

Shear strength of steel-fiber-reinforced deep hollow-core slabs

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- ACI 318-14 permits no shear reinforcement in prestressed concrete hollow-core units less than 12.5 in. (320 mm) deep.
- An investigation of 18 in. (460 mm) deep steel-fiber-reinforced hollow-core slabs showed that ultimate shear strengths were approximately twice those predicted by ACI 318-14.
- Steel-fiber-reinforced concrete is more efficient in hollow-core slabs than in typical beams due to a preferential fiber orientation in thin webs that enhance fiber bridging efficiency.

Prestressed concrete hollow-core slabs have been used since the 1950s.¹ They are commonly used in residential, industrial, or commercial buildings; parking structures; and other civil structures throughout the world. The typical thickness of hollow-core units in the United States is between 6 and 12 in. (150 to 300 mm). Prestressed concrete hollow-core slabs are widely known for their low maintenance cost; speed of construction; reduced weight; long, economical spans; better control of deflection and flexural cracking; acoustical and heat transfer control; and fire resistance. The same shear design criteria given in the American Concrete Institute's (ACI's) *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*² are used for both hollow-core slabs and typical prestressed concrete flexural members. ACI 318-14 (Section 7.6.3.1) requires shear reinforcement for hollow-core units with a total untopped depth greater than 12.5 in. (320 mm), where the factored shear force at section V_u exceeds $0.5\phi V_{cw}$ (where V_{cw} is nominal shear strength provided by concrete when diagonal cracking results from high principal tensile stress in web; ϕ is the strength-reduction factor and equals 0.75). The depth and shear demand limitations were based on research results that indicated that prestressed concrete hollow-core slabs with a depth greater than 12.5 in. generally showed web-shear strength in end regions much less than that predicted by the ACI 318-14 V_{cw} equation.^{1,3}

By contrast, flexural-shear strengths in those tests were generally higher than the strengths predicted by ACI 318-14 Eq. (22.5.8.3.1a). The common practice is to increase the depth of the section when shear demand exceeds the calculated capacity rather than providing shear reinforcement due to difficulties in production and to economic factors. As a consequence, the span or loading is generally limited by the available shear strength. For instance, the allowed uniform load based on shear capacity will drop to about 20% for a typical 8 in. (200 mm) deep hollow-core slab when the span is extended from 10 to 30 ft (3 to 9 m).⁴ To achieve the same shear capacity, the section needs to be five times deeper, but according to ACI 318-14, the depth of the section can be increased to only 12.5 in. without additional shear reinforcement. This means that the loading on a 30 ft slab can be increased by only 31% of the loading possible on a 10 ft slab. This situation could be even more critical where there is a loading, such as in an industrial application, or where there are loads on the slabs.

Several experimental investigations have been conducted on the shear capacity of hollow-core slabs in which most of the specimens had depths less than 12 in. (300 mm).¹ While Becker and Buetter confirmed that the shear capacity of a prestressed concrete hollow-core slab was higher than the strength calculated according to ACI 318-83⁵ (shear strength equations are essentially unchanged since 1963⁶), all previous research has been limited to section depths of no more than 10 in. (250 mm). Meanwhile, hollow-core slabs with depths greater than 12 in. are commonly produced in Europe and are being produced in the United States. In addition, 59 tests conducted by Yang⁷ indicated that while the flexure-shear capacity from *Swedish Regulation for Concrete Structures*⁸ code provisions agreed with the test results, the web-shear capacity from test results appeared to be considerably scattered compared with code predictions. Based on Hawkins and Ghosh's recommendations,¹ ACI 318-08⁹ imposed a 50% reduction for the web-shear strength as the limiting shear force that can be applied on unreinforced hollow-core slabs with a total untopped depth no greater than 12.5 in. (320 mm). Palmer and Schultz¹⁰ conducted experiments on hollow-core slabs with 16 and 20 in. (410 and 510 mm) depths with a/d_p ratios ranging from 2.5 to 4.0, where a is the shear span and d_p is the distance from the extreme compression fiber to the centroid of the prestressing steel, and found that the shear strengths from these test results were substantially scattered. They also reported a considerable disparity in the strand-end slip in hollow-core slabs with 16 in. depth and concluded that web-shear strength decreased as strand-end slip increased.

ACI 318-14 shear-strength prediction

ACI 318-14 recognizes two different types of shear cracking for prestressed concrete members: web-shear cracking and flexure-shear cracking. The flexure-shear cracking-strength

expression was derived by adding the incremental shear force (the first term of ACI 318-14 Eq. [22.5.8.3.1a]). For the web-shear cracking strength, the equation was derived based on a simplification of the classical principal stress formula where the maximum principal tensile stress is equal to the tensile strength of the concrete. Details for the derivations of the ACI 318-14 predicted shear strength equations can be found in ACI 318-63.⁶

In ACI 318-14, the nominal shear strength provided by the concrete V_c for hollow-core slabs with total untopped depth not greater than 12.5 in. (320 mm) shall be the lesser of the flexure-shear cracking strength V_{ci} and the web-shear cracking strength V_{cw} , where V_{ci} and V_{cw} are expressed in the following equations:

$$V_{ci} = 0.6\lambda\sqrt{f'_c}b_wd_p + V_d + \frac{V_iM_{cre}}{M_{max}} \quad \text{Eq. (ACI 22.5.8.3.1a)}$$

where

λ = lightweight concrete modification factor

f'_c = specified compressive strength of concrete

b_w = web width

V_d = shear force at section due to unfactored dead load

V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}

M_{cre} = moment causing flexural cracking at section due to externally applied loads

M_{max} = maximum factored moment at section due to externally applied loads

$$V_{cw} = \left(3.5\lambda\sqrt{f'_c} + 0.3f_{pc}\right)b_wd_p + V_p \quad \text{Eq. (ACI 22.5.8.3.2)}$$

where

f_{pc} = compressive stress in concrete at centroid of cross section resisting externally applied loads after allowance for all prestress losses

V_p = vertical component of effective prestress force at section

For hollow-core slabs, V_p generally equals zero because the prestressing strands are normally not draped. When the total untopped depth of hollow-core slabs is more than 12.5 in. (320 mm), ACI 318-14 section 7.6.3.1 allows no shear reinforcement when the factored shear force at section V_u exceeds $0.5\phi V_{cw}$ (where ϕ equals 0.75).

Hollow-core-slab shear-enhancement technique

The minimum shear-reinforcement exception in ACI 318-14 creates concerns for hollow-core slabs deeper than 12.5 in. (320 mm) because shear reinforcement is virtually impractical for hollow-core slabs. Girhammar and Pajari¹¹ proposed a construction method to increase the shear capacity of hollow-core slabs by reducing the depth of sections while simultaneously increasing the concrete-topping thickness. According to their tests on 10.4 in. (260 mm) deep hollow-core slabs, reducing the section thickness by 2.75 in. (70.0 mm) with a 1.24 in. (31.5 mm) concrete topping, the shear capacity was increased by 30%. While this approach is doable, it somewhat defeats the purpose and reduces the benefits of the hollow-core-slab system, as the overall weight of the unit increases while time- and labor-associated costs increase as well. Another common practice to increase the shear strength of hollow-core slabs is to fill a portion of the cores during casting while the concrete is still plastic. This approach has been used since the 1980s.¹² However, limited research has been conducted to investigate the effectiveness of this shear-enhancement technique. Palmer and Schultz¹⁰ tested 16 in. (410 mm) deep hollow-core slabs with one of the four cores filled by the same concrete used for the specimens. The core was filled for a length of 4 ft (1.2 m) at the ends immediately after the specimens were extruded. Test results indicated that the bond between the filled core and the surrounding concrete was questionable. The contribution from the filled cores to the web-shear strength was approximately 50% of that predicted by the ACI 318-14 V_{cw} equation. They inferred that the low efficiency might be due to the anchorage loss of the prestressing strands at ultimate load. They also did an experiment using plastic fibers to reinforce concrete with volume fractions of 0.67% and 1% using dry-cast fabrication. Their results showed that the web-shear strength increased by 17% compared with unreinforced specimens when 0.67% fiber content by volume was used. When the fiber content was increased to 1.0%, the web-shear strength increased by 44% compared with conventional concrete.

Numerous tests have been conducted on the shear behavior of steel-fiber-reinforced concrete beams over the past decades, and results have shown that steel fibers can considerably increase the shear strength and ductility of concrete.¹³ Due to the increasing evidence from these research results, ACI 318-08 started allowing steel fibers as an alternative to conventional shear reinforcement (that is, steel stirrups) used in prestressed and nonprestressed concrete beams. For the same reason, it is expected that the use of steel fibers could significantly enhance the shear capacity of prestressed concrete hollow-core slabs without much construction difficulty. Hence, it is important to investigate the efficiency of steel fibers in enhancing the shear strength of deep hollow-core slabs. It is worth mentioning that fiber reinforcement adds a host of benefits

to the performance of a concrete structure in addition to increasing shear resistance. These benefits include controlling of cracking due to camber and release of strands at the end zones of prestressed members, ductility, and increased impact- and energy-absorption capacity with reduced crack widths—which all resulted in marked improvement of durability of the structure.¹⁴ Peaston, Elliott, and Paine¹⁵ investigated the potential of steel fibers as secondary reinforcement in dry-cast 200 mm (8 in.) deep hollow-core slabs with a/d_p ratios of 2.0 and 2.8. Test results showed increases of the first cracking load and ultimate shear capacity with substantial improvement in postcracking ductility in steel-fiber-reinforced slabs compared with plain concrete slabs. They also pointed out that no adjustment for the extruding machine was needed to allow the use of steel-fiber-reinforced concrete (up to 1% by volume fraction). However, to the best knowledge of the authors, no experiment has been conducted to investigate such efficiency for steel-fiber-reinforced hollow-core slabs with depths greater than 8 in. (200 mm).

Experimental program

The experimental program was designed to investigate the shear behavior of steel-fiber-reinforced deep hollow-core slabs. A total of eight full-scale slabs 18 in. (460 mm) deep, 48 in. (1220 mm) wide, and 192 in. (4900 mm) long (**Fig. 1**) were fabricated by professional workers at a local precast concrete plant. Two control specimens were fabricated using plain concrete with the nominal aggregate size of $\frac{3}{8}$ in. (9.5 mm). Approximately 90 to 95% of the aggregates had this nominal size. Six specimens were cast using the same concrete reinforced with 0.75% fiber volume fraction. **Figure 2** shows the fabrication of the specimens.

Specimen design

Specimens were designed in accordance with ACI 318-14 and the *PCI Manual for the Design of Hollow-Core Slabs*¹⁶ to withstand the ultimate flexural loads at the corresponding loading points. Six additional no. 4 (13M) mild steel reinforcing bars (ASTM A615¹⁷ Grade 60 [410 MPa]) were used in each specimen to prevent premature flexural failure before the specimens failed in shear. The total longitudinal reinforcement ratio for the prestressing strands and the mild steel reinforcing bars was approximately 0.7% for the specimens. The increase in shear strength due to the addition of mild steel reinforcing bars was marginal, according to a prior study on the effect of flexural reinforcement on shear strength of reinforced concrete beams and slabs.¹⁸ The voids were created through the use of closed-cell extruded polystyrene foam blocks (**Fig. 2**). A total of six 0.5 in. (13 mm) diameter low-relaxation prestressing strands (ASTM A416 Grade 270 [1860 MPa yield]) were used in each slab. **Figure 1** shows the cross section of the hollow-core slabs used in this



Concrete mixing with steel fiber



Preparation of formwork



Specimen casting



Concrete vibration



Specimen finishing



Specimen curing

Figure 2. Specimen fabrication.

temperature before the prestressing strands were cut. Cylinders and beams were cured in a controlled curing room in accordance with ASTM C192²¹ until the day of the test.

Materials

Concrete The specified 28-day concrete compressive strength of 5,000 psi (34 MPa) with a target water-cementitious material ratio of 0.50 was prepared at a local pre-

Table 1. Concrete proportions: Weight of materials per cubic yard

Material	Weight, lb
ASTM C-150 Type I portland cement	408
ASTM C-618 fly ash/Class C	102
ASTM C-33 pea gravel (#8)	1737
ASTM C-33 river sand	1452
Water	277

Note: 1 lb = 4.448 N.

cast concrete manufacturer for all of the specimens. The concrete mixture proportions (Table 1) are typical of this plant’s commercial precast concrete products (wet cast). Plain concrete was mixed in a pan mixer for the control specimens. The second batch of concrete using the same mixture proportions with the added steel fibers was used for the steel-fiber-reinforced specimens.

Three compressive-strength tests were performed on 4 × 8 in. (100 × 200 mm) cylinders in accordance with ASTM C39²² for both the plain and the steel-fiber-reinforced concretes at 28 days and on the day of test of each specimen. The mean 28-day compressive strength was 5250 psi (36.2 MPa) for the plain concrete cylinders and 4990 psi (34.4 MPa) for the steel-fiber-reinforced concrete cylinders.

Steel fibers Figure 3 shows the hooked-end steel fibers conforming to ASTM A820.²³ The effect of fiber orientation and distribution is discussed in the Results section. Table 2 shows the mechanical properties of the steel fibers used in the experimental program.

Test setup and procedure

For concrete beams without shear reinforcement, the shear strengths vary with the shear span-to-depth ratios (Fig. 4).^{24,25} The setup was designed to generate two failure



Figure 3. Steel fibers. Note: 1 in. = 25.4 mm.

Table 2. Mechanical properties of steel fibers

Dimensions			Tensile strength, ksi
Length <i>L</i> , in.	Diameter <i>D</i> , in.	Aspect ratio, <i>L/D</i>	
1.97	0.03	67	~159.5

Note: The information is provided by the technical paper from the manufacturer. 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.

modes: web-shear failure, which has a span-to-depth ratio approximately less than 3.0; and flexure-shear failure, which has a greater shear span-to-depth ratio. Due to the lack of a standard test method in the United States, the web-shear-failure-mode test setup was prepared according to the requirements described in EN 1168,²⁶ which recommends the applied load span across the whole width of the slab at a distance of 2.5*h* from the support with a minimum distance of 600 mm (24 in.), where *h* is the depth of the slab. This translates to an *a/d_p* ratio of 2.73 for the test specimens. A total of four specimens, two control specimens and two fiber-reinforced specimens, were tested. For the flexure-shear failure mode, the load was applied at the midspan of the specimen with an *a/d_p* ratio of 5.09 for one of the fiber-reinforced specimens. Additional test setups with *a/d_p* ratios of 2.0, 3.45, and 4.36 were performed to investigate the shear behavior (Fig. 5). For an *a/d_p* ratio of 2.0 (that is, the loading point is 33 in. [840 mm] from the support), the section is beyond the end of the transfer length, which is approximately 25 in. (635 mm) from the end of the slab. Figure 4 notes all of the *a/d_p* ratios used in this research. Although short bearing lengths of 3 to

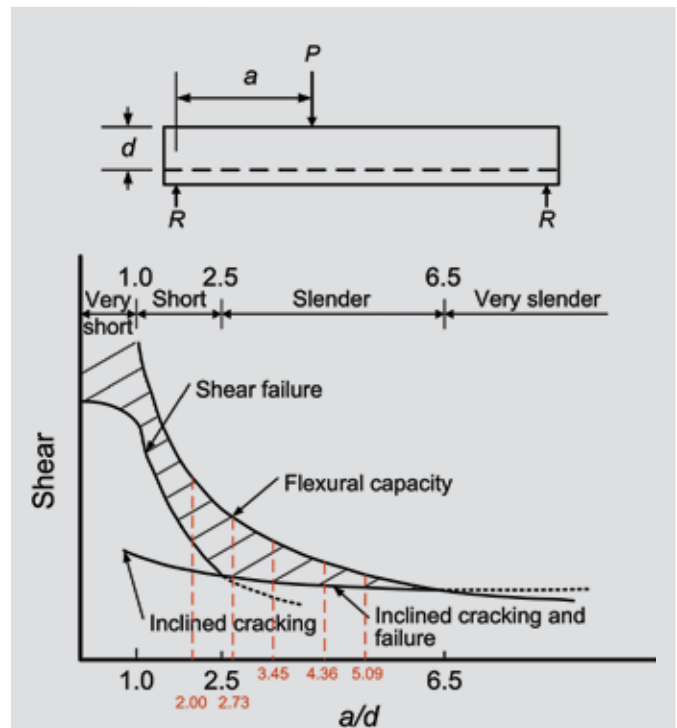
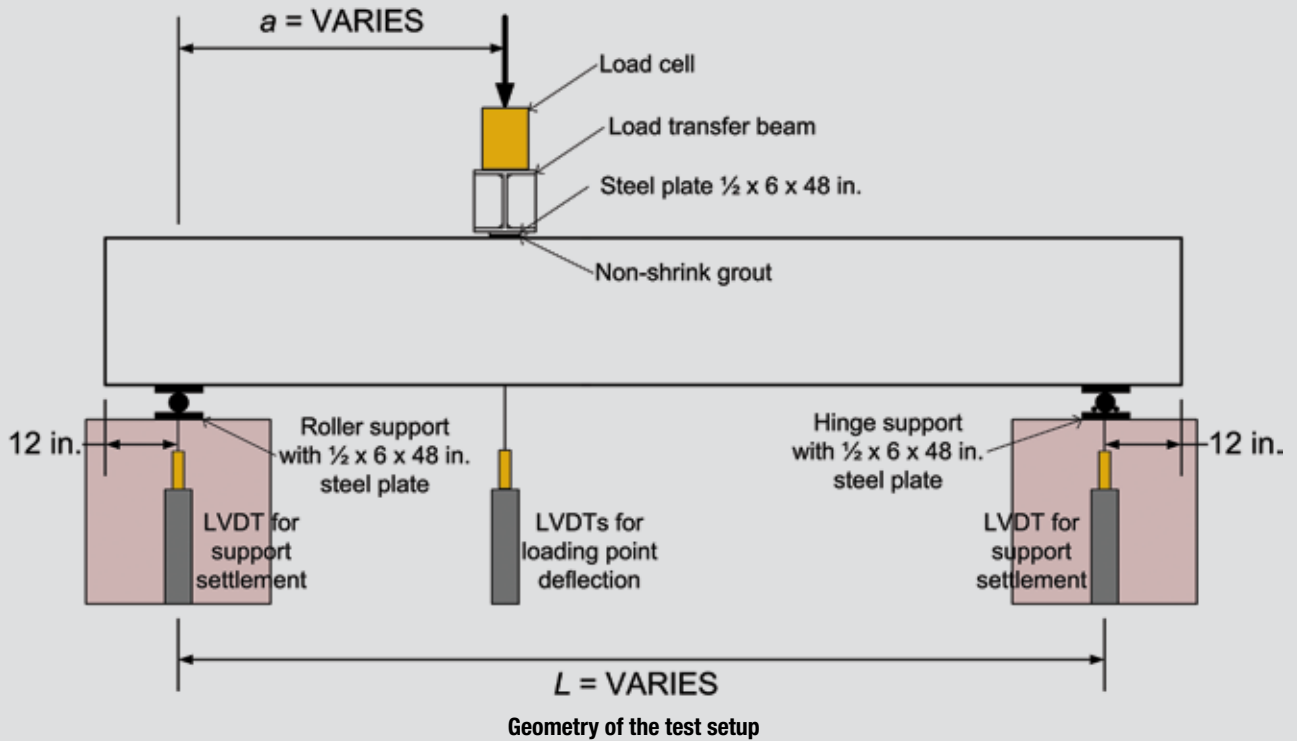


Figure 4. Failure mechanisms with various shear span ratios. Note: *a* = shear span; *d* = distance from extreme compression fiber to centroid of longitudinal tension reinforcement; *P* = load; *R* = reaction.



Test setup



European test setup



Midspan test setup

Figure 5. Experimental test setups. Note: a = shear span; L = span length; LVDT = linear variable differential transformer. 1 in. = 25.4 mm.

4 in. (75 to 100 mm) are typically used with hollow-core slabs, the centers of the steel supports were placed 12 in. (300 mm) from the end of the specimens to minimize any confinement effect to the strand ends due to the vertical reaction at the supports.²⁷ Tests by Palmer and Schultz^{10,28} indicate that significant anchorage failure and slip were noticed at the ends of the hollow-core-slab specimens. The vertical reaction can significantly enhance the bond strength. Although V_{cw} will slightly increase at the failure sections that are closer to the end of the transfer length, this has no effect on the test results. This is because this study was investigating the first term ($3.5\lambda\sqrt{f'_c}$) of ACI 318-14 Eq. 22.5.8.3.2, which is the contribution from the concrete properties, rather than the second term ($0.3f_{pc}$), which considers the contribution from the prestress. The contribution of the prestress can be applied depending on the actual location of the section.

Load was monotonically applied through a 600 kip (2700 kN) hydraulic cylinder mounted on a steel reaction

frame in 10 kip (44 kN) increments up to failure. Loading was paused regularly for detailed investigation of the cracking patterns, including marking cracks and photographing the specimens. A deep wide-flange steel section with reinforced web stiffeners was used to spread the applied load over the width of the slab (Fig. 5).

Instrumentation

Two linear variable differential transformers (LVDTs) were used to measure the vertical displacement directly under the load, one at the middle of the slab and the other 12 in. (300 mm) in the transverse direction as a backup (Fig. 5). Two additional LVDTs were placed at the supports. A 600 kip (2700 kN) manual hydraulic cylinder was used to apply the load to the specimens. A 500 kip (2200 kN) load cell was placed between the hydraulic cylinder and the load spreading beam. A data-acquisition system was used to collect the applied load and displacement from the sensors at a rate of 5 Hz.

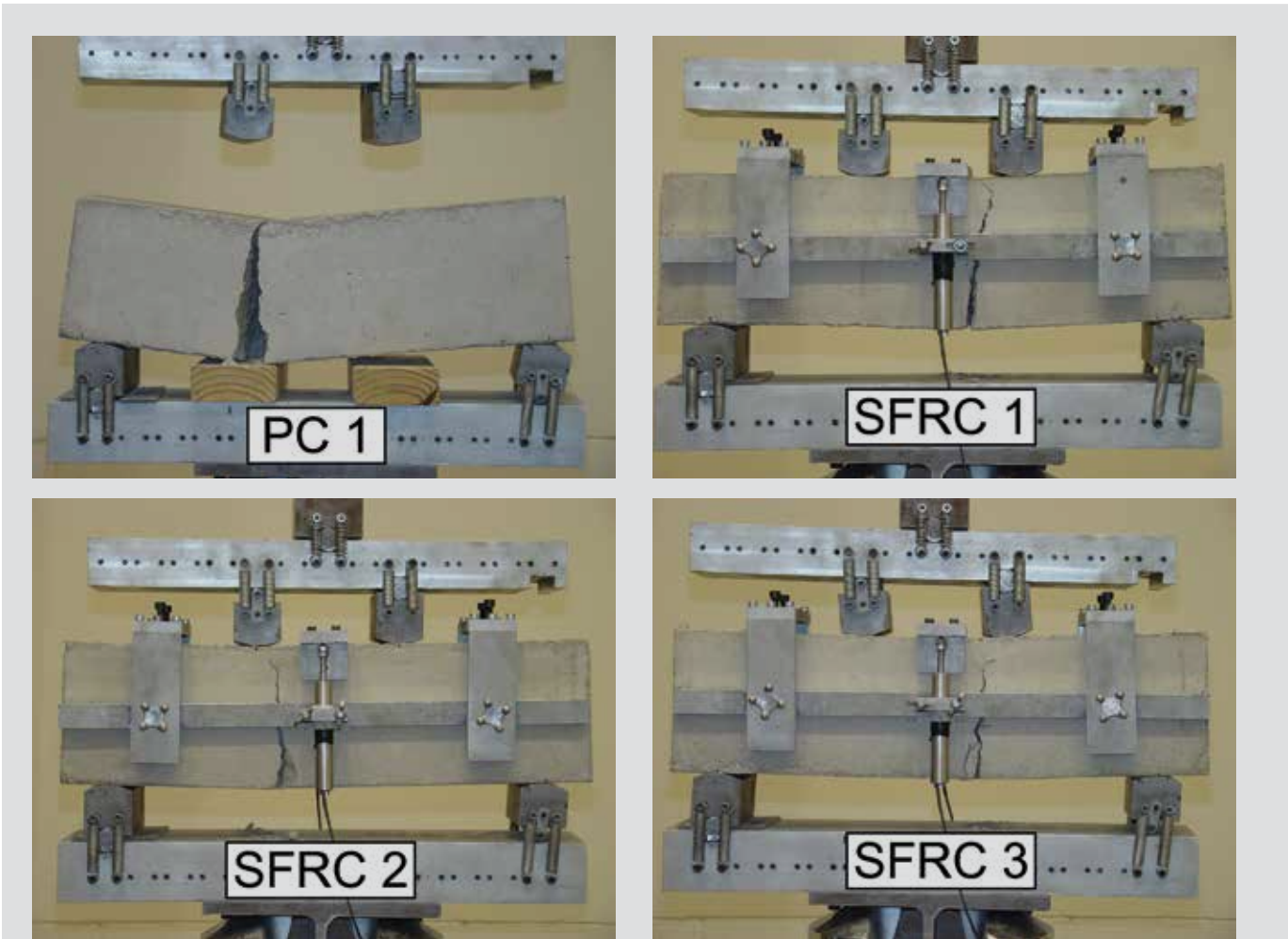


Figure 6. ASTM C78 and C1609 test specimens.

Results

Steel-fiber-reinforced concrete flexural performance

In addition to compressive strength, both plain and fiber-reinforced concrete specimens were tested according to ASTM C78²⁹ and ASTM C1609³⁰ was performed to investigate the acceptance of using steel-fiber-reinforced concrete as shear resistance (used in beams) according to ACI 318-14.² Figures 6 and 7 show the test specimens and results of the flexural performance tests, respectively. Table 3 shows the summary of the test results for one control and three fiber-reinforced concrete beams. Interestingly, the test results did not meet the ACI 318-14 requirements that the residual strengths at midspan deflections of $L/300$ and $L/150$, where L is the span length, must be greater than 90% and 75% of the first-peak strength, respectively. However, the performance of the full-scale hollow-core-slab specimens (discussed in the following sections) showed a significant increase in shear resistance due to the use of steel fibers. In one of the flexural-performance-test

specimens, the results showed highly ductile response compared with the other test specimens due to multiple cracks in the concrete, which can be expected in fiber-reinforced concrete. This high variability in the third-point bending test has been well recognized as an inherent problem due to the lack of control over the position of cracks.³¹

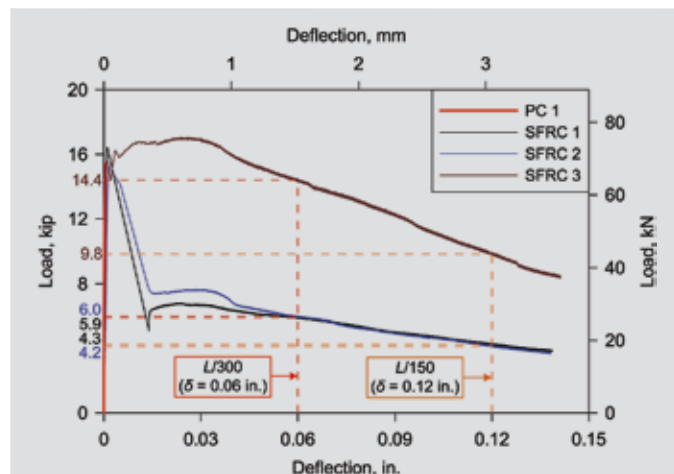


Figure 7. ASTM C78 and C1609 test results. Note: L = length; δ = deflection. 1 in. = 25.4 mm.

Table 3. ASTM C78 and C1609 test results

Specimen	Peak			L/300 ($\delta = 0.06$ in.)		L/150 ($\delta = 0.12$ in.)	
	Load, lb	Strength, psi	Deflection, in.	Load, lb	Strength, psi	Load, lb	Strength, psi
SFRC 1	16,433	1370	0.0010	5885	490	4260	355
SFRC 2	15,279	1275	0.0014	5976	500	4194	350
SFRC 3	17,055	1420	0.0270	14,418	1200	9767	815

Note: SFRC = steel-fiber-reinforced concrete; L = length; δ = deflection. 1 in. = 25.4 mm; 1 lb = 4.448 N; 1 psi = 6.895 kPa.

Hollow-core slab specimen test results

Figure 8 shows the specimens at failure. All control specimens failed in web shear. For the steel-fiber-reinforced specimens, both web- and flexure-shear crack failures were observed depending on the a/d_p ratios. Figures 9 and 10 show the corresponding shear forces versus deflections under the loading point for specimens that failed in web shear and flexure shear, respectively. Table 4 summarizes a/d_p ratios, ultimate loads, shear strengths, and failure modes. The fiber-reinforced specimens exhibited greater strength and stiffness than the control specimens. Figure 11 shows normalized shear strengths versus a/d_p ratios of the test specimens. Figure 12 shows the normalized shear strengths from this research compared with previous test results summarized by Palmer and Schultz²⁸ with hollow-core slabs of various depths.

Control specimens failed in web-shear failure mode ($a/d_p = 2.73$) For the control specimens tested in the web-shear failure mode test setup, no flexure-shear cracking was observed. The average normalized shear stress at failure ($4.7 \sqrt{f'_c}$) was 11% less than the predicted V_{cw} value ($5.3 \sqrt{f'_c}$), as calculated according to ACI 318-14, if no size effect is considered. Test results from this study conform to previous test results¹ on hollow-core slabs with depths greater than 12.5 in. (320 mm).

Steel-fiber-reinforced specimens failed in web-shear failure mode ($a/d_p = 2.0$ and 2.73) For the steel-fiber-reinforced specimens tested in the web-shear failure mode test setup, in addition to the major diagonal shear crack, some flexural cracks were observed near or at the ultimate shear force for specimens tested with a/d_p equal to 2.73. The first flexural and web-shear cracks occurred at shear stresses of approximately $7.0 \sqrt{f'_c}$ and $7.7 \sqrt{f'_c}$, respectively, for those specimens. The average ultimate shear strength of $9.5 \sqrt{f'_c}$ for the web-shear-crack failure mode was 79% higher than that of the predicted V_{cw} (ACI 318-14 Eq. [22.5.8.3.2]) calculated by using the properties of the plain concrete, and was approximately two times greater than that of the control specimens (Fig. 9).

Steel-fiber-reinforced concrete specimens failed in flexure-shear failure mode ($a/d_p = 3.45, 4.36, \text{ and } 5.09$) The first flexure-shear cracks and web-shear cracks for the specimens that failed in the flexure-shear crack mode occurred at shear stresses of approximately $4.3 \sqrt{f'_c}$ and $5.3 \sqrt{f'_c}$, respectively. The average ultimate shear strength of $7.4 \sqrt{f'_c}$ from the three specimens was approximately 90% greater than the predicted V_{ci} values (Fig. 10).

Discussion

Although the residual strength obtained from ASTM C1609³⁰ flexural testing of the steel-fiber-reinforced concrete did not meet the criteria specified by ACI 318-14, test results showed that both web- and flexure-shear strengths of deep hollow-core slabs were significantly increased with the addition of steel fibers. The efficiency of steel fibers is greater when they are used in hollow-core slabs than in beams. This is due to the thin webs of the hollow-core sections, which lead to a preferred orientation along the axis of the slabs (Fig. 13). The steel fibers used in this study had a length of 1.97 in. (50.0 mm), slightly less than the thickness of the webs of the hollow-core slabs. Everything else being equal, the tensile capacity of a fiber-reinforced cement composite is affected by the fiber orientation. Prior testing indicated that the tensile strength (and thus the shear strength) of steel-fiber-reinforced concrete with two-dimensional fiber orientation is greater than with three-dimensional fiber orientation.³² This can be accounted for by a “bridging efficiency” factor, which defines the number of fibers bridging across a crack with respect to the fiber-orientation effect. Generally, the three-dimensional random distribution leads to the lowest bridging efficiency due to loss of fiber bridging when oriented at high angles with respect to tensile stress. Krenchel³³ derived efficiency ratios for one-dimensional, two-dimensional, and three-dimensional fiber distribution, leading to numerical values of 1, 0.636, and 0.5, respectively. This translates into a composite tensile-capacity ratio (two-dimensional divided by three-dimensional) of 1.27.

Another advantage of steel fibers is their enhancement of the bond between the prestressing strands and the concrete matrix. Palmer and Schultz¹⁰ observed that web-shear strength decreased as the prestressing strands’ end slip

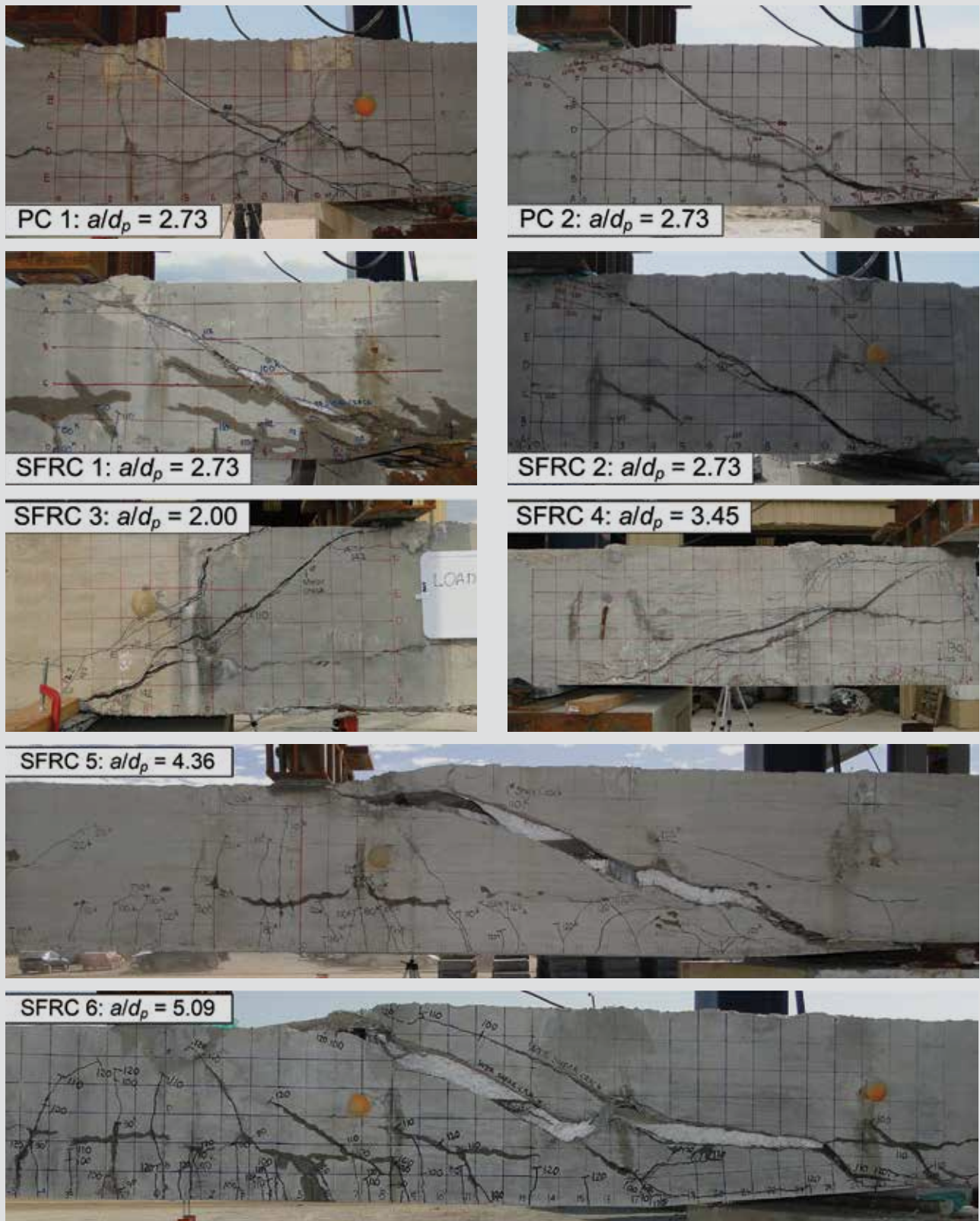


Figure 8. Specimens at failure. Note: a = shear span; d_p = distance from extreme compression fiber to centroid of prestressing steel.

increased. A previous study³⁴ showed that the confinement effect provided by fibers after concrete cracking increased friction and mechanical interlocking between prestressing strands and the concrete matrix, leading to an enhanced

bond up to three times greater than with conventional concrete matrices. As shown in **Fig. 14**, for specimens with a/d_p equal to 2.73, there was no indication of strand slip nor splitting of concrete matrix at the end of the strand in

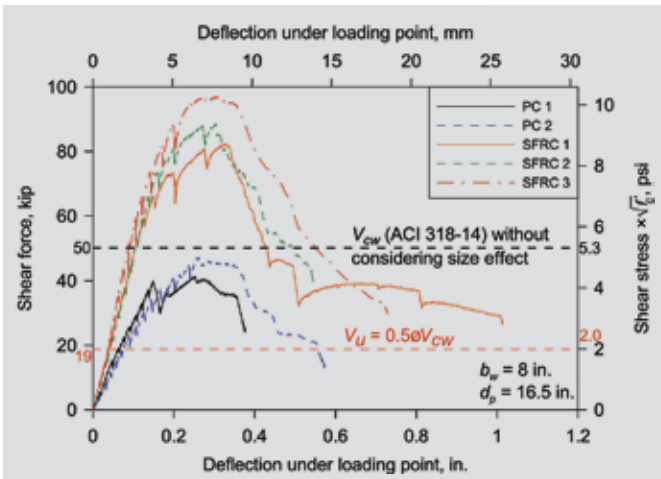


Figure 9. Shear-deflection diagram for specimens failed in web-shear failure mode. Note: b_w = web width; d_p = distance from extreme compression fiber to centroid of prestressing steel to centroid of prestressing steel; f'_c = specified compressive strength of concrete = 5120 psi; V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from high principal tensile stress in web. 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 psi = 6.895 kPa.

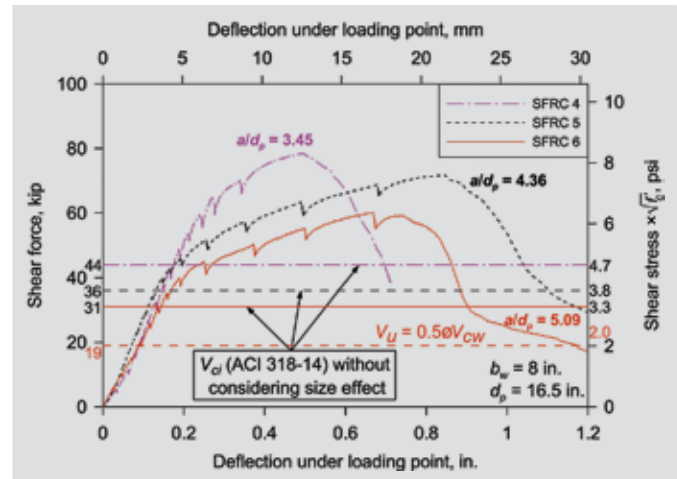


Figure 10. Shear-deflection diagram for specimens failed in flexure-shear failure mode. Note: a = shear span; b_w = web width; d_p = distance from extreme compression fiber to centroid of prestressing steel; f'_c = specified compressive strength of concrete = 5120 psi; V_{cw} = nominal shear strength provided by concrete when diagonal cracking results from high principal tensile stress in web; V_{ci} = factored shear force at section. 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 psi = 6.895 kPa.

steel-fiber-reinforced concrete specimens at failure (web-shear cracking). On the other hand, the control specimens showed obvious bond-splitting failure at ultimate testing points.

Steel-fiber-reinforced hollow-core slab shear-strength prediction

Based on the test results, ACI 318-14 Eq. (22.5.8.3.1a) and (22.5.8.3.2) are not suitable to predict the shear capacity of steel-fiber-reinforced concrete hollow-core slabs. Without considering the load-reduction factor in hollow-core slabs with depths greater than 12.5 in. (320 mm), the ACI 318-14

Eq. (22.5.8.3.2) for V_{cw} significantly underestimates the web-shear strength of steel-fiber-reinforced concrete specimens by as much as 45% (Fig. 9). In addition, ACI 318-14 Eq. (22.5.8.3.1) for V_{ci} underestimates the flexure-shear strength by as much as 100% (Fig. 10). The ultimate shear strengths of steel-fiber-reinforced concrete specimens were approximately 4.7 and 3.2 times for the web-shear and flexural-shear modes (Table 4 and Fig. 9 and 10), respectively, of the design shear force ($0.5\phi V_{cw}$) given in section 7.6.3.1 of ACI 318-14.

A simple remedy, without significantly altering the current format of the ACI 318-14 equations, to estimate the shear

Table 4. Specimen test result summary

Specimen	a/d_p	Load at first flexure-shear crack, kip	Load at first web-shear crack, kip	Ultimate load, kip	Corresponding shear force, kip	Normalized shear strength $\frac{V_u}{\sqrt{f'_c b_w d_p}}$	Governing failure mode
PC 1*	2.73	n/a	55	56	41	4.4	Web shear
PC 2*	2.73	n/a	35	65	47	5.0	Web shear
SFRC 1*	2.73	90	100	113	82	8.7	Web shear
SFRC 2*	2.73	110	90	121	88	9.4	Web shear
SFRC 3*	2.00	75	110	132	97	10.3	Web shear
SFRC 4†	3.45	80	120	146	79	8.3	Flexure shear
SFRC 5†	4.36	70	110	126	72	7.6	Flexure shear
SFRC 6†	5.09	80	100	120	60	6.4	Flexure shear

Note: a = shear span; b_w = web width; d_p = distance from extreme compression fiber to centroid of prestressing steel; f'_c = specified compressive strength of concrete = 5120 psi; PC = plain concrete; SFRC = steel-fiber-reinforced concrete; V_u = factored shear force at section. 1 kip = 4.448 kN.

* Specimen failed in web-shear failure mode.

† Specimen failed in flexure-shear failure mode.

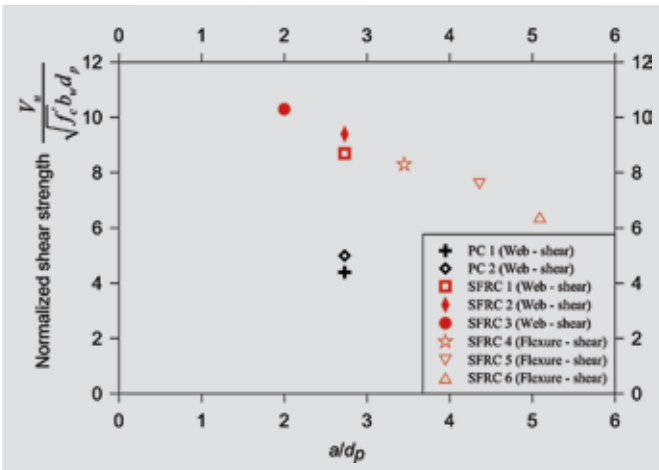


Figure 11. Normalized shear strength versus a/d_p ratio of test specimens. Note: a = shear span; b_w = web width; d_p = distance from extreme compression fiber to centroid of prestressing steel; f'_c = specified compressive strength of concrete = 5120 psi; V_u = factored shear force at section.

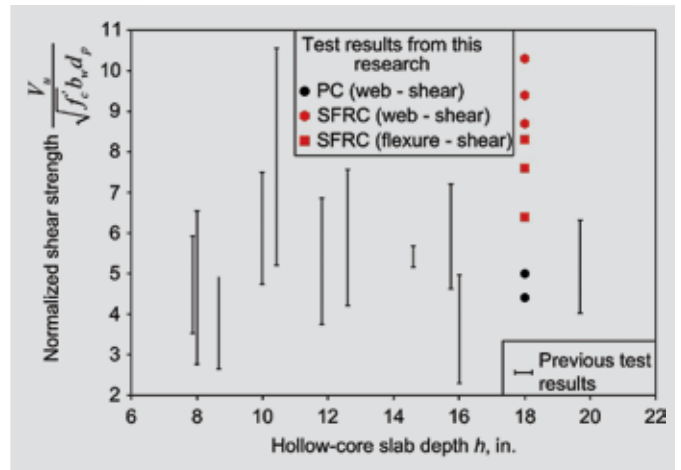


Figure 12. Normalized shear strength versus depths of hollow-core slabs. Note: b_w = web width; d_p = distance from extreme compression fiber to centroid of prestressing steel; f'_c = specified compressive strength of concrete = 5120 psi; V_u = factored shear force at section. 1 in. = 25.4 mm.

strength of steel-fiber-reinforced concrete with a minimum amount of 0.75% steel-fiber volume fraction is proposed in this study. The following equations are proposed for hollow-core slabs constructed of steel-fiber-reinforced, normalweight concrete with f'_c not exceeding 6000 psi (41 MPa) and h not greater than 18 in. (460 mm) with the minimum steel fiber specified in ACI 318-14:

$$V_{ci} = 3.0\lambda\sqrt{f'_c}b_wd_p + V_d + \frac{V_iM_{cre}}{M_{max}} \quad (1)$$

$$V_{cw} = (5.5\lambda\sqrt{f'_c} + 0.3f_{pc})b_wd_p + V_p \quad (2)$$

Equations (1) and (2) apply for hollow-core units with a total untopped depth not greater than 18 in. (460 mm), where the factored shear force at section V_u is suggested to be not greater than $0.75\phi V_c$, where V_c at a particular section is either V_{ci} or V_{cw} , whichever is smaller. ACI 318-14 limits the allowable shear force V_u to be equal to $0.5\phi V_{cw}$ for hollow-core units without shear reinforcement, though V_{ci} can be much less than V_{cw} at sections away from support.

The proposed equations are conservative estimates of the shear capacity of steel-fiber-reinforced concrete hollow-core slabs based on 80% of the calculated coefficients of test results. To be consistent with the design philosophy, the coefficients for both equations were formulated such that the calculated V_{ci} values would be less than the calculated V_{cw} values for specimens in flexure-shear-crack failure mode and vice versa. Because the first cracking strength of specimens with a 0.75% steel-fiber volume fraction is only slightly greater than that of the controls (Fig. 8), M_{cre} is calculated based on plain concrete. **Table 5** compares numerical values of both the ACI 318-14 and proposed equations. The contribution of the prestress (the second term of V_{cw})



Figure 13. Steel-fiber distribution.

was considered to be 17 kip (76 kN) (based on effective prestress of 150 ksi [1030 kN]) at the fully developed cross section for all of the specimens in this study.

Quality control of steel-fiber-reinforced concrete

Quality control has always been a concern with using fibers in concrete. The authors' suggestion is to use a standard mixing procedure (this can be established easily) and a standard material-testing method that can be done easily and gives small variability between specimens. ACI 318-14 specifies ASTM C1609³⁰ to verify steel fibers that are to be used to replace conventional shear reinforcement; however, this testing method has several disadvantages^{31,35} and the ACI 318-14 requirements are generally too stringent to meet, as shown by the ASTM C1609 tests done in this study. The double-punch test can be a good material-test method for fiber-reinforced concrete.³⁵ It was originally developed as an indirect tensile test method for plain concrete.³⁶ Compression force is applied to 6 × 12 in. (150 × 300 mm) cylinder specimens through two



Control specimen



Steel-fiber-reinforced concrete specimen

Figure 14. Prestressing strands at the end of specimens at failure. Note: $a/d_p = 2.73$; a = shear span; d_p = distance from extreme compression fiber to centroid of prestressing steel.

1 × 1.5 in. (25 × 38 mm) cylinder steel punches placed at the top and bottom surfaces of the specimen surfaces along its central axis. The equivalent tensile stress can be calculated from the applied compression force.³⁷ The double-punch test consistently exhibited low variability along the entire load-versus-deformation curve. In addition, the double-punch test can distinguish among different

fiber-reinforced concretes in terms of strain hardening or softening, ductility, residual strength, and toughness. Other major advantages of using the double-punch test are that only a few specimens are needed and only a small-capacity compression-testing machine (no need for a closed-loop servo-controlled machine), which is commonly available in laboratories, are required.

Table 5. Specimen test result versus calculated values

Specimen	a/d_p	Calculated values						Test results	
		V_{ch} kip*	V_{cw} kip†	Governing normalized shear strength $\frac{V_u}{\sqrt{f'_c b_w d_p}}$	V_{ch} kip‡	V_{cw} kip‡	Governing normalized shear strength $\frac{V_u}{\sqrt{f'_c b_w d_p}}$	Ultimate shear force, kip	Normalized shear strength $\frac{V_u}{\sqrt{f'_c b_w d_p}}$
PC 1 [§]	2.73	55	50	5.3	n/a	n/a	n/a	41	4.4
PC 2 [§]	2.73	55	50	5.3	n/a	n/a	n/a	47	5
SFRC 1 [§]	2.73	55	50	5.3	78	69	7.3	82	8.7
SFRC 2 [§]	2.73	55	50	5.3	78	69	7.3	88	9.4
SFRC 3 [§]	2.00	73	50	5.3	96	69	7.3	97	10.3
SFRC 4	3.45	44	50	4.7	67	69	7.1	79	8.3
SFRC 5	4.36	36	50	3.8	59	69	6.2	72	7.6
SFRC 6	5.09	31	50	3.3	54	69	5.7	60	6.4

Note: a = shear span; b_w = web width; d_p = distance from extreme compression fiber to centroid of prestressing steel; f'_c = specified compressive strength of concrete = 5120 psi; PC = plain concrete; SFRC = steel-fiber-reinforced concrete; V_{ch} = flexure-shear cracking strength; V_{cw} = web-shear cracking strength nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment; V_u = factored shear force at section. 1 kip = 4.448 kN.

*ACI 318-14 Eq. (22.5.8.3.1a) using plain concrete properties without considering size effect.

†ACI 318-14 Eq. (22.5.8.3.2) using plain concrete properties without considering size effect.

‡Proposed equation.

§ Specimen failed in web-shear failure mode.

|| Specimen failed in flexure-shear failure mode.

Conclusion

Shear failure in plain concrete members is brittle and consequently predisposes structures to sudden failure. One measure to protect concrete members from brittle shear failure under excessive loads is to use steel-fiber-reinforced concrete. Although this research used wet-cast fabrication of the 18 in. (460 mm) hollow-core-slab specimens, it provides a proof-of-concept pilot experiment to demonstrate the feasibility of using steel fibers to considerably enhance the shear performance of hollow-core slabs. With more reliable and greater strengths, deeper and longer spans can be used for hollow-core units, which will extend their application to a wider work scope for concrete-related engineering solutions. Although prior research used either steel or plastic fibers in dry-cast hollow-core slabs,^{10,15} further study is warranted to include fibers in hollow-core slabs using dry-cast extrusion.

This research shows that the shear strength of deep hollow-core slabs can be considerably enhanced by the addition of steel fibers. Experimental test results indicate that deep plain concrete hollow-core slabs of 18 in. (460 mm) depth showed web-shear strengths up to 17% less than those predicted by ACI 318-14 Eq. (22.5.8.3.2).

Modifications to the ACI 318 14 V_{ci} and V_{cw} equations for hollow-core slabs of up to 18 in. (460 mm) total untopped depth using steel fibers are proposed based on the results of this research. These results can also be applied to bridge box beams because their fabrication is similar to that of the specimens in this research.

The following conclusions can be drawn:

- With 0.75% steel fibers by volume, deep hollow-core slabs have higher shear capacity than those predicted by ACI 318-14 for all a/d_p ratios. Both the flexural-shear and web-shear strengths of steel-fiber-reinforced deep hollow-core slabs are approximately two times those predicted by ACI 318-14 Eq. (22.5.8.3.1a) and (22.5.8.3.2), respectively.
- The efficiency of the fiber-bridging effect is greater when fibers are used in hollow-core slabs than in beams. This is due to the thin webs of the hollow-core sections, which lead to the preferable fiber orientation along the axis of the slabs. While long fibers are preferred for this type of application, they can impose difficulty in concrete mixing if they are too long. It is suggested that fiber length should be at least 80% of the thickness of the web of the hollow-core units, with an aspect ratio between 50 and 100. The former is to ensure a preferred fiber orientation while the latter is to maintain workability of the concrete.
- Due to the fiber orientation in thin-web hollow-core slabs as well as the high shear strength of the fiber-

reinforced hollow-core slabs observed in this study, a smaller dosage of steel fiber could be sufficient. This would further increase the economy and make for easier mixing if dry-cast concrete is to be used. Further study is needed to verify this point.

- Based on test results, following the conventional V_{ci} and V_{cw} format, design equations are proposed for steel-fiber-reinforced hollow-core slabs up to 18 in. (460 mm) deep with 0.75% steel hooked fiber by volume as specified in ACI 318-14.
- The fabrication procedure and design suggestions used for the slabs in this study could be applied to box beams typically used in bridges because they are similar to deep hollow-core slabs, and placing conventional shear reinforcement is always labor intensive.
- This research was conducted by using wet-cast deep hollow-core slabs as a proof of concept. Because most current practices use dry casting to fabricate hollow-core slabs, testing needs further validation with actual steel fibers being cast in the deeper extruded specimens. The conclusion may not be necessarily applied to extruded hollow-core slabs.

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References

1. Hawkins, N. M., and S. K. Ghosh. 2006. "Shear Strength of Hollow Core Slabs." *PCI Journal* 51 (1): 110–114.
2. American Concrete Institute (ACI) Committee 318. 2014. *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary*. Farmington Hills, MI: ACI.
3. ACI Committee 318. 2011. *Building Code Requirements for Structural Concrete (ACI 318-11) and Commentary*. Farmington Hills, MI: ACI.
4. Nitterhouse Concrete Products. 2014. "Prestressed Concrete 8"x4'-0" NiCore Plank." <http://nitterhouse.com/wp-content/uploads/2013/01/8-Inch-NiCore-Plank-1-Hour-Fire-Resistance-Untopped.pdf>.

5. Becker, R. J., and D. R. Buettner. 1985. "Shear Tests of Extruded Hollow Core Slabs." *PCI Journal* 30 (2): 40–54.
6. ACI Committee 318. 1965. *Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-63)*, SP-10. pp. 78–84. Farmington Hills, MI: ACI.
7. Yang, L. 1994. "Design of Prestressed Hollow Core Slabs with Reference to Web Shear Failure." *Journal of Structural Engineering* 120 (9): 2675–2696.
8. Board of Physical Planning and Building. 1979. *Swedish regulation for concrete structures (BBK 79)*. Stockholm, Sweden.
9. ACI Committee 318. 2008. *Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary*. Farmington Hills, MI: ACI.
10. Palmer, K. D., and A. E. Schultz. 2011. "Experimental Investigation of the Web-Shear Strength of Deep Hollow-Core Units." *PCI Journal* 56 (4): 83–104.
11. Girhammar, U. A., and M. Pajari. 2008. "Tests and Analysis on Shear Strength of Composite Slabs of Hollow Core Units and Concrete Topping." *Construction and Building Materials* 22: 1708–1722.
12. Anderson, R. G. 1987. "Web Shear Strength of Prestressed Concrete Members." Technical Bulletin 85B1. Tacoma, WA: Concrete Technology Associates.
13. Parra-Montesinos, G. 2006. "Shear Strength of Beams with Deformed Steel Fibers." *Concrete International* 28 (11): 57–66.
14. ACI Committee 544. 2002. *State-of-the-Art Report on Fiber Reinforced Concrete*. Farmington Hills, MI: ACI.
15. Peaston, C., K. Elliott, and K. Paine. 1999. "Steel Fiber Reinforcement for Extruded Prestressed Hollow Core Slabs." *Structural Applications of Fiber Reinforced Concrete*, Special Publication 182-6, pp. 87–108. Farmington Hills, MI: ACI.
16. Buettner, D. R., and R. J. Becker. 1998. *Manual for the Design of Hollow Core Slabs*, Second Edition. Chicago, IL: PCI.
17. ASTM International. 2015. *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement*. ASTM A615-15. West Conshohocken, PA: ASTM International.
18. Khaja, M. N., and E. G. Sherwood. 2013. "Does the Shear Strength of Reinforced Concrete Beams and Slabs Depend Upon the Flexural Reinforcement Ratio or the Reinforcement Strain?" *Canadian Journal of Civil Engineering* 40 (11): 1068–1081.
19. Spancrete. 2015. "Spancrete Cross-Sections." http://handbook.spancrete.com/index.cfm?act=file.view&file_id=348&dld=1.
20. Structural Engineering Society of New Zealand. 2009. *Seismic Performance of Hollow Core Floor Systems – Guidelines for Design Assessment and Retrofit*. <http://www.dbh.govt.nz/consulting-on-hollow-core-floor-systems>.
21. ASTM International. 2014. *Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory*. ASTM C192-14. West Conshohocken, PA: ASTM International.
22. ASTM International. 2015. *Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens*. ASTM C39-15. West Conshohocken, PA: ASTM International.
23. ASTM International. 2011. *Standard Specification for Steel Fibers for Fiber-Reinforced Concrete*. ASTM A820-11. West Conshohocken, PA: ASTM International.
24. Joint ASCE–ACI Task Committee 426. 1973. "The Shear Strength of Reinforced Concrete Members," *Journal of the Structural Division, Proceedings of the American Society of Civil Engineers* 99(ST6): 1091–1187.
25. Wight, J. K., and J. G. MacGregor. 2012. *Reinforced Concrete – Mechanics and Design*, 6th Edition. Upper Saddle River, NH: Prentice Hall.
26. British Standards Institution. *Precast Concrete Product – Hollow Core Slabs*. BS EN 1168:2005+A3:2011. London, UK: BSI.
27. Untrauer, R. E., and R. L. Henry. 1965. "Influence of Normal Pressure on Bond Strength." In *ACI Journal Proceedings* 62 (5): 577–586.
28. Palmer, K. D., and A. E. Schultz. "Factors Affecting Web-Shear Capacity of Deep Hollow-Core Units." *PCI Journal* 55 (2): 123–146.
29. ASTM International. 2010. *Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)*. ASTM C78-10. West Conshohocken, PA: ASTM International.

30. ASTM International. 2012. *Standard Test Method for Flexural Performance of Fiber-Reinforced Concrete (Using Beam with Third-Point Loading)*. ASTM C1609-12. West Conshohocken, PA: ASTM International.
31. Chao, S. H., J. S. Cho, N. B. Karki, D. R. Sahoo, and N. Yazdani. 2011. "FRC Performance Comparison: Direct Tensile Test, Beam-Type Bending Test, and Round Panel Test." *Durability Enhancements in Concrete with Fiber Reinforcement*. ACI Special Publication 276-5. Farmington Hills, MI: ACI.
32. Chao, S. H., W. C. Liao, T. Wongtanakitcharoen, and A. E. Naaman. 2007. "Large Scale Tensile Tests of High Performance Fiber Reinforced Cement Composites." *Fifth International RILEM Workshop on High Performance Fiber Reinforced Cement Composites (HPFRCC5)*. RILEM Publications, Bagneux, France.
33. Krenchel, H. 1964. *Fibre Reinforcement: Theoretical and Practical Investigations of the Elasticity and Strength of Fibre-Reinforced Materials*. English translation. Copenhagen, Denmark: Akademisk Forlag.
34. Chao, S.-H., A. E. Naaman, and G. J. Parra-Montesinos. 2006. "Bond Behavior of Strands Embedded in Fiber Reinforced Cementitious Composites." *PCI Journal* 51 (6): 56–71.
35. Chao, S. H., N. B. Karki, J. S. Cho, and R. N. Waweru. 2011. "Use of Double Punch Test to Evaluate the Mechanical Performance of Fiber Reinforced Concrete." High Performance Fiber Reinforced Cement Composites (HPFRCC 6), International Workshop, Ann Arbor, MI.
36. Chen, W. F. 1970. "Double Punch Test for Tensile Strength of Concrete." *ACI Journal* 67 (12): 993–995.
37. Chen, W. F., and R. L. Yuan. 1980. "Tensile Strength of Concrete: Double-Punch Test." *Journal of Structural Division, Proceeding of American Society Engineers* 106 (ST8): 1673–1693.

Notation

- a = shear span
- b_w = web width
- d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement
- d_p = distance from extreme compression fiber to centroid of prestressing steel

- D = diameter
- f'_c = specified compressive strength of concrete
- f_{pc} = compressive stress in concrete at centroid of cross-section resisting externally applied loads after allowance for all prestress losses
- h = depth of member
- L = the span length
- M_{cre} = moment causing flexural cracking at section due to externally applied loads
- M_{max} = maximum factored moment at section due to externally applied loads
- P = load
- R = reaction
- V = shear force in section
- V_c = nominal shear strength provided by the concrete
- V_{ci} = flexure-shear cracking strength
- V_{cw} = web-shear cracking strength
- V_d = shear force at section due to unfactored dead load
- V_i = factored shear force at section due to externally applied loads occurring simultaneously with M_{max}
- V_p = vertical component of effective prestress force at section
- V_u = factored shear force at section
- δ = deflection
- λ = lightweight concrete modification factor
- ϕ = strength-reduction factor

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Abstract

ACI 318-14 permits no shear reinforcement in prestressed concrete hollow-core units less than 12.5 in. (320 mm) deep. However, due to the difficulty of placing shear reinforcement in hollow-core slabs, the only effective practice is to increase the depth of the section when shear demand

exceeds calculated capacity. One promising method of protecting concrete members from brittle shear failure is the use of steel-fiber-reinforced concrete. An investigation of 18 in. (460 mm) deep steel-fiber-reinforced hollow-core slabs showed that ultimate shear strengths were approximately twice those predicted by ACI 318-14. Steel-fiber-reinforced concrete is more efficient in hollow-core slabs than in typical beams due to a preferential fiber orientation in thin webs that enhance fiber bridging efficiency.

Keywords

Hollow-core, shear strength, steel-fiber-reinforced concrete.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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