure

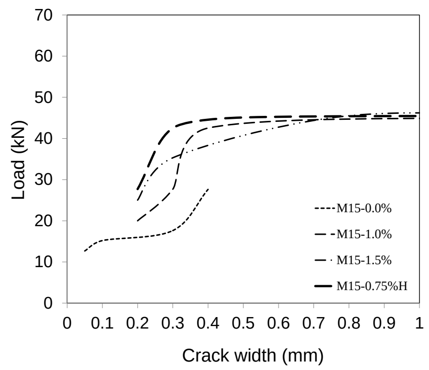
** comparison to the nominal shear strength of a beam without fibers as predicted using the CSA A23.3 general shear design method

5.6 Effect on crack widths

As expected the utilization of steel fibers in the SFRC beams had an important influence on shear and flexural crack widths (plots for the growth of crack widths for a sample of the beams are shown in **Figures 6(a)** and **6(b)**). For the M15-series, the control specimen (Beam *M15-0.0%)* as well as the beams with 0.5% normal strength and high-strength fibers (Beams *M15-0.5%* and *M15-0.5%H*, respectively) failed suddenly in shear. For the control beam the failure was accompanied with very little warning of the eminent shear failure with the crack width just before sudden failure remaining at 0.2 mm. Beam *M15-0.5%* which also failed under shear, was able to resist a slightly larger crack width of approximately 0.4 mm before failure. The use of high-strength BP 80/30 fibers in Beam *M15-0.5%H* permitted this specimen to show very large critical shear crack width (exceeding 1.0 mm) prior to failure (see **Figure 7**). This enhancement is linked to the higher tensile strength and increased aspect-ratio of this fiber type, which results in enhanced pullout behavior allowing the BP 80/30 fibers to more effectively bridge the critical diagonal shear crack and the fact that some flexural yielding had initiated in this beam prior to the brittle shear failure. Beams *M15-1.0%* and *M15-1.5%* of the M15-series failed in flexure. A comparison of the flexure crack widths for these beams demonstrates that increasing the fiber content from 1.0% to 1.5% results in a better control of flexural crack widths at equivalent load stages. The results also show that using 0.75% high-strength BP80/30 fibers results in a better control of crack widths when compared to the companion specimens with 1.0% and 1.5% normal strength fibers. In addition to the above observations, examination of the diagonal shear cracks in the SFRC beams reveals that the fibers pullout rather than fraction across the cracking plane when shear failure occurs. An interesting observation is that the hooked-end fibers deform and straighten during pullout (see **Figure 7**).



(a) Shear crack-widths as a function of shear load



(b) Flexural crack-widths as a function of total load

Figure 6. Effect of fibers on crack widths

Figure 7. Other observations: (left) Control of shear crack up to 1 mm in width prior to failure in beam M15-0.5%H; (right) observed straightening of hooked-end fibers during pullout.

5.7 Effect on crack patterns

In addition to the improvements in controlling crack widths, the use of steel fibers had a noticeable influence on crack patterns in the beams tested in the experimental program. **Figure 8** shows the crack patterns for the various specimens at the end of testing. Comparing Beam *M15-0.0%*, *M15-0.5%* and *M15-0.5%H*, it can be seen that use of 0.5% fibers has reduced the spacing between cracks when compared to the control specimen. The specimen with 0.5% high-strength fibers shows further reductions in crack spacing when comparing to the control beam and the companion specimen constructed with normal-strength fibers.

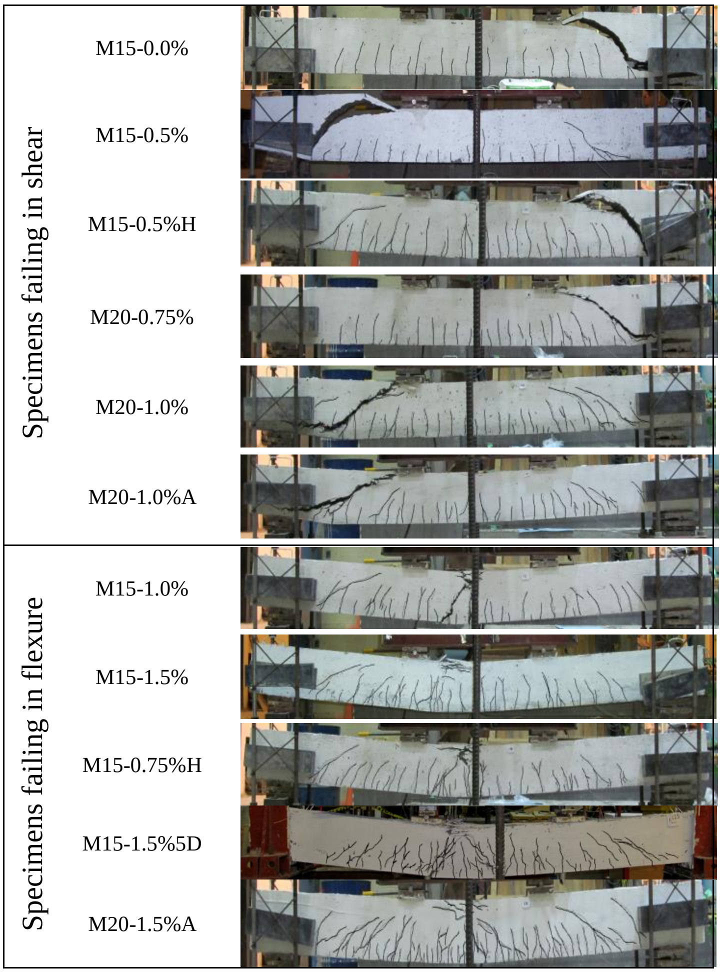


Figure 8. Comparison of crack patterns in the various beams at the end of testing

When comparing the control beam with the specimens that failed in flexure (Beams *M15-1.0%*, *M15-1.5%*, *M15-0.75%H*) one can see that the use of higher fiber content (greater or equal to 1.0%) led to both reductions in crack widths and spacing. In addition, the higher fiber contents resulted in a more diffused cracking pattern with the formation of secondary cracks growing out of primary cracks. This is particularly evident in the failure photo of Beam *M15-0.75%H* which contained high-strength fibers. Similarly the use of 1.5% 5D fibers has also resulted in a more diffused cracking pattern when comparing to the beam with 1.5% ZP305 fibers. For the M20 series, it is noted that Beam *M20-1.0%A*, which was constructed the customized SCC mix and 1.0% fibers, showed significant reductions in crack spacing with multiple flexural and inclined shear cracks despite having failed in shear. Beam *M20-1.5%A* which failed in flexure showed multiple cracking and a more diffused cracking pattern when compared to Beam *M15-1.5%* of the previous test series.

5.7 Comparison to performance requirements in the ACI 318 code

As noted previously, the ACI-318 code now includes provisions permitting the use of SFRC to replace minimum shear reinforcement in beams. In addition to the dimensional and loading limits, the SFRC must have minimum fiber content (0.75% by volume of concrete) and meet minimum performance requirements (residual strength limits from the ASTM C1609 flexural toughness test). In this study the use of steel fibers resulted in more ductile behavior, promoting flexural failure. This tendency increased with higher fiber factor ($v_f\, L_f/d_f$) and demonstrates that when the fiber factor approaches the minimum residual strength requirements specified in the ACI 318 code, the SFRC mix becomes more suitable for improving shear resistance and response.

6 ANALYSIS

6.1 Proposed procedure for predicting the shear resistance of SFRC beam

This section of the paper describes a procedure for predicting the shear resistance of SFRC beams without web reinforcement. In the proposed model the shear resistance of an SFRC beam is assumed to be equivalent to the expected shear strength of a traditional reinforced concrete beam (V_{no}) plus the additional shear resistance provided by the steel fibers (V_{fib}) as shown in Eq. 1:

$$V_{nf} = V_{no} + V_{fib} \qquad (1)$$

Eq. 2 is used to estimate the shear resistance provided by concrete and is the nominal shear strength equation of the general shear design method of the CSA A23.3-04 Standard (CSA, 2004):

$$V_{no} = \lambda\beta\sqrt{f_c'}b_w d_v \quad (2)$$

In the equation, b_w and d_v represent the beam width and effective shear depth, λ is a factor that takes into account the use of low-density concrete, and β is a parameter that accounts for the ability of the concrete to transmit tensile stresses between the cracks. The angle θ is the angle of inclination of the diagonal compressive stresses to the longitudinal axis of the member. In the general design method both these parameters are a function of the longitudinal strain at mid-depth of the cross-section, ε_x, as shown in Eq. 3 and Eq. 4. The crack spacing parameter, s_{ze} is a function of the maximum aggregate size.

$$\beta = \frac{0.40}{1+1500\varepsilon_x} \times \frac{1300}{1000+s_{ze}} \quad (3)$$

$$\theta = (\varepsilon_x \times 7000) + 29° \quad (4)$$

The longitudinal strain at mid-depth of the cross section, ε_x is computed using Eq. 5 for the case of moment and shear:

$$\varepsilon_x = \frac{\left(\frac{M}{d_v}\right) + V}{2 \times E_s \times A_s} \quad (5)$$

In the above expression the shear, V, corresponds to the expected shear resistance of the beam without fibers. The moment, M, represents the corresponding moment at the critical section in the beam (for the beams in this analysis the critical section is taken at a distance d_v from the loading point). A_s and E_s are the cross-sectional area and modulus of elasticity of the longitudinal tension steel, respectively.

The second component of Eq. 1 accounts for the shear strength contribution of the fibers as they pullout across an inclined shear crack (see **Figure 9**) and is predicted using Eq. 6:

$$V_{fib} = 1.72\frac{d}{a} \times N_{fib} \times 0.8F_p \times b_w d_v \cot\theta \quad (6)$$

The effective number of fibers per unit area, N_{fib}, for fibers randomly oriented in three dimensions can be calculated using Eq. 7:

$$N_{fib} = \frac{v_f}{A_f} \times \alpha \times \eta_l \quad (7)$$

Where A_f is the cross-sectional area of the fiber, and where v_f is the volume fraction of fibers in the matrix. The orientation factor, α, is used to account for the random orientation of the fibers crossing any arbitrary cracking plane and is taken as 3/8 (Foster, 2001). The length factor η_l is used to account for the variability in the fiber embedment length across the cracking plane and is taken as 0.5 (Aoude, 2008).

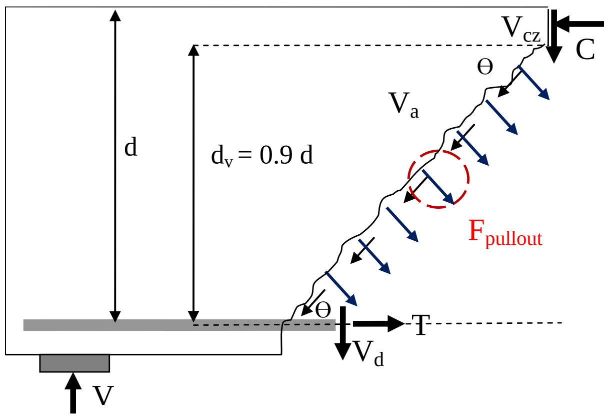


Figure 9. Contribution of fibers to shear resistance with pullout across inclined shear crack

Anchorage properties play an important role in improving the pullout resistance of steel fibers. In the case of hooked-end fibers Alwan et al.(1999) showed that the mechanical contribution of the hook is a function of the cold work needed to straighten the fiber as it is being pulled out from the matrix. It was also demonstrated that this contribution is approximately independent of matrix type and fiber embedment length. In the case of hooked-end fibers, the pullout strength can therefore be estimated by adding a hook contribution (ΔP_{hook}) to the load needed to cause fiber debonding as shown in Eq. 8:

$$F_p = \left(\tau_{bond} * \pi * d_f * \frac{L_f}{2}\right) + \Delta P_{hook} \quad (8)$$

In the above expression, the parameters d_f and L_f represent fiber diameter and fiber length (taken as the length of the straight portion of the fiber). The bond-shear strength, τ_{bond}, is a function of the concrete matrix strength and is determined using Eq. 9:

$$\tau_{bond} = 1.11 \times (f_{ct})^{1.35} \quad (9)$$

where f_{ct} is the tensile strength of concrete taken as $f_{ct} = 0.33\sqrt{f_c'}$. **Table 6** shows data for average bond shear stress determined from single fiber pull-out tests for a range of concrete matrix strengths as reported by Grünewald (2004). **Figure 10** shows that Eq. 9 provides a good fit to the data in **Table 6**.

Table 6. Range of bond shear strength values

Matrix strength class	Concrete strength range f_c' (MPa)	Bond shear stress τ_{bond} (MPa)
Normal strength	≤ 50	2.0-3.0
Medium strength	≥ 50 & ≤ 70	3.4-4.5
High strength	> 70	5.0-6.0

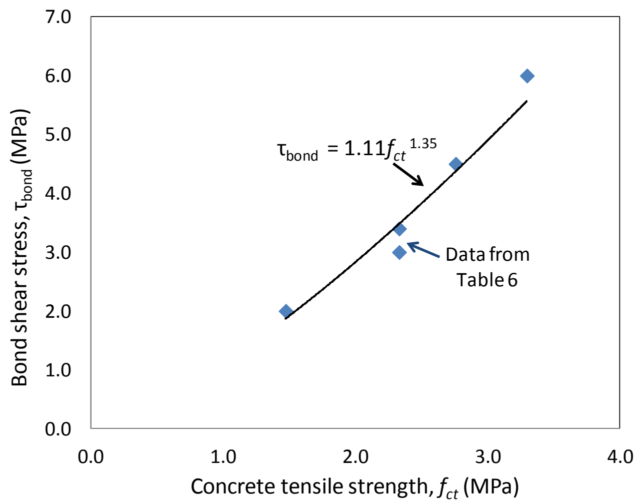


Figure 10. Bond shear strength according to Eq. 9

For a typical hooked-end fiber, the hook contribution to pullout can be estimated using Eq. 10:

$$\Delta P_{hook} = \frac{3.05}{\cos\left(45^o * \pi/180^o\right)}\left(f_{fy} * \frac{\pi\left(d_f/2\right)^2}{6}\right) \quad (10)$$

where f_{fy} is the fiber yield strength (in psi) and d_f is the fiber diameter (in inches).

For the case of crimped fibers the pullout strength is determined using Eq. 1, where the load needed to cause the debonding of a smooth fiber is scaled using a form factor, α_c, that accounts for improved pullout resistance in crimped fibers. In the current study the form factor is estimated to be 2.8 based on regression analysis of data from single fiber pullout tests on crimped fibers (Cohen, 2012):

$$F_p = \alpha_c \times \left(\tau_{bond} * \pi * d_f * \frac{L_f}{2}\right) \quad (11)$$

The pullout strength in Eq. 8 and 11, F_p, is valid for the situation of pure tensile pullout. However in a beam, the fibers will resist tension along the diagonal cracks while undergoing a shear deformation (see **Figure 11**). To account for reduced pullout efficiency in the situation of combined tension and shear, an empirically determined pullout modification factor of 0.8 is applied to Eq. 6 to reduce the pullout resistance of the fibers. The factor was determined based on a regression analysis of a large database of SFRC beam test results (Aoude, 2008).

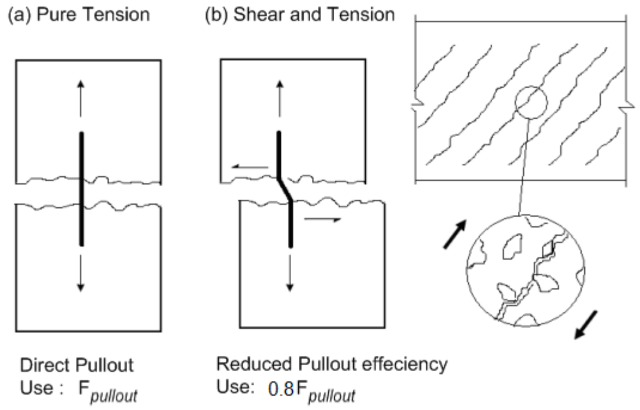


Figure 11. Reduced pullout resistance in the case of combined tension and shear.

Several researchers have studied the effect of shear span-to-depth ratio (a/d) on the behavior of SFRC beams. This effect is shown in **Figure 12** where test data from 5 studies shows that a decrease in a/d ratio leads to an increase in beam shear resistance when all other parameters are kept constant. The figure also shows that as the a/d ratio increases beyond 3.0 the effect of increasing the a/d ratio is present but less pronounced. In the proposed model the effect of a/d ratio is taken into account using the factor "1.72 d/a", determined based on regression analysis of a database of SFRC beam test results from the literature (Cohen, 2012).

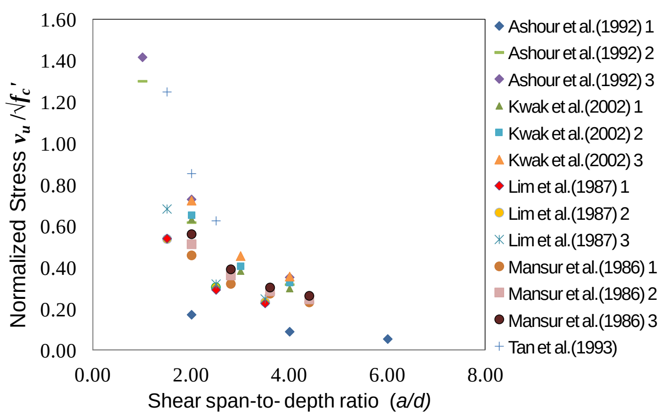


Figure 12. Effect of shear span-to-depth (a/d) ratio on shear strength in SFRC beams tested in the literature

6.2 Analysis results - Database of SFRC beams

The procedure described in this paper was used to predict the shear capacities of a database of SFRC beams tested by other researchers. The database includes results from 129 SFRC beams having effective depths (d) ranging from 180-570 mm, shear-span to depth ratio (*a/d*) ranging from 2.4-6.0, compressive strength (f_c') between 18-104 MPa, longitudinal reinforcement ratios (ρ) ranging from 0.4%-5%, fiber volume fractions (v_f) ranging from 0.25%-2% (Cohen, 2012). **Figure 13** compares the experimental shear capacities of the beams to those predicted using the proposed model. The predictions using the proposed model agree reasonably well with the actual capacities for the majority of the beams with means of 1.11 and 0.94 with standard deviations of 0.20 and 0.21 for the case of beams having hooked-end and crimped fibers, respectively.

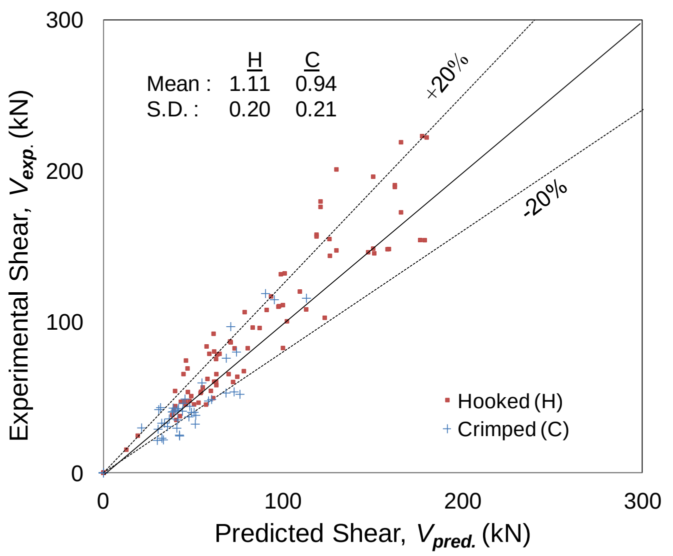


Figure 1. Experimental vs. predicted shear capacities for SFRC beams having hooked-end and crimped fibers.

6.3 Analysis results - SFRC beams from current study

In this section the shear resistance of the SFRC specimens from this experimental program that failed in shear (Beams *M15-0.5%*, *M15-0.5%H*, *M20-0.75%*, *M20-1.0%*, *M20-1.0%A*) are predicted using the proposed model. Various equations proposed in the literature were also considered in the analysis, including equations proposed by Khuntia et al. (1999), Imam et al. (1997), Ashour et al. (1992), Mansur et al. (1986) and Sharma (1986). **Table 7** summarizes the results of the analysis; as can be clearly seen most of the SFRC models from the literature show wide scatter. Similar observations have been made by other researchers when predicting the shear capacities of traditional SFRC beams (Minelli, 2005). **Table 7** shows that the proposed equation gives reasonably good predictions, however the shear strengths are generally over-predicted (over-prediction is also observed for most of the other models considered in the analysis). As discussed previously, Lachemi et al.(2005) found that SCC beams can have reduced shear capacity when compared to traditional concrete beams due to reduced aggregate size and content, which is typical of SCC. Thus the first term in **Eq. 1** may be over-estimating the concrete contribution to shear resistance in SCC beams with reduced aggregate size and content. Further research is recommended to study the effect of these parameters in SCC and SFRC beams.

Overall, the results demonstrate the need of developing reliable and accurate equations for predicting the shear capacity of SFRC beams constructed with SCC and steel fibers.

Table 6. Range of bond shear strength values

Beam I.D.	V_{exp} KN	Predictions (V_{exp} / V_{pred})					
		Khuntia et al. (1999)	Imam et al. (1995)	Mansur et al. (1986)	Sharma et al. (1986)	Ashour et al. (1992)	Proposed Model
M15-0.5%	43	0.84	0.93	0.98	0.93	0.87	0.97
M15-0.5%H	45	0.79	0.88	0.91	0.97	0.81	0.97
M20-0.75%	44	0.72	0.71	0.87	0.95	0.78	0.79
M20-1.0%	58	0.86	0.84	1.01	1.24	0.91	0.93
M20-1.0%A	59	0.86	0.84	1.02	1.22	0.91	0.94
	Mean:	0.81	0.84	0.96	1.06	0.86	0.92
	S.D.:	0.06	0.08	0.07	0.15	0.06	0.07

7 SUMMARY AND CONCLUSIONS

This paper presented the experimental results from eleven slender constructed with SCC and highly workable SFRC tested under four-point loading. The following conclusions are drawn from this experimental program: (1) The addition of steel fibers to a SCC mix improves the shear capacity of reinforced concrete beams and results in improved crack control and damage tolerance; (2) When added in sufficient quantity, provision of steel fibers to a SCC mix can alter the brittle shear failure mode into a more ductile flexural mode in shear-critical beams; (3) The use of high-strength BP80/30 fibers was shown to improve behavior of the SFRC beams by improving ductility and enhancing crack control, although workability of the SFRC was adversely affected; (4) A procedure which modifies the general method of the CSA A23.3-04 was proposed for predicting the shear capacity of SFRC beams and was shown to provide reasonably accurate predictions for a variety of beams tested by other researchers; (4) Existing equations in the literature as well the proposed model were used to predict the shear resistance of the SFRC beams constructed with SCC and steel fibers tested in this study and the analysis demonstrated that there is a need to develop accurate equations for predicting the shear strength of SFRC beams constructed with SCC and steel fibers; (5) Further research is recommended to examine the influence of reduced aggregate size and content on shear capacity in SCC and SFRC beams.

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