Use of Steel Fiber Reinforced Concrete for Enhanced Performance of Deep Beams with Large Openings

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ABSTRACT

Reinforced concrete deep beams are used as primary load distribution elements in various civil engineering structures. Large openings often interrupt the load transfer by concrete struts in these beams and cause a sharp decrease in strength and serviceability. Although the strength evaluation and reinforcement details around the openings are essential considerations, the ACI Building Code does not provide explicit guidance for designing these elements with openings. Strut-and-tie models are commonly used for strength evaluation and design of deep beams with openings. However, reinforcement detailing based on these models can be very complex and the failure of deep beams may be due to localized damages that could not be predicted by the strut-and-tie models. In this study, an experimental investigation was conducted on two concrete deep beam specimens with large single opening, namely, reinforced concrete (RC) and steel fiber reinforced concrete (SFRC), to evaluate their performance under monotonically increased load. The reinforcement detailing in the SFRC specimen was considerably reduced since the steel reinforcement bars were only used for the tensile longitudinal reinforcements and the boundary elements. Both test specimens had significantly higher strength than the designed load computed based on one assumed strut-and-tie model.

INTRODUCTION

Deep beam can be defined as a beam in which either clear span is equal to or less than four times the overall member depth or concentrated loads are within a distance equal to or less than two times the depth from the face of support (ACI Committee 318, 2008). Very often, openings are located through these deep beams to provide access for conduits or mechanical chases. These openings disrupt the stress flow from the loading zones to the supports. Code-specified empirical formulas used to design these members do not explicitly address the design of D-regions with...
openings. Instead, various strut-and-tie models have been used for designing these discontinuous regions.

Past studies (e.g. Maxwell and Breen, 2000; Chen et al., 2002; Kuchma and Park, 2007; Tan and Zhang, 2007; Breña and Morrison, 2007) showed that the strut-and-tie models provide consistent and conservative results in terms of ultimate strength of deep beams with openings. Further, large differences can exist between the calculated forces from strut-and-tie model and actual instrumented experimental specimens. A poorly selected and detailed strut-and-tie model can lead to severe cracking and damages under service loads (Kuchma et al., 2008). From the construction point of view, the primary difficulty associated in the design based on strut-and-tie models is the problem of anchorage and congestion due to large amount of reinforcement bars in the member.

Steel Fiber Reinforced Concrete (SFRC) has gained increased popularity in construction industries in the recent years. Reinforcing plain concrete with steel fibers has been used to reduce conventional steel reinforcement in structural members such as slabs (ACI Committee 544, 1996). SFRC members can exhibit enhanced shear strength, more ductile behavior and reduced crack widths (Dupont et al., 2003). In addition, SFRC offers a multidirectional reinforcement, simple detailing without congestion, and higher post-cracking residual stress and ductility. Past studies (Narayanan and Darwish, 1986; Mansur and Ong, 1991) have shown that including discrete fibers in concrete enhance the strength and the deformation capacities of deep beams in addition to better cracking control.

This paper presents the behavior of a RC deep beam with single opening under the action of monotonically increased loading. The deep beam was designed using a strut-and-tie model and the observed ultimate strength and failure modes were compared with those predicted by the model. Further, a geometrically similar SFRC deep beam was tested under the same loading conditions to compare its behavior, in terms of ultimate strength and failure modes, with that of the RC deep beam. SFRC deep beam consisted of conventional longitudinal steel bars as flexural tensile reinforcement only at the bottom face. The effectiveness of reinforcement detailing at the critical locations and the importance of steel fibers in concrete are recognized in this study to enhance the performance of concrete deep beams with openings.

TEST SPECIMENS

The specimens tested in this experimental study had the same but 1/4 scale geometry as the analytical model originally considered by Schlaich et al. (1987). Earlier, the specimen with the same geometry and dimensions were also tested by other researchers (Breña and Morrison, 2007). The overall dimension of the specimens was 74 in. (1875 mm) long, 46 in. (1170 mm) deep, and 4.5 in. (112 mm) thick. The specimens had a 15 in. (380 mm) square opening near the left-bottom corner of the beams as shown in Figure 1(a). The position and size of the openings in the specimens were selected to interfere with the direct load paths that could
potentially form between the loading point and the supports (Breña and Morrison, 2007). Two types of test specimens (i.e., RC and SFRC) were investigated in the present study. The RC specimen consisted of reinforcement bars designed and detailed as per a strut-and-tie model as discussed later, whereas the SFRC specimen consisted of reinforcement bars for longitudinal tensile reinforcements only at its bottom. Because of complete absence of complex detailing of reinforcement bars, the construction of the SFRC specimen was simple and fast.

There is no unique strut-and-tie model to design a particular discontinuous structure. Strut-and-tie model, originally proposed by Schlaich et al. (1987), approximately follows the elastic (principal) stress distribution was used in the present study for the test specimens (Breña and Morrison, 2007). As shown in Figure 1(b), solid and dashed lines indicate ties and struts in the model, respectively. The imposed load in the test specimen was transferred directly from the loading point to the right support through a bottle-shaped strut, but the opening near the lower left corner impairs the direct load transfer from the load point to the left support. This model also provided a platform to compare the experimental results with those available in the literature.

Test specimens were designed for an ultimate load-carrying capacity of 31.3 kips computed as per ACI Committee 318 (2008) procedure for strut-and-tie models. The nominal compressive strength of concrete was assumed as 5000 psi, whereas the tensile strength of reinforcement bars was considered as 60000 psi. A strength-reduction factor (β_s) equal to 0.75 was used for struts and nodes in the strut-and-tie model. Reinforcing bars in the RC specimen was provided in two layers using No. 3 bars. Clear cover of concrete was provided as 1 in. to the edge of steel reinforcement. Bars ending at nodes located near the beam edges were hooked at their ends to avoid pullout. As shown in Figure 2(a), bottom longitudinal reinforcing bars were anchored at the supports using standard 180-degree hooks. Prior experimental study (Breña and
Morrison, 2007; Flores, 2009) showed that these specimens suffered severe cracking and crushing of concrete near the supports. To avoid these premature local failures in the RC specimen, a steel cage formed by four longitudinal reinforcement bars at the corners and transverse stirrups at a spacing of 4 in. was used as boundary elements on either side of the RC specimen. Further, no wire meshes were used in the RC specimen.

Figure 2(b) shows the reinforcement detailing of the SFRC specimens. As stated earlier, two No. 3 steel reinforcing bars were used only for bottom longitudinal tensile reinforcement. Similar to the RC specimen, steel cages were used as boundary elements near the left and right supports of the specimen. Further, two additional No. 6 bars were placed diagonally (normal to the line connecting the loading point to the corner of opening) in two layers just above the opening so as to restrain the propagation of cracks emanated from the corner of the opening.

A concrete mixture with a nominal 28-day compressive strength equal to 5000 psi was used in all specimens. No chemical admixtures or super-plasticizers were added in the concrete mix. The mix design was carried out for the target compressive strength of concrete so that the optimum quantity and similar proportions of materials could be used in both RC and SFRC specimens. The design mix proportion (by weight) used for both specimens was 1.0 (cement): 0.5 (Fly ash): 1.7 (fine aggregate): 1.0 (coarse aggregate). Type-1 portland cement and Class-C fly ash were used in the mix. The maximum size of coarse aggregate used in the concrete mix was limited to 0.5 in. A constant water-to-cementitious materials ratio of 0.4 was used for both specimens. SFRC specimen consisted of end-hooked steel fibers (diameter = 0.03 in.; length = 2.4 in.; aspect ratio = 80; tensile strength = 150 ksi) of volume equal to 1.5% of the total volume of the specimen. The total weight of steel fibers was 89 pounds as compared to the cement weight of 360 pounds used in the SFRC specimen.
Standard tests (ASTM C31 and C39; ACI 318 Sect. 5.6.2.4) were carried out to evaluate the compressive strength of concrete (six 4 in. by 8 in. cylinders) and the tensile strength of steel reinforcement bars. The nominal and actual properties of concrete and steel reinforcement bars are compared in Table 1. The average values of compressive strengths of concrete at the day of testing of the RC and the SFRC specimens were 6.7 ksi and 6.3 ksi, respectively. The actual yield strength of No. 3 steel reinforcement bars was 81.2 ksi against their nominal value of 60 ksi. Moreover, the ultimate tensile strength of steel reinforcement bars was found to be 126.7 ksi. Tensile testing of No. 6 bars were not carried out because they were designed to be remain elastic. However the state of strain in No. 6 bars was monitored by strain gauges during the tests.

Table 1: Nominal and measured material properties

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Concrete compressive strength (ksi)</th>
<th>Tensile strength of No. 3 bars (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>28-days nominal strength</td>
<td>Measured at the day of testing</td>
</tr>
<tr>
<td>RC</td>
<td>5.0</td>
<td>6.7</td>
</tr>
<tr>
<td>SFRC</td>
<td>6.3</td>
<td></td>
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TEST SET-UP AND INSTRUMENTATION

Both RC and SFRC specimens were subjected to gradually increased monotonic loading by using a universal testing machine of 400 kips capacity. The loading magnitude was increased at an interval of 5 kips until the load-carrying capacity of specimens reduced significantly. As shown in Figures 1(a) and 3(a), a 2-in. diameter steel roller was placed between two 1-in. thick steel plates at the supports to avoid local crushing of the concrete due to bearing. Horizontal restraints were applied to the roller at one support to simulate the ‘hinge’ support condition, whereas the roller at the other support was free to move in both ways to prevent the possible confinement pressure to the supports.

Figure 3. (a) Test set-up used in the present study (b) Comparison of load-displacement response of test specimens
Extensive instruments were used to measure the response of test specimens at different load levels. A 600-kip capacity load cell was used at the load point to monitor the magnitude of load applied to the specimens. Uniaxial strain-gauges (resistance = 120 ohms; gauge length = 0.2 in.) were bonded to the surface of steel reinforcements at specified locations to measure the tie forces at various load levels. As shown in Figure 1(a), four linear variable differential transformers (LVDTs) were mounted on the surface of test specimens to measure the deformation of concrete struts during the testing. Also, a linear potentiometer was used exactly below the load point to measure the deflection of test specimens and two additional linear potentiometers were also used to monitor the movement of both supports.

**TEST RESULTS**

The behavior of test specimens under the action of monotonic loading was monitored at various stages of loading. The primary parameters investigated in the present study are load versus deflection response, mode of failure and ultimate strengths of specimens as discussed in the following sections.

**Load-Deflection Response**

The deflection of the specimens just below the loading point was monitored at the different stages of loading. The load-deflection behavior of the RC specimen is shown in Figure 3(b). The RC specimen showed almost linear response up to 95 kips beyond which yield plateau and strain-hardening was observed up to peak load of 132.1 kips in the load-deflection response. The design load-carrying capacity of the specimen was 31.3 kips, which was estimated by using the strut-and-tie model. Thus, the RC specimen was over-designed by a factor of 4.2. The excellent post-yield behavior of the RC specimen suggested that the bottom longitudinal reinforcement bars were yielded in tension. This was also confirmed from the state of strains in these steel reinforcement measured using uniaxial strain gauges. The delaying of premature local failures of the specimen near the supports due to steel cages at the boundaries helped the RC specimen to exhibit excellent displacement ductility of nearly 4.0. As expected, the descending branch of the load-deflection response of the RC specimen exhibited by the sudden drop from its peak strength.

Figure 3(b) also shows the load-deflection behavior of the SFRC specimen. The SFRC specimen showed nearly a linear response up to a peak load of 96.8 kips, which was much higher than the design load of 31.3 kips that used for the RC specimen. It should be noted that even though there were absolutely no steel reinforcement bars used as struts and ties as per strut-and-tie models, the SFRC specimen reached 3.0 times the design load. Further, the SFRC specimen showed more gradual post-peak descending branch in the load-displacement response in contrast to a sudden drop in case of the RC specimen indicating significant contribution of steel fibers to the residual strength of the specimen. The boundary elements in addition to diagonal steel reinforcement bars helped the specimen to
achieve the design strength without premature local crushing and cracking of concrete at the boundaries.

Mode of failure

Figure 4(a) shows the observed crack pattern in the RC specimen. As expected, the local failure at the supports of the RC specimen was not observed during the entire loading because of sufficient confinement action proved by the steel cage to the compressive stresses. The major flexural crack running from the bottom face of the specimen to the loading point was observed at the failure stage. The specimen collapsed due to shear failure of concrete in the horizontal segment of the opening due to lack of shear reinforcement as shown in Figure 4(b). This ultimately led to the fracture of bottom longitudinal tensile reinforcing bars after reaching their failure strains. It is interesting to note that the major cracks were developed away from the opening region indicating that the flow of force was least affected by the opening due to local strengthening of the boundaries near supports.

![Figure 4. Mode of failure of the RC specimen](image)

(a) Overall state of specimen at the failure stage (b) Shear failure of concrete in bottom segment of the opening

![Figure 5. Failure mode of the SFRC specimen](image)

(a) Specimen at the failure stage (b) Failure of horizontal segment of the opening
As shown in Figure 5(a), the failure mode of the SFRC specimen was completely different from that of the RC specimen. A major crack (compressive strut) was developed just above the opening of the SFRC specimen. Due to presence of diagonal steel reinforcement bar, the major crack deviated from the corner of the opening and formed at the end of this bar. A plastic mechanism was developed after several plastic hinges formed at the specimens (Figures 5(a) and 5(b)). The failure was fairly ductile as evidenced by the large deformation of the specimen shown in Figure 5(a).

Ultimate strength

As stated earlier, the design strength of test specimens was 31.3 kips. The specimen was analyzed by a strut-and-tie computer program, CAST (Tjhin and Kuchma, 2002). This program has a feature that allows the analysis of nodes to ensure that geometry and stress limits are not exceeded. Using nominal (specified) values of material strength and assuming a strength reduction factor as 0.75, the nominal ultimate strength of the RC specimen was estimated to be 41.2 kips. The expected actual capacity of the specimens was 70.3 kips considering the actual material properties obtained from coupon testing of concrete and steel bars and the strength-reduction factor as unity. The ultimate strength of the RC specimen was observed as 132.1 kips which was 1.9 times the expected strength predicted by the computer model. The main reason for the large difference between measured and predicted strengths is the redistribution of stress in the specimen after the yielding of steel bars acting as tie members. Similarly, the SFRC specimen had ultimate strength of 96.8 kips which was about 1.4 times the predicted ultimate strength of the RC frame. It should be noted that the ultimate strength of the SFRC specimen was higher than the predicted ultimate strength (70.3 kips) of the RC specimen even though no steel reinforcement bars were used as tie members. Further, the fiber bridging effects limited the widening of cracks and allowed significant stress redistribution and a plastic mechanism formation upon failure.

SUMMARY & CONCLUSIONS

This paper investigates the behavior of deep beams with large opening that were designed using strut-and-tie model. One reinforced concrete (RC) and one steel fiber reinforced concrete (SFRC) specimens were tested under monotonically-increased loading. The main objectives of this paper were: (a) to investigate the effect of local strengthening on load-transferring mechanism and failure modes of test specimens; (b) to study the behavior of SFRC specimen and compare with the RC specimens designed using strut-and-tie models; and (c) to identify critical regions that are not identified by strut-and-tie models and to suggest reinforcing detailing to avoid localized failures.

The following conclusions were drawn in this present study:
1. Design strut-and-tie models significantly underestimated the strength of specimens. RC specimen showed the load-carrying capacity greater than four times of the nominal strength predicted using the design strut-and-tie model.

2. SFRC specimen reached the three times of design strength of RC specimen even though steel reinforcement bars were not used as tie members except only bottom longitudinal tensile bars.

3. Local strengthening of test specimens using steel cage at the supports significantly enhanced load-carrying capacity and changed the mode of failure to a much ductile manner. As a consequence, significant bending (flexural) action was noticed in the RC specimen without any premature local failures.

4. Steel fibers in the SFRC specimen restrained the widening of crack growth and increased the number of cracks. This helped in better stress redistribution in the specimen even if the steel reinforcement bars were not present. A ductile plastic mechanism was developed after the formation of plastic hinges. Further research is needed to investigate the effects of volume fraction of steel fibers on strength of members designed as per strut-and-tie method.

5. The construction of RC specimen was usually time-consuming and labor-intensive due to complicated detailing of reinforcing bars in contrast to the SFRC specimen. Hence, the complete replacement of conventional reinforcing bars by deformed steel fibers at a volume of 1.5% can be a feasible alternative to the current practice.

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