New Methodology for Design and Construction of Concrete Members with Complex Stress Fields Using Steel Fiber–Reinforced Concrete

Xuejian Liu, M.ASCE1; Tarun Pareek2; and Shih-Ho Chao, M.ASCE3

Abstract: Reinforced concrete members with significant geometric discontinuities or D-regions experience complex stress fields under loading, which require considerable analytical effort and usually complicated reinforcement detailing. Large openings in RC members can interrupt the direct load transfer provided by concrete struts, thereby leading to overstressed localized regions and unexpected failure modes. Empirically based strut-and-tie models (STMs) are generally used to design the reinforcement detailing of these RC members. However, many prior studies indicate that the resulting details can be very complicated while the actual stress fields deviate significantly from that assumed by STMs, thereby leading to unpredictable failure modes. This study investigates the feasibility of using steel fibers to replace the majority of conventional reinforcing bars in RC deep beams with significant D-regions. The test beams have two large openings, which are located between the loading point and the supports, thus disrupting the direct flow of forces. A simplified procedure is proposed for designing and detailing the reinforcement of steel fiber–reinforced concrete (SFRC) specimens based on the stress fields from elastic finite-element analyses. Experimental results show that when the critical regions of a test specimen were reinforced appropriately by conventional reinforcing bars and the remaining portion of that specimen was reinforced by SFRC with 1.0% volume fraction of fibers, the reinforced SFRC specimen exhibited a ductile failure mechanism with very large plastic deformation. The reinforced SFRC specimens also showed much higher strength than the nominal design load and experienced slow postpeak strength loss. In comparison, although the RC specimen reached very high strength, it also showed an unexpected brittleness and localized failure behavior. This study also shows that finite-element simulation based on the modified compression field theory (MCFT) is able to identify possible failure mechanisms of reinforced SFRC specimens. DOI: 10.1061/(ASCE)ST.1943-541X.0001588. © 2016 American Society of Civil Engineers.

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Introduction

Reinforced Concrete Members with Complex Stress Fields

RC deep beams are generally used as load-transferring elements, such as transfer girders, pile caps, tanks, folded plates, and foundation walls. In buildings, a deep beam (transfer girder) or a wall is used when a large open span is needed for the lower floor. These walls can sometimes extend to multiple floors. Large openings through structural members are frequently required for mechanical and electrical conduits; these openings can also serve as passageways such as doors, windows, and hallways in walls (Muttoni 2011). In-span hinges in RC box-girder bridges are another example of deep RC members with multiple large openings.

These hinges are typically used to accommodate the longitudinal expansion and contraction of the girders, which have very complex internal stress distribution due to the geometries of the seat and discrete bearing locations as well as geometric discontinuities when utility openings exist (Hube and Mosalam 2009). Large openings, if located between the loading point and supports, will disrupt the direct force transfer provided by concrete struts, thus leading to complex stress distribution and concentration and usually significant loss of the load-carrying capacity (Ray 1990). In addition, when the openings are placed close to the edges of the members, it creates localized vulnerable segments whose stress states are difficult to identify and once overstressed, unexpected failure modes can occur. In general, these members with significant geometric discontinuities and complex stress distributions under loading require considerable analytical efforts and usually very complicated reinforcement detailing (Zellerer and Thiel 1956).

Strut-and-Tie Model and Its Limitations

The building code provisions and computer-aided procedures for the design and analyses of structural concrete members where plane strain distribution remains plane (otherwise known as B-regions) are well established today. However, it is still challenging to have a systematic and unified design procedure for members with D-regions, which have complex stress fields and nonlinear strain distributions. These members or regions normally include geometrical discontinuities such as large openings in deep beams or walls, dapped-end beams, and corbels, which experience significant shear
effect or flexural-shear interactions under loading. In the past few decades, significant efforts have been devoted to developing theories, design philosophies, and numerical models toward unified design and analyses of D-regions. Among them, the landmark work by Schlaich et al. (1987) systematically described the strut-and-tie model (STM), a generalized truss model using elastic analysis and load paths to orient truss members and formulate equilibrium based on lower-bound theory of plasticity to obtain member forces. The STM provides a rationale and conservative design method toward design and reinforcement detailing of D-regions by simplifying the complex stress fields with truss systems. The first STM for the design of RC deep beams with large openings was proposed by Schlaich et al. (1987), which has been used in various experimental studies to demonstrate the validity of STM or to develop suitable stress fields (Ruiz and Muttoni 2007; Muttoni et al. 1997; Breña and Morrison 2007, 2008; Maxwell and Breen 2000; Sahoo et al. 2012).

In general, no unique STM exists for a particular structure so the same structure can have different calculated capacities when it is designed by different designers. Prior experimental studies have shown STMs, due to lower-bound theory of plasticity, provide consistent conservative results for deep beams with openings but fail to predict the ultimate load and failure mechanism due to localized damages (Maxwell and Breen 2000; Chen et al. 2002; Park and Kuchma 2007; Tan and Zhang 2007; Ley et al. 2007; Breña and Morrison 2007; Kuchma et al. 2008). Also, some tests have shown that large differences can occur between the calculated forces from STM and the actual instrumented experimental specimens (Breña and Morrison 2007). The actual stress fields in such members are typically very different from the assumed stress field given by STMs, as indicated by several prior experimental investigations (Chen et al. 2002; Park and Kuchma 2007). A poorly detailed STM can lead to unacceptable levels of cracking and damage under service loads, and limited postpeak ductility (Kuchma et al. 2008).

**Effect of Using Steel Fibers in Concrete Members with Complex Stress Fields**

The concept of STMs was originally developed based on the plastic truss analogy, where the structure is assumed to be sufficiently ductile. However, due to the fact that concrete has a limited capacity to sustain plastic deformation, along with the complex stress field after cracking, members with complex regions based on a STM design generally have limited postpeak ductility (Kuchma et al. 2008). In fact, it has been shown that a full plastic mechanism is unlikely to occur in reinforced concrete structures due to the limited plastic deformation capacity of concrete sections in bending as compared to steel (Moy 1996). The only exception is concrete slabs, which typically have only a small amount of reinforcement, and their full plastic mechanism can be predicted by the yield line analysis.

The addition of fibers to improve the inelastic behavior of concrete has been well established. Discrete steel fibers bridge cracks and inhibit the widening of cracks and trigger plastic stress redistribution to a wider area of concrete, which eventually leads to a more ductile and predictable failure mode. Further, the application of steel fibers in replacing shear reinforcement could eliminate the complicated reinforcement detailing. Prior study has shown that RC members with complex stresses can develop large plastic deformation before significant strength degradation even though reinforcing bars were considerably reduced (Sahoo et al. 2012).

**Objectives of This Study**

The main objectives of this study are

1. Investigate the load-carrying performance of steel fiber-reinforced concrete (SFRC) deep beams with two large openings under monotonically increased loading. The addition of steel fibers can significantly reduce conventional reinforcing bars.

2. Develop a simple and unified design procedure for reinforced SFRC (R-SFRC) members with complex stress fields. One control RC specimen and three R-SFRC specimens were designed and tested. Compared to the complex reinforcement detailing as per STM requirements for the RC specimen, the reinforcing bars for the R-SFRC specimens are largely minimized, which results in considerable reduction of the design and construction effort. The reinforcing bars were configured only at the most stressed regions, which were identified from the elastic stress fields at design load to improve the overall flexural strength of the specimens and to prevent undesirable failures due to localized concrete damage under tensile cracking or compressive crushing.

3. Verify internal stress redistribution due to the fiber bridging effect by application of the acoustic emission (AE) technique.

4. Carry out nonlinear finite-element (FE) simulation based on the modified compression field theory (MCFT) to identify possible failure mechanisms and estimate the ultimate strengths of both RC and R-SFRC specimens.

**Experimental Program**

**Specimen Description**

The prototype of the specimens with one opening was originally designed by Schlaich et al. (1987). Four one-quarter-scale simply supported deep beams with two large openings were tested under monotonically increased concentrated loads. The same geometry and dimension have been used in specimens tested by Maxwell and Breen (2000) with one opening and Breña and Morrison (2007) with one and two openings. All the specimens tested in this study were 1,880 mm (74 in.) long, 1,170 mm (46 in.) deep, and 112 mm (4.4 in.) thick. Each specimen had two 380-mm (15-in.) square openings, one located near the bottom-left corner and the other at the top-right region (Fig. 1). The loading was applied at 1,195 mm (47 in.) from end of the hinge support and 685 mm (27 in.) from end of the roller support. The position and size of the openings were arranged to disrupt the direct load paths between the hinges and the supports.

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**Fig. 1.** Specimen geometry, dimensions, and test setup (1 in. = 25.4 mm)
the loading point and the supports. One RC specimen and three R-SFRC specimens containing end-hooked steel fibers of 1–1.5% fractions by volume were tested.

**Specimen Design and Reinforcement Detailing**

**Design of RC Specimen with STM**

Test specimens were designed for an ultimate load-carrying capacity of 151.7 kN (34.1 kips) close to that used in prior research (Breña and Morrison 2007). The detailing of reinforcing bars for the RC specimen was similar to the one in a prior study by Breña and Morrison (2007) as per STM (Fig. 2) and redesigned by using a computer program, CAST (Tjhin and Kuchma 2002). Detailed information regarding the strut geometry, forces, and efficiency factors are given elsewhere (Pareek 2011). In the CAST analysis, the actual material properties of concrete and reinforcing bars were used to compute ultimate strength, leading to a strength of 320 kN (72 kips), which is 2.1 times of the design strength, 151.7 kN (34.1 kips). Fig. 3(a) shows the reinforcement layout for the RC specimen with two layers of #10M (10-mm) bars placed at each location. All reinforcing bars were provided with standard hooks to provide anchorage in order to avoid their pullout.

**Fig. 2.** Design strut-and-tie model for RC specimen (solid lines represent ties and dashed lines represent struts) (data from Breña and Morrison 2007)

**Fig. 3.** Layout of reinforcement (two layers at each bar location) and locations of strain gauges: (a) RC; (b) R-SFRC1; (c) R-SFRC2; (d) R-SFRC3
Secondary reinforcing bars were not used in the RC specimen as per the prior study by other researchers (Breña and Morrison 2007). Although the secondary reinforcement was expected to increase the ductility of the concrete, thus allowing a trusslike plastic mechanism to form, it was hardly evidenced by prior experimental results (Breña and Morrison 2007; Kuchma et al. 2008). Prior experimental studies with the same specimen geometries showed that the RC specimen designed based on STM suffered from severe cracking and crushing of concrete near the supports (Breña and Morrison 2007; Sahoo et al. 2012). To avoid these undesirable localized failures, steel cages were placed over the supports. Each cage was used as a boundary element and was formed by four #10M longitudinal bars and #10M transverse stirrups at a spacing of 100 mm (4 in.). A clear concrete cover of 25 mm (1 in.) to the edge of the steel reinforcing bars was provided.

**Design of R-SFRC Specimens with a Simplified Procedure**

The use of SFRC is based on the expectation that, if breakdown of the most overstressed locations is prevented by the reinforcing bars, the greater plastic deformation capacity of SFRC will allow internal force redistribution and, thus, also increase the ultimate load-carrying capacity (Moy 1996). While deciding the reinforcement layout of R-SFRC specimens, the primary focus was to reduce the complexity of detailing and reinforce only certain critical locations. In this study, a simplified procedure was conducted to configure the reinforcing bars in which a two-dimensional elastic FE analysis was implemented, and the principal stress distributions were generated at the design load level, 151.7 kN (34.1 kips). The contribution of steel fibers is ignored in this elastic analysis for simplicity.

Fig. 4 shows the critical locations where the principal tensile stresses are close to or greater than the modulus of rupture of plain concrete expressed as $f_{t} = 7.5 \sqrt{f_{c'}} = 3.65 \text{ MPa (530 psi)}$, where $f_{c'}$ is the nominal design compressive strength of concrete, which was 34.5 MPa (5,000 psi). The number of critical locations is expected to decrease for members with less complex geometries. The steel reinforcing bars were then placed in those critical locations in two layers by using #10M (10-mm) bars. The total force that the reinforcing bars was provided.

**Mixture Compositions and Material Properties**

The concrete material used for all specimens had a nominal compressive strength of 34.5 MPa (5,000 psi). Table 1 gives the mixture proportion by weight ratio for both RC and R-SFRC specimens. The water amount for the R-SFRC specimens was slightly adjusted to achieve optimum workability, resulting in a water–cementitious materials ratio of 0.386 compared with 0.40 for the RC specimen.

No chemical admixtures or high-range water reducers were added to the concrete mixture. Geometry and mechanical properties of the deformed hooked-end steel fibers used for R-SFRC specimens are given in Table 2. The fiber dosage by volume ($V_f$) was 1.5% for both R-SFRC1 and R-SFRC2, and 1.0% for R-SFRC3, as shown in Table 1.

The compressive strength of concrete was obtained according to ASTM C39/C39M-09 (ASTM 2009), while the tensile properties of reinforcing bars was obtained through tensile coupon tests.
Experimental Testing

All specimens were tested by using a 1,780-kN (400-kips) universal testing machine. The loading was applied monotonically at an increment of 22.5 kN (5 kips) up to the ultimate strength of the specimens followed by manually adjusting the pressure of the hydraulic pump to obtain the postpeak behavior of the R-SFRC specimens (i.e., switching to displacement-control). A 127 × 127 mm (5 × 5 in.) steel plate was placed underneath the 2,670-kN (600-kips) load cell (Fig. 1). In order to compare the load-displacement response, a reference point was needed to measure the displacement. In this study, a point 483 mm (19 in.) below the loading point was selected and a string potentiometer was used to measure the displacement. Because the cracking and deformation in the vicinity of this point could be very different between specimens, the measured elastic stiffness (slope of the load-displacement curves within the ascending part) could be largely affected by this difference. In any case, the overall ductility of the specimens can still be represented by the obtained load-displacement responses. Strain gauges used to measure the rebar strains were mounted at specific locations on reinforcing bars to monitor the strain levels. During testing, loading was put on hold at load increments to examine the crack propagation and measure crack widths. Testing ended when failure was identified, i.e., when the load-carrying capacity significantly decreased. A detailed description of the test setup can be found elsewhere (Pareek 2011).

Test Results

The performance of the test specimens was evaluated in terms of load-displacement responses, cracking pattern and propagation, failure mechanism, and ultimate strength as well as postpeak stiffness and ductility.

Load-Displacement Responses

Fig. 6 shows the load-displacement response for all specimens. The variation of the elastic stiffness of the specimens was discussed previously. The RC specimen exhibited high ultimate strength (2.9 times the design strength 151.7 kN), but it also experienced a brittle failure mode and the abrupt loss of load-carrying capacity upon reaching ultimate strength (see Fig. 7 with details discussed in the following section). In contrast, the R-SFRC specimens exhibited a more ductile behavior. The ductility and strength of R-SFRC
specimens improved once the critical regions were configured with the reinforcing bars. Even with only 1% $V_f$, R-SFRC3 showed the highest postpeak strength, stiffness, and ductility with very gradual strength loss. All the specimens showed higher ultimate strength than the required design load of 151.7 kN (34.1 kip). Even with a lack of longitudinal and shear reinforcing steel, R-SFRC1 reached a strength 1.5 times the design strength purely attributed to the effect of SFRC. R-SFRC2 and R-SFRC3 exhibited comparable strength (2.4 and 2.6 times the design strength, respectively) but experienced distinct postpeak stiffness and ductility. Compared to the obvious shear failure mode of R-SFRC2, R-SFRC3 exhibited a flexural failure mode resulting in a superior ductility and energy-dissipating capacity, as well as better stress redistribution (Fig. 7).

**Cracking Pattern, Propagation, and Failure Modes**

Concrete tensile cracking in the specimens was developed under either a flexural/flexure-shear or a shear deformation mode. Generally, the flexural cracking was first observed at the bottom of the specimen, followed by the shear cracking initiated from the top-right corner of the bottom-left opening, which was consistent with the critical stress regions identified from the principal stress distribution at the design load (Fig. 4). The RC specimen, which had been reinforced according to the STM method, exhibited a distributed cracking pattern [Fig. 7(a)], and the further widening

**Fig. 5.** SFRC material (R-SFRC1 and R-SFRC 2: 1.5% $V_f$; R-SFRC3: 1% $V_f$) testing: (a) three-point bending test setup; (b) load-displacement behavior of beams under three-point bending test; (c) direct tension test setup; (d) stress-strain behavior of dog-bone-shaped specimens under direct tension

**Fig. 6.** Load-displacement response of the test specimens
Fig. 7. Crack propagation in test specimens: (a) RC; (b) failure at the top-right corner of RC specimen at 445.4 kN (100 kips); (c) R-SFRC1; (d) debonding of rebar at the opening of R-SFRC1; (e) R-SFRC2; (f) local failure at section with discontinuous rebar in R-SFRC2; (g) R-SFRC3 without localized undesirable localized failure.
of cracking was suppressed by the reinforcing bars. The first crack occurred at 89 kN (20 kips) from the bottom-right corner of the top-right opening. The specimen had a very high strength but failed in a brittle mode where a large chunk of unreinforced concrete at the top-right corner fractured explosively [Fig. 7(b)]. This region was not reinforced according to STM shown in Figs. 2 and 3(a).

Rather than a diagonal shear crack at the corners of the openings, the first observed crack in R-SFRC1 (with 1.5% fiber by volume) was a flexural crack originating from the bottom of the specimen exactly under the loading point at 133 kN (30 kips). This indicated the effectiveness of SFRC in delaying the occurrence of major cracking as compared with conventional RC. There was no longitudinal reinforcement used at the bottom of R-SFRC1. The flexural cracks originated at this location eventually resulted in failure of the specimen as shown in Fig. 7(c) at 222 kN (50 kips). It was clear that the failure of R-SFRC1 was due to the lack of longitudinal reinforcement along the bottom of the specimen and hence resulted in the low flexural strength. A severe debonding failure near the opening of R-SFRC1 was also observed as shown in Fig. 7(d), indicating the need of a larger cover thickness.

The R-SFRC2 specimen, reinforced with the longitudinal steel at the bottom, developed greater strength and a major cracking pattern along the paths between the loading point and the two openings [Fig. 7(e)]. The first crack was observed at 156 kN (35 kips), which was a flexural crack originating from the bottom of the specimen exactly under the loading point. The diagonal shear cracks started around the openings at 178 kN (40 kips). The strength degradation occurred at 356 kN (80 kips) and was initiated by the localized fracture of the short beam segment under the bottom-left opening where the longitudinal steel was discontinued (see “Design of SFRC Specimens with a Simplified Procedure”). The failure indicated the incompleteness of the STM in detailing the local regions to prevent undesirable localized failure.

The failure mode of R-SFRC3, with only 1% fiber by volume, was similar to that of R-SFRC1, but much more ductile and with a more gradual descending response after peak strength. This is attributed to the improved detailing of reinforcing bars based on the results from R-SFRC1 and R-SFRC2. The first crack occurred at 156 kN (35 kips) and was a flexural crack occurring at the same location as that in R-SFRC2 specimen. The failure started at 387 kN (87 kips) due to excessive flexural cracking. At this stage the strength gradually dropped, but due to the effect of fibers it showed a ductile behavior, and the testing was stopped at a large deflection of 41 mm (1.6 in.). The top-right opening showed very significant plastic deformation as shown in Fig. 7(g). No undesirable localized failures occurred before this plastic mechanism developed.

Crack Detection by AE Technique

Acoustic emission was used to investigate how SFRC allows redistribution of internal stresses in a member with a highly complex stress field. AE is a nondestructive evaluation (NDE) method...
for crack detection and damage evaluation of concrete material and structures (RILEM Technical Committee 2010a, b, c; Aggelis et al. 2011). An AE measuring system detects the motion of stress waves, which cause local dynamic material displacement and then converts these stress waves to an electrical signal with sensors. In this study, seven sensors were deployed on the surface of each specimen to monitor cracking and damage accumulation. The sensitivities of all the sensors were calibrated by using the standard source produced through 0.3-mm (0.012-in.) pencil-lead break (Pareek 2011). Each sensor had a radius of influence of 762 mm (30 in.).

The signal of AE events was amplified through preamplifiers with an amplification gain or threshold being set at 40 dB to eliminate hydraulic and mechanical background noises. The source of an AE event was located by the time difference method, which is based on the arrival time difference of the waves traveling at a known velocity. Three sensors were used to locate a source by triangulation.

Figs. 8(a and c) show the locations of the AE sensors for RC and R-SFRC specimens, respectively, which were selected to monitor cracking in the critical regions. The AE cumulative hits were recorded and synchronized with the application of the loading. The recording of the AE events was affected by the interruption of the stress waves due to the openings and the limited number of available sensors. For each specimen, the number of AE hits increased with the increase of the loading. Figs. 8(b and d) show the distribution of the cumulative AE hits on the specimen surfaces at peak loads of 445 kN (100 kips) for RC and 387 kN (87 kips) for R-SFRC, respectively. It can be seen that the AE hits were more widely and densely distributed in the R-SFRC specimen than in the RC specimen, indicating a redistribution of internal stresses due to fiber bridging. Conventional reinforcing bars in the RC specimen did not effectively help the redistribution of the stresses. The cracking of the short pier next to the top-right opening was not captured due to the locations of the sensors.

Further Discussions on the Test Results

Strut-and-Tie Model

The high ultimate strength of the RC specimen, as anticipated by the lower-bound theory of plasticity, verifies that the design based on STM satisfies the strength requirement. However, an overstrength factor of 2.9 shows a significant overdesign by using STM. The unpredictable brittle failure mode of the RC specimen indicates that members with a complex stress field designed by STM could behave like compression-controlled members; that is, reinforced with a high amount of reinforcing bars but subject to failure in a brittle manner. Also, the actual behavior after cracking can significantly deviate from the assumed stress field by STM. Specifically, the crossing of the diagonal shear cracking over concrete struts (see Strut C1 in Figs. 2 and 7(a)) contradicts the STM where no tensile crack could cross a compression truss member.

Strains in Reinforcing Bars at Ultimate Loads

Table 4 summarizes the strains in the reinforcing bars (labeled in Fig. 3) of the RC and R-SFRC specimens. For the RC specimen, the strain gauge data showed that not all the ties (defined in the STM shown in Fig. 2) yielded at ultimate loads, and only Location 3 [Fig. 3(a)] showed significant inelastic behavior. This indicates that the load transfer path did not quite follow the truss model that was set up to transfer the forces across the D-regions. The majority of reinforcing bars in the RC specimen were not fully utilized. In R-SFRC3, the major yielding locations, 6 and 11, coincided with the predicted critical locations (Fig. 4), which

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Note: B = bottom layer of the bars; T = top layer of the bars. All positive strain values are in tension and negative in compression.

aDamaged strain gauges.

bStrain gauges not used.

cBar yielded.

efficiently utilized the bars to prevent undesirable localized failure, thereby allowing the ductility by SFRC to redistribute the internal forces and develop large plastic deformation.

Failure Mechanisms of RC and R-SFRC Specimens from Nonlinear Finite-Element Simulations

The MCFT, developed by Vecchio and Collins (1986), describes a methodology used to study the behavior of cracked reinforced concrete elements based on equilibrium and compatibility conditions. The tensile and compressive characteristics of the cracked concrete as well as the effect of tensile strains on the compressive strength were described in the model. Frequently, nonlinear finite-element analysis is required to simulate the complex stress fields with appropriate material constitutive models (Ruiz and Muttoni 2007; Muttoni et al. 1997; Wong et al. 2013). The two-dimensional finite-element program, VecTor2 Ver. 3.8 (Wong et al. 2013), based on MCFT (Vecchio and Collins 1986) and the disturbed stress field model (DSFM) (Vecchio 2000) was used to predict the force-displacement behavior and failure mechanisms of both RC and R-SFRC specimens.

Due to the irregular distribution of reinforcing bars, all four specimens were modeled with constant-strain triangular elements for concrete and truss elements for reinforcing bars. Perfect bond was applied between concrete and reinforcement for all
the specimens except the four right-angle hook-shaped bars in R-SFRC2 located at the corners of the openings where cover was not sufficient and debonding was observed during testing. To simulate the debonding behavior in R-SFRC2, the link elements and the Eligehausen bond stress-slip curve (Eligehausen et al. 1983) were used, where the bond stress-slip curve was related to the material properties, reinforcement layout, and confining pressures. Because the embedded bars were not confined, the unconfined Eligehausen model was used to simulate the bond stress-slip relation.

Among the available material models in the program, the Popovics model (Popovics 1973) for concrete’s compressive prepeak response and the Popovics–Mander model (Mander et al. 1988) for concrete’s compressive postpeak response were used. In VectoR2, the slope of the postpeak compressive curve (that is, the confinement effect due to reinforcement) is automatically considered once the amount of reinforcement is defined in the model. The average crack spacing used to calculate crack width was computed with the Comité Euro-International du Béton-Fédération International de la Précontrainte (CEB-FIP) model (1990) by default for all the R-SFRC specimens; however, for the RC specimen, an approximate crack spacing of 50 mm was applied at both x- and y-directions based on the crack pattern observed during testing. For SFRC modeling, the same concrete compression as plain concrete was used because the failures in R-SFRC specimens were due to shear or tensile cracking (Fig. 7). For SFRC tensile properties, the experimental stress-strain relations from direct tensile testing (Fig. 5) were used for the material properties including tension softening. The stress-strain curves from DTT shown in Fig. 5(d) were used for the tensile properties of SFRC materials. As noted previously, although after peak tensile stress the deformation mainly comes from crack opening, the stress-strain curves are still used to represent an average (or smeared) deformation of the SFRC materials. Using this approach has been proven to give the best simulation results (discussed subsequently). The DTT data may underestimate the strength of specimens, especially when the cracking was induced by flexural stresses; therefore, the simulation provided conservative results on the strength and displacement ductility of R-SFRC specimens.

The location and distribution of the loading and support reaction was modeled closely to the real test setup by applying load and restraints at the corresponding nodes. The displacement-controlled loading was uniformly applied along a width equal to that of the bearing plate. Different mesh densities were used and no obvious difference on the results was found. Fig. 9 shows the simulated crack patterns at failure for all four specimens. It can be seen in Fig. 9(a) that the localized failure of the RC specimen at the top-right corner was captured by the analysis; furthermore, cracking of the short beam under the bottom-left opening was also identified. Fig. 9(b) shows a cracking pattern comparable to the pattern in the R-SFRC1 specimen. With the bond failure of the reinforcing bars at the opening corners being simulated for R-SFRC2, the cracking of the short beam under the bottom-left opening and the failure of the specimen was simulated appropriately as shown in Fig. 9(c). Fig. 9(d) shows the simulated flexural

![Fig. 9. Crack propagation of the simulated specimens: (a) RC; (b) R-SFRC1; (c) R-SFRC2; (d) R-SFRC3 (20 times magnification of the deformed shape)](image-url)
cracks of the R-SFRC3 specimen. The large plastic shear deformation in the vicinity of the top opening was approximated and reproduced.

The simulated force-displacement curves for the test specimens are given in Fig. 10. The strength of the RC specimen was slightly overestimated, while those of the R-SFRC specimens were underestimated. The postpeak behavior of the R-SFRC specimens, especially the ductile behavior of the R-SFRC3 specimen, was appropriately simulated. For comparison, the R-SFRC3 specimen was modeled using the simplified diverse embedment model (SDEM) in VecTor2 (Lee et al. 2013) to automatically calculate the tensile stress-strain curve of the SFRC instead of using the DTT stress-strain curves. It was found that the R-SFRC3 specimen modeled with SDEM failed with the widening of the diagonal cracking originating from the bottom-left opening instead of the observed flexural cracking mode, and the ductile behavior is not as well simulated as that with DTT stress-strain curve. In general, the numerical FE model is stiffer than the observed behavior; possible computational issues can be attributed to the use of the constant-strain triangular elements for concrete and the truss elements for reinforcing bars. Both can cause an artificially stiffer response of the model, together with the lack of slippage between concrete and all the reinforcing bars as well as between concrete and steel fibers.

In general, the nonlinear FE analysis created in VecTor2 could be used to predict the actual failure mechanisms and the ductility of either RC or R-SFRC members with complex stress fields. Further improvement on the modeling of R-SFRC members is still under development and it is believed that VecTor2 is a reliable tool for this purpose should suitable refined material models be implemented.

Summary and Conclusions

This study investigated the effectiveness of using steel fibers in reinforced concrete members with significant geometric discontinuities. The use of SFRC in replacing conventional reinforcing bars to improve tensile strength and ductility of concrete was evaluated by the load-carrying capacity and ductility of four one-quarter-scale deep beam specimens. The deep beam specimens had two large openings, which induced complex stress fields upon loading. The RC specimen was designed according to the STM. The three R-SFRC specimens had steel hooked fibers from 1.0 to 1.5% by volume. A simplified procedure for the design and detailing of the R-SFRC specimens was proposed where the three R-SFRC specimens with steel hooked fibers were designed using two-dimensional elastic FE analysis. Reinforcing bars in the R-SFRC specimens were required only at a few critical locations where the tensile stresses exceeded the plain concrete tensile strength. These bars served as ductile links to prevent the premature breakdown of highly stressed regions so that the plastic redistribution of internal forces could be developed through steel fibers. Widespread internal stress redistribution was clearly shown by acoustic emission sensors used in this study.

The primary conclusions of this study are summarized as follows:

- The RC specimen designed according to the STM exhibited a very high ultimate strength (nearly three times that of the design strength) but limited deformation ductility. The failure was due to the unexpected localized damage at the location where no reinforcement was required by STM. Only a limited number of ties yielded at ultimate load, indicating a very different load transfer path than that assumed by STM.
- R-SFRC specimens showed better serviceability than the RC specimen in terms of delaying the crack initiation up to nearly the design strength. The first cracking load in R-SFRC specimens was approximately 50% higher than that of the RC specimen.
- R-SFRC3 specimen with 1% steel fibers by volume was designed according to the proposed simple procedure and exhibited plastic failure mechanism, but with high ultimate strength (2.6 times the design strength). The R-SFRC3 specimens also showed a slow strength loss after peak and high displacement ductility. The reinforcing bars placed at the critical locations effectively prevented undesirable localized failures at these high-stress concentration regions, and thus allowed the subsequent stress redistribution by SFRC. This specimen demonstrates the effectiveness of the straightforward approach in the design of the R-SFRC members with complex stress fields.
- The nonlinear finite-element simulation based on MCFT and DSFM when the direct tensile test data of the SFRC material was used identified possible failure mechanisms and postpeak behavior of the R-SFRC specimens but underestimated the peak strengths of the R-SFRC specimens. Further improvement on the present material models based on an appropriate testing method is warranted.
- The weight of the conventional reinforcing bars used in R-SFRC specimens was approximately 40% of that used in the RC specimen, which eased the detailing and construction of such members under complex stress fields by saving design efforts, time, and labor.
- In this study, due to the scaled size, the beams had a thin thickness of 112 mm (4.4 in.), which could lead to biased orientation of the fibers (47.5 mm long) along the planar direction of the specimens. That amount of fibers along the through-thickness direction could be less when SFRC is used in a full-scale deep beam. The limitation of this research is recognized and further larger scale testing is warranted to further verify the results obtained in this study.

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