Shear capacity of hollow-core slabs with concrete-filled cores

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- This paper presents the procedures and results of an experimental test program that studied the shear capacity of hollow-core slab specimens whose center cores were filled with concrete.
- The test program studied the effects of timing relative to the specimen age and the effects of enhancements to the core fill, including fiber-reinforced concrete fill, headed steel bar and welded-wire reinforcement in the core fill, and roughening the core wall before filling.
- The objective of the test program was to provide an understanding of the additional web-shear strength afforded to the hollow-core slabs by unaltered core-fill concrete, as is currently used in practice, and experimentally identify potentially promising core-fill improvement strategies.

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Precast, prestressed concrete hollow-core slabs are commonly used as floor and roof systems in concrete buildings and parking structures. The cross section of this type of precast concrete member is economical and efficient because it uses less concrete due to continuous voids (cores) along the length. With the reduced cross-sectional area at middepth, the member self-weight is significantly decreased, which only slightly affects the flexural capacity because the cross-sectional area at the extreme fibers and member depth preserve the internal moment arm. Beyond structural benefits, the voids in hollow-core slabs allow concealed routing of electrical and ventilation systems.[1]

In podium construction, one to two lower levels of precast concrete construction are frequently topped with several floors of wood or cold-formed steel construction in midsize mixed-use concrete buildings. Due to architectural floor layouts, bearing walls frequently align either with the longitudinal axis of a slab or transversely to a hollow-core slab. Both scenarios may generate a significant line load that must be supported by one or more hollow-core sections.

Background

The *PCI Manual for the Design of Hollow Core Slabs and Walls*[1] permits the flexural demand to be resisted by a width larger than a single hollow-core slab width, thus distributing the flexural demand to several members. Unlike flexure, the distribution of shear at the ends of slabs in a hollow-core floor system is more complicated. Loading is often eccentric, which results in torsional stresses across the hollow-core slab's cross section. Additionally, the shear keys between hollow-core slabs ensure equal edge displacements between adjacent members and, consequently, allow some shear and torsion forces to transfer between the hollow-core slabs. Due to the complexity of shear transfer in a hollow-core floor or roof system, the current PCI design procedure does not allow for distribution of shear demand near the ends of adjacent hollow-core slabs during the worst-case shear loading scenario in design.[1] Therefore, a single hollow-core slab must be designed to carry its entire respective shear demand.

Some hollow-core slabs in a floor system may require additional shear capacity where the demand from bearing walls or columns exceeds the hollow-core slab's capacity. Normally, the depth of the member could be increased to accommodate an increased shear demand, but floor systems are often limited in depth by architectural constraints. Alternatively, hollow-core slabs made entirely of steel-fiber-reinforced concrete (SFRC) have been shown to have an increased shear capacity while maintaining section depth but are not commonly used in practice.[2–4] Currently, most manufacturers enhance the shear capacity of hollow-core slabs by filling one or more of the cores with concrete where the shear demand exceeds the capacity of the hollow-core slab without filled cores. This increases the concrete cross-sectional area without interfering with the hollow-core slab production (that is, without affecting the mixture proportions or the prestressing profile) and offers a direct solution to selectively increase the web-shear capacity.

It is difficult for designers to reliably quantify the increase in shear capacity afforded by the increased cross-sectional area of added core-fill concrete. The American Concrete Institute's (ACI's) *Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)*[5] offers Eq. (22.5.5.1) and (22.5.8.3.2) to calculate unreinforced concrete shear capacity and prestressed concrete web-shear capacity, respectively. At a minimum, the added core-fill concrete should provide additional shear capacity calculated as if the core fill were an independent, unreinforced piece of concrete.

In some scenarios, a hollow-core slab is fabricated months before the specific building occupancy or use is defined. In this case, if additional web-shear capacity is required due to an increase in loading, the fully cured hollow-core section is cut open and fresh concrete is used to the fill core(s). Because this new core-fill concrete is placed within a cured specimen, the system is not conducive to composite action. In this case, the core-fill concrete may act independently and have the shear capacity of unreinforced concrete. In a more common scenario, cores are filled at the time of hollow-core fabrication. In this case, hollow-core slabs are extruded on a prestressing bed and specific cores are filled within the fresh concrete before transferring the strand stress to the hollow-core section. The shear capacity of the core-fill concrete can be calculated assuming prestressed concrete as a maximum because the extruded concrete for the hollow-core slab is in contact with the core-fill concrete and allowed to cure before the strands are released.

There has been insufficient research to quantify the additional shear capacity provided by the core-fill concrete. Palmer and Schultz[6] demonstrated that the overall shear capacity of a hollow-core slab can be increased by filling a core with concrete. In addition, little research has been conducted to explore innovative ways to further enhance the shear capacity of concrete-filled cores.

Research objectives

This research program experimentally investigated the added web-shear capacity of hollow-core sections with concrete-filled cores and quantified the core-fill shear strength contribution.[7] The effects of core-fill timing relative to slab age and several core-fill strength enhancement strategies were explored, including SFRC core fill, roughening the slab core walls before core-fill placement, introducing a longitudinal headed steel bar into the core-fill material, and placing vertically oriented welded-wire reinforcement (WWR) in the core fill. The objective of the project was to provide an understanding of the additional web-shear strength afforded to the hollow-core slabs by the unaltered core-fill concrete, as is currently used in practice, and experimentally identify potentially promising core-fill improvement strategies.

This project also sought to determine whether the core-fill concrete behaved like unreinforced concrete or like fully prestressed concrete. Two scenarios exist where the cores may be filled to increase the web-shear capacity:

- Core-fill concrete is placed within a cured specimen that is fabricated months before the specific building occupancy is defined.
- Core-fill concrete is placed adjacent to fresh hollow-core-slab concrete at the time of fabrication before transferring the strand stress to the section.

In the first scenario, the core-fill concrete may act as unreinforced concrete due to a lack of composite action, whereas in the second scenario the core-fill concrete may act as fully prestressed concrete due to the fabrication sequence.

Experimental program

The experimental program for this project included manufacturing and testing both ends of eight hollow-core slab specimens. Of the eight specimens, one served as a control specimen, or baseline, for the testing program, and the cores remained unfilled. The remaining seven specimens were modified by adding concrete filling to the center void using various methods. These methods included cold joint core fill, typical immediate fill, typical immediate fill aged 209 days before testing, fiber-reinforced fill, core wall surface roughening, headed steel bar in core fill, and WWR in core fill. The specimens and associated

Table 1. Variables investigated in the experimental program				
Hollow-core slab specimen and associated core modification	**Number of slabs**	**Number of tests***	**Core-fill concrete type**	**Slab age when core was filled**
Empty cores	1	2	n/a	n/a
Cold-joint core fill	1	2	Typical	Mature†
Typical immediate fill	1	2	Typical	Early‡
209-day typical fill§	1	2	Typical	Early
Fiber-reinforced fill	1	2	Steel-fiber reinforced	Early
Core wall surface roughening	1	2	Typical	Early
Headed steel bar in core fill	1	2	Typical	Early
Welded-wire reinforcement in core fill	1	2	Typical	Early

Note: n/a = not applicable.

* Each end of the hollow-core slab specimen was tested.

† Approximately three weeks following slab extrusion.

‡ Approximately one hour following slab extrusion (typical manufacturing process).

§ Immediately core-filled slab allowed to cure for 209 days prior to testing.

core modifications are summarized in **Table 1** and in the subsequent sections.

Design specifications of the hollow-core slabs

The hollow-core slabs were designed to fail in web-shear under a single, applied point load. Adequate moment capacity was achieved by selecting an appropriate slab depth and prestressing strand configuration. The shear span–to–depth ratio a/d_p was considered concurrently with the preliminary shear capacity for flexural requirements. Based on previous shear research, a minimum a/d_p value of 2.5 notably eliminated the effects of arching action.[8] Because the hollow-core slabs were not expected to have flexural cracking before web-shear failure, the shear span–to–depth ratio considering the entire section depth a/h was selected to be 2.5. This resulted in an a/d_p value of 3.0.

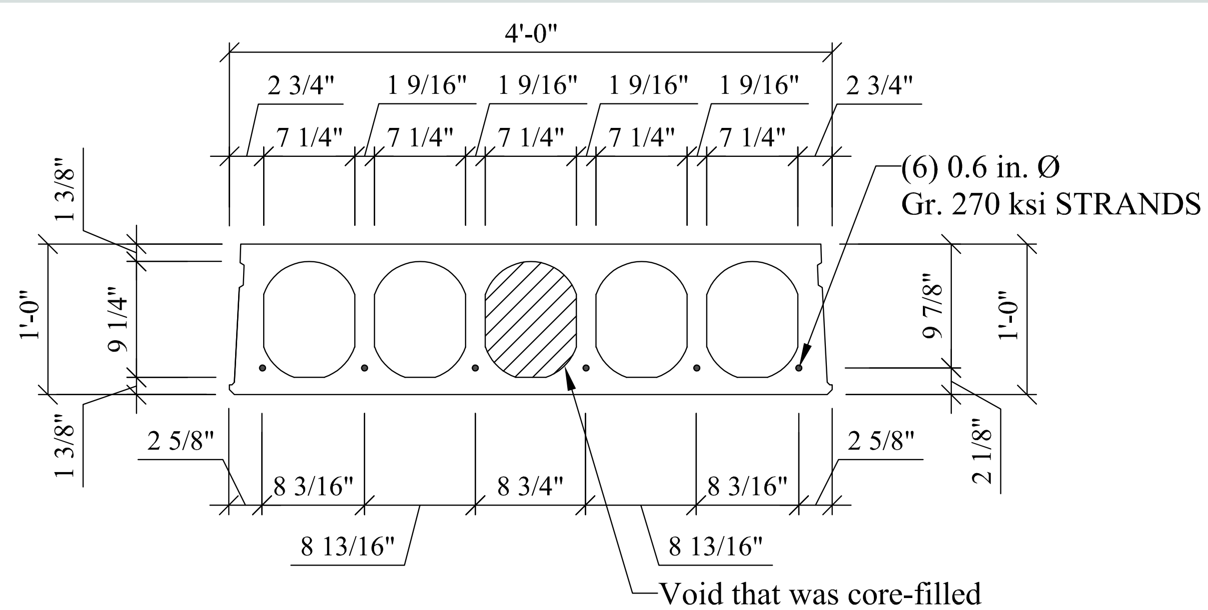


Figure 1. Cross-sectional geometry of the 12 in. hollow-core slabs. Note: ø = diameter. 1" = 1 in. = 25.4 mm; 1' = 1 ft = 0.35 m; 1 ksi = 6.895 MPa.

Hollow-core slabs measuring 12 in. deep × 4 ft wide × 20 ft long (304.8 mm × 1.2 m × 6.1 m) were used to meet the design specifications. All of the slabs had six 0.6 in. (15.2 mm) diameter strands stressed to $0.7f_{pu}$, where f_{pu} is the prestressing steel tensile strength, at a depth d_p of 9.875 in. (250.8 mm). The hollow-core cross sections had five voids across the width. Of the modified specimens, each slab had core fill in the center void for a length of 4 ft (1.2 m) measured from each end. The 4 ft core fill extended beyond the point of loading to ensure that its full shear contribution was realized. **Figure 1** shows the cross-sectional geometry of the hollow-core. The normalweight concrete mixture proportions used by the manufacturer in typical hollow-core production were employed for all of the specimens. This concrete was designed with a water-cement ratio of 0.35 and developed to have no slump for use in an extruding machine. The same concrete mixture used in the hollow-core slab was deployed in all of the filled cores; the core-fill concrete had additional high-range water-reducing admixtures (HRWRAs) and water to increase the workability.

Hardened properties of both the core-fill concrete and hollow-core-slab concrete were characterized at 28 days and on each test day. Three concrete compressive tests were performed on the hollow-core-slab and core-fill concrete cylinders per ASTM C39.[9] Similarly, three split cylinder tests were conducted according to ASTM C496[10] using hollow-core-slab and core-fill concrete cylinders at 28 days. The relationship between split cylinder tensile strength and compressive strength was determined to be $9.0\sqrt{f_c'}$ for the hollow-core-slab concrete and $5.8\sqrt{f_c'}$ for the core-fill concrete at 28 days. The tensile strength of SFRC can be very different from that of unreinforced concrete, so additional compressive and split cylinder tests were performed on the core-fill SFRC. The relationship between the split cylinder tensile strength and compressive strength of the SFRC core fill was determined to be $9.2\sqrt{f_c'}$.

Variables investigated with core-fill timing specimens

Four hollow-core slabs were used to study the effects of core-fill timing on shear performance, including a baseline hollow-core slab. Both the hollow-core slab age at the time of core filling and the age of the core-filled hollow-core slab were studied. An unaltered, unfilled hollow-core slab was tested and served as a baseline for all of the tests. The producer's standard core-fill practice was used for two identical slabs to represent real-world manufacturing techniques of hollow-core slabs filled during fabrication. In this standard practice, the top flange of the core to be filled was first collapsed by the heel of a worker's boot. This broken flange concrete was collected by hand and gathered at the end of the intended core fill length to create a plug of concrete, which prevented excess core fill from moving longitudinally along the void. A portion of the no-slump concrete mixture used for slab extrusion was placed in a small mixer with additional water and HRWRAs to prepare the core-fill material. The core-fill concrete was placed in the desired core, and the top of the hollow-core slab was finished by hand.

One of these standard core-filled hollow-core slabs was tested approximately two months after fabrication, and the other was tested 209 days (approximately seven months) after

Figure 2. Saw-cut residual dust coating the interior of the core before cleaning (left) and core cleaned with a grinder and wire brush attachment (right).

fabrication. Results from the hollow-core slab tested 209 days after fabrication were used to address the potential long-term hollow-core-slab and core-fill concrete shrinkage effects because all of the other specimens in the study were tested within approximately two months of fabrication. Concrete shrinkage may cause the core-fill concrete to contract inward and the hollow-core-slab concrete to contract away from the void, which means that the interlock between the core-fill and hollow-core-slab concretes could be compromised over time.

To compare the effects of hollow-core slab age at the time of core filling, a hollow-core slab was cured for approximately three weeks before core filling. This cold-joint hollow-core slab captured the core-fill behavior when the hollow-core slab core filling occurs after curing and strand release, which is the less common core-filling practice. This case is very different from the typical immediate-fill case because a cold joint between the core-fill and hollow-core-slab concrete exists. There is often concrete dust left in the cores from saw cutting each hollow-core slab to length (**Fig. 2**). This residual dust is powdery when dry and coats the interior core surface for several feet in each direction from the cut at the end of a hollow-core slab. Figure 2 shows uncleaned and cleaned cores. Consequently, there is likely less composite action between the core-fill and hollow-core-slab concretes when the core-fill concrete is added to an already-cured hollow-core slab. The first four entires in Table 1 are the four core-fill timing specimens.

Variables investigated with core-fill enhancement specimens

Four hollow-core slabs with novel core-fill enhancements were included to investigate potentially promising strategies to improve shear capacity. These enhancements sought to improve the shear capacity of the hollow-core slab with core-fill concrete, to promote additional bond between the core-fill concrete and the hollow-core-slab concrete, or both. A performance baseline was established via the hollow-core slab with cores filled at fabrication (typical immediate fill) and tested shortly thereafter. Each of the novel core-fill enhancements altered the fill of a typically filled core, so comparisons between each enhanced specimen and the unaltered typical-fill specimen provided an indication of the potential improvement due to the novel technique.

One hollow-core slab in the study employed steel fibers at a 0.5% volume fraction in the core-fill concrete to formulate SFRC. The 0.5% volume fraction indicated the ratio of steel-fiber volume to concrete volume and was selected based on the literature to maximize the steel-fiber contribution to the strength while maintaining workability.[2–4] Results from previous tests by others of hollow-core slabs fabricated entirely with SFRC showed improved shear capacity, but two secondary benefits attributed to the fibers also likely contributed to the SFRC hollow-core slab strength in those tests. First, the prestressing bond was shown to be improved when the concrete included fibers.[11] Second, the steel-fiber orientation along the axis of the hollow-core slabs due to their thin webs was ideal.[4] Both of these added benefits cannot be realized when only the core-fill material is SFRC. Therefore, although improvements were shown in hollow-core slabs made entirely from SFRC, somewhat smaller improvements were expected in the tests of the SFRC core-fill specimens.

Another enhancement technique was to roughen the center core wall surfaces with vertically oriented grooves using a small handheld garden rake, which provided an indentation of about 0.25 in. (6.4 mm) spaced at about 1 in. (25.4 mm) along the length of the web walls. These measurements were obtained using a handheld ruler to quantify the enhancement; this enhancement was not repeated enough to create a quality control plan. The goal of this enhancement was to improve the composite action between the core-fill and hollow-core-slab concretes and potentially transfer more prestressing force to the core-fill concrete. For a core-filled specimen to act as a monolithic and fully prestressed section, the core-fill concrete must act compositely with the hollow-core-slab concrete. In a specimen where the core is filled immediately after extrusion, the core-fill concrete is in contact with the hollow-core-slab concrete before either has set, which may result in some bonding.

In another specimen, a 4 ft (1.22 m) long no. 6 (19M) reinforcing bar with 0.25 × 2 × 2 in. (6.4 × 50.8 × 50.8 mm) steel plates welded to each end was placed longitudinally at the bottom of the center void before core filling for ease. The bar was not placed near the neutral axis of the specimen. The end plates provided anchorage for the relatively short bar length. This core-fill modification was intended to maintain the axial precompression in two ways. First, the addition of a headed steel bar in the core fill may transfer some of the applied load carried by the strands to the bar, thus reducing the prestressing strand demand and slip. Second, the headed steel bar was intended to maintain any axial precompression potentially present in the core-fill concrete at the time of prestress transfer. The headed steel bar in the core fill could improve the web-shear capacity of the section by maintaining a higher precompression force and reducing strand slip.

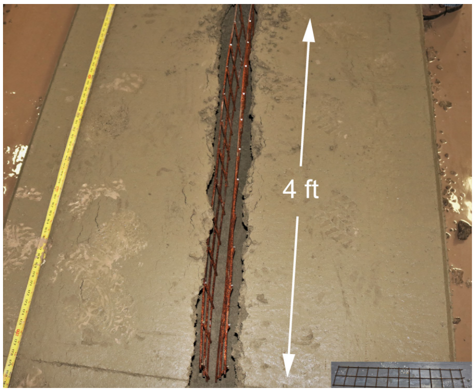
Vertically oriented WWR was included in the core fill of a hollow-core slab as another method to provide reinforcement in the core-fill concrete. In typical reinforced concrete design, transverse shear reinforcement is used as the default way to provide additional shear capacity where necessary. Unfortunately, inclusion of transverse shear reinforcement is not possible in common hollow-core fabrication methods.[1] Furthermore, ACI 318[5] section 7.6.3.1 restricts the maximum shear demand applied to hollow-core slabs deeper than 12.5 in. (317.5 mm) with no transverse reinforcement by limiting the factored shear demand to half the calculated shear strength.

For the WWR specimen, the filled core had two 4 ft × 8 in. (1.2 m × 203.2 mm) pieces of four-gauge WWR with a 4 × 4 in. (101.6 × 101.6 mm) grid placed vertically in the ends of the core. The WWR could augment web-shear capacity in two ways. First, the vertical bars in the mesh provided regularly

Steel-fiber-reinforced concrete specimen

Core-wall-surface-roughening specimen

Headed-steel-bar specimen



Welded-wire-reinforcement specimen

Figure 3. Hollow-core specimens with the top flange removed over the center void to facilitate core filling near the end of the slab (indicated by the line marked in the concrete) before transferring the prestress force to the concrete via cutting the specimens to length. Note: 1 ft = 0.3048 m.

spaced transverse reinforcement that was anchored by the longitudinal bars. Second, the longitudinal bars in the WWR potentially maintained any axial precompression in the core fill and possibly alleviated slip in the prestressing strands, similar to the potential behavior of the specimen with the headed steel bar. **Figure 3** shows the four core-fill enhancement specimens.

Laboratory test setup

Each hollow-core slab was tested twice (once at each end) to gather more data from the specimens included in this program. During the test on the first hollow-core slab end, the opposite end was overhung beyond the support farthest from the applied load for protection. The distance from the support to the end of the overhung hollow-core slab exceeded the distance between the applied point load and support nearest the applied load. This ensured that virgin material was being tested when the hollow-core slab was rotated and tested a second time. The simply supported span length between the pin and roller supports was 16 ft (4.9 m). The point load was applied with a single actuator 30 in. (762 mm) from the centerline of the nearest support. The point load was transversely distributed over the 4 ft (1.2 m) specimen width. **Figure 4** shows the testing arrangement.

Instrumentation and loading

The applied load and member self-weight were proportioned to determine the applied shear force at the critical section using statics based on the distance between the point load and each support, which was 2.5 ft (0.8 m) to the nearest support and 13.5 ft (4.1 m) to the support farthest from the load. The critical section was $h/2$ from the face of the support, which was 6 in. (152.4 mm). The actuator was equipped with a 100 kip (444.8 kN) load cell. Six linear variable displacement transducers (LVDTs) were placed around the test setup to measure hollow-core slab motion during testing. The relative difference in deflections across the width or along the length of the hollow-core slab was used to identify any undesirable twisting motion of the specimen. Hollow-core slab twisting could be indicative of improper load distribution or bearing contact because the hollow-core slab had a sym-

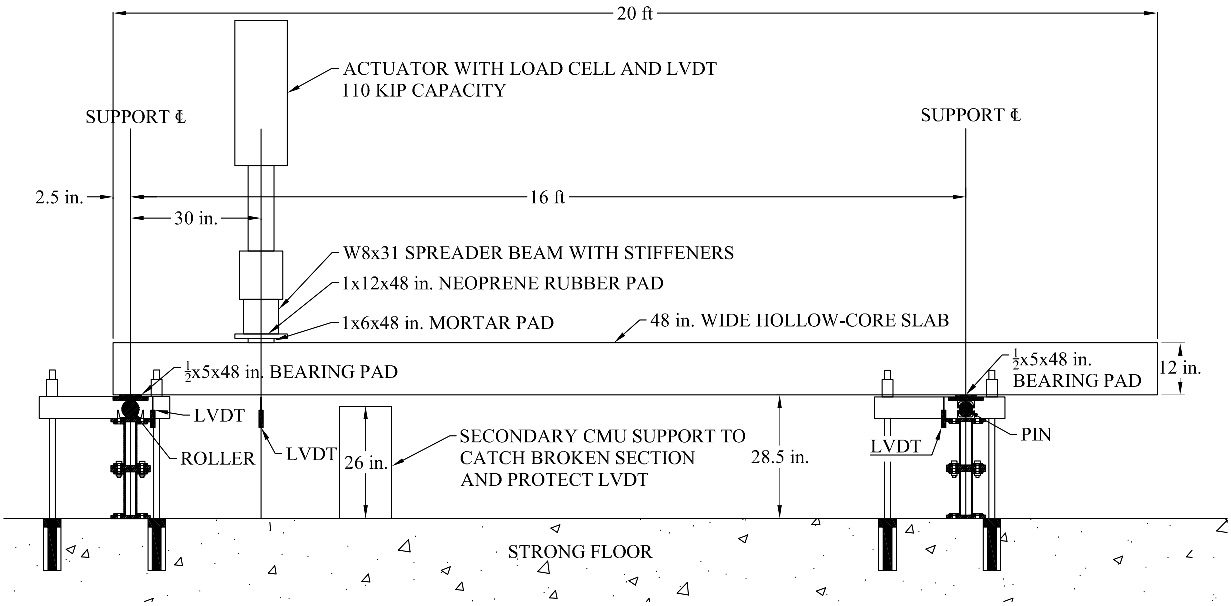


Figure 4. Test setup shown in an elevation view. Note: CMU = concrete masonry unit; LVDT = linear variable displacement transducer. 1 in. = 25.4 mm; 1 ft = 0.3048 m; 1 kip = 4.448 kN.

metrical cross section. Hollow-core slab twisting generates additional torsional stresses, thus reducing the capacity to resist vertical shear. If these torsional stresses were notable, the results of a test may not have been suitable for comparison with web-shear capacity calculated with design codes and obtained from other specimens. Consequently, two LVDTs were attached to the hollow-core slab on either side at the load point and each support. No significant differential displacements were observed in the LVDT data collected during testing, indicating proper loading and bearing. The actuator was advanced in a displacement-controlled mode at 0.008 in./sec (0.203 mm/sec) initially and at 0.004 in./sec (0.102 mm/sec) after reaching an applied load of 10 kip (44.5 kN) until web-shear failure.

Crack face documentation

A three-dimensional (3-D) scanner and accompanying data collection and postprocessing software were used to capture a 3-D scan of the failure plane from one side of each core-filled

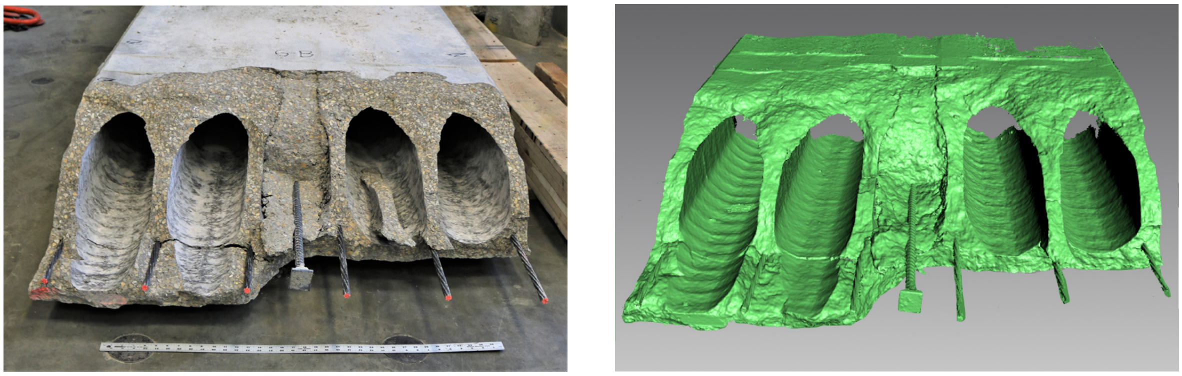


Figure 5. Photograph of the cracked face of a test specimen (left) and scanned three-dimensional surface model of the same cracked face (right).

Figure 6. Typical web-shear crack in a hollow-core slab.

specimen after the testing and dismantling of the broken hollow-core slab. The resulting high-resolution 3-D point cloud was processed to generate a realistic surface model (**Fig. 5**). Using the model, the crack angle of each hollow-core-slab web and core fill at the critical depth was determined.

Shear test results

Behavior at failure

Sudden, brittle web-shear cracking defined failure (**Fig. 6**) and was accompanied by a sudden drop of applied load and abrupt deflection. The crack pattern, peak applied load (that is, largest nominal dead load and live load), strand slip, and overall hollow-core-slab behavior were inspected and documented for each specimen after web-shear failure (**Table 2**). Additional load was applied to the second side of each specimen after postfailure inspections were completed. This second phase of loading was performed to widen the web-shear cracks and allowed dismantling of the broken hollow-core slab end for further inspection and 3-D scanning.

No shear or flexural cracking was observed in any of the test specimens before web-shear cracking. Rather, a web-shear crack appeared across each of the six webs at the same time upon failure. It was not possible to determine the precise location of the initial web-shear cracking due to the sudden failure, but the diagonal crack propagated between the load point and the prestressing strand. At the strand depth, the crack direction shifted and extended longitudinally along the prestressing strand to the member end at the nearest support. In addition, transverse cracking of the bottom flange near the support and of the top flange near the load point appeared during some of the tests.

In each core-filled test specimen, the core-fill concrete exhibited a web-shear crack near the adjacent web-shear cracks in the hollow-core slab. This indicated that the core-fill concrete was engaged and carried some of the shear force, as intended. Following testing and dismantling of the broken section, it was apparent that the core-fill concrete slipped and was not composite with both adjacent webs of the hollow-core-slab concrete in the cold-joint-core-filled, 209-day-typical-filled, SFRC-core-filled, and headed-steel-bar specimens. In the cold-joint core-fill hollow-core slab, a powdery dust was found on the outer surface of the hardened core-fill concrete. This material was likely residual concrete dust from hollow-core slab saw cutting that was not removed before core filling. In the three other core-filled test specimens, the core-fill concrete remained composite with one or both of the adjacent hollow-core-slab webs.

Shear capacity predictions and data interpretation

ACI 318[5] shear capacity predictions consider the average shear stress across the entire cross-sectional area of a mem-

Table 2. Experimental program average test results and predicted strength results

Hollow-core slab specimen and associated core modification	Slab concrete compressive strength f'_c, psi	Core-fill concrete compressive strength f'_{cf}, psi	Average prestressing strand slip, in.	Average transfer length,* in.	Compressive stress in concrete f_{pc}, psi	Predicted slab web-shear strength, kip	Peak shear force, kip
Empty cores	12,920	n/a	0.07	50	133	47.4	58.8
Cold-joint core fill	12,940	7290	0.05	42	136	47.4	63.1
Typical immediate fill	13,210	8070	0.07	50	110	47.9	71.0
209-day typical fill	13,640	9040	0.08	63	85	48.6	62.4
Fiber-reinforced fill	11,940	7300	0.07	51	104	45.8	56.4
Core wall surface roughening	10,580	6050	0.06	41	139	43.4	64.5
Headed steel bar in core fill	11,350	7100	0.07	56	96	44.8	59.5
Welded-wire reinforcement in core fill	11,430	7290	0.07	50	104	44.9	66.8

Note: d_b = nominal diameter of prestressing strand; n/a = not applicable. 1 in. = 25.4 mm; 1 kip = 4.448 kN; 1 psi = 6.895 kPa; 1 ksi = 6.895 MPa.
*Transfer length was calculated using Eq. (4). For comparison, $50d_b$ = 30 in. for 0.6 in. diameter strands.

ber. Based on mechanics, the shear stress is parabolically distributed through the depth of a section with the maximum stress occurring at middepth.[12] This distribution is especially important for hollow-core slab sections because the narrow web widths generally occur at or near the middepth of a section.

ACI 318[5] defines the nominal shear capacity V_n of a section as the sum of the concrete shear capacity V_c and transverse shear reinforcement capacity V_s in ACI 318 Eq. (22.5.1.1) and the *PCI Design Handbook: Precast and Prestressed Concrete*[13] section 5.3, as presented in Eq. (1). The transverse steel contribution to shear strength in Eq. (1) was taken as zero for extruded hollow-core slabs because transverse shear reinforcement was not present. To accurately compare shear capacity predictions with experimental test results, the shear strength-reduction factor ϕ was taken as 1.0 and all load factors were set equal to 1.0.

$$V_n = V_c + V_s \tag{1}$$

In this study, the core fill was estimated to have the shear strength of an individual unreinforced piece of concrete, at a minimum. Unreinforced concrete shear strength V_c was predicted using ACI 3185 Eq. (22.5.5.1) and the *PCI Design Handbook*[13] Eq. (5-22), as shown in Eq. (2). However, Eq. (2) was modified to specifically calculate the unreinforced concrete shear strength of the core fill only by replacing f'_c with the compressive strength of the core-fill concrete f'_{cf} and $b_w d$ with the cross-sectional area of the filled core A_{cf}.

$$V_c = 2\sqrt{f'_c}b_w d \tag{2}$$

where

b_w = width of web

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement

The shear capacity at the ends of prestressed concrete hollow-core slabs typically is controlled by web-shear behavior, which is defined by ACI 318[5] Eq. (22.5.8.3.2) and the *PCI Design Handbook*[13] Eq. (5-28), as shown in Eq. (3). The web-shear equation considers the nominal shear stress in a section, not the maximum shear stress. Although this makes Eq. (3) inherently conservative, it may better characterize a wider variety of prestressed concrete members. In this study, the prestressed concrete hollow-core slabs were expected to have the web-shear strength predicted using Eq. (3), at a minimum. Only straight prestressing profiles are employed in extruded hollow-core-slab sections. Therefore, the vertical component of prestressing force V_p was taken as zero in Eq. (3). Furthermore, Eq. (3) was modified to specifically calculate the prestressed concrete shear strength of the core fill only by replacing f'_c with the compressive strength of the core-fill concrete f'_{cf} and $b_w d_p$ with the cross-sectional area of the filled core A_{cf}.

$$V_{cw} = \left(3.5\sqrt{f'_c} + 0.3 f_{pc}\right) b_w d_p + V_p \tag{3}$$

where

V_{cw} = nominal shear capacity provided by concrete where diagonal cracking results from high principal tensile stress in web

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

For prestressed concrete members, the axial precompression increases from 0 ksi (0 MPa) to its maximum within the transfer length and consequently the web-shear capacity similarly increases over this length. For the calculation of shear in prestressed concrete members, ACI 318[5] section 22.5.9.1 specifies that $50d_b$, where d_b is the prestressing strand diameter, be used as the transfer length of prestressing reinforcement. Although this simplified equation is widely used in concrete design, it may not capture the true strand bond behavior because only the strand diameter and stress are considered.

The ACI 318[5] transfer-length equation was developed and validated using test results from sections fabricated with wet-cast concrete and may not provide accurate results if extrapolated to no-slump concrete, as described in ACI 318 section R25.4.8. This may affect the transfer length in extruded hollow-core slabs, which are commonly fabricated with no-slump concrete. Palmer and Schultz[6] found that the transfer length l_{tr} could be more accurately determined for extruded hollow-core slabs when strand-slip measurements are known. Their equation uses the difference between initial and peak shear load strand slip. This relationship (Eq. [4]) was used for the predicted shear capacities in this paper because strand slip was measured after each web-shear failure.

$$l_{tr} = 5\delta_{es}\frac{E_p}{f_{pi}} \tag{4}$$

where

δ_{es} = measured end slip of strand

E_p = modulus of elasticity of prestressing steel

f_{pi} = prestressing steel stress immediately before transfer

The concrete compressive strength f'_c of both the hollow-core-slab and core-fill material varied because the specimens were fabricated on two separate days and tested on different days and some of the duplicate tests were performed at different times. As a result, the peak applied shear forces cannot be directly compared. Consequently, the experimentally measured shear capacity (that is, peak applied live load plus dead load shear force at the time of web-shear cracking) was compared with the respective predicted capacity to assess the performance of each test. This comparison normalizes the differences in concrete strength between each of the individual hollow-core slabs and allows realistic comparison between specimens.

This project sought to determine whether the core-fill concrete behaved like unreinforced concrete or like fully prestressed concrete. Oftentimes, the shear strength of concrete is quantified by a coefficient multiplied by the square root of the concrete compressive strength and appropriate cross-sectional area. For example, unreinforced concrete shear strength uses a coefficient of 2 (Eq. [2]). The shear strength coefficient of the core-fill concrete for each specimen was calculated and compared directly with results from the other tests. For example, the cold-joint core-filled specimen had an average peak applied load of 64.1 kip (285.1 kN). With a hollow-core-slab concrete compressive strength of 12,940 psi (89.2 MPa) and transfer length of 42 in. (1067 mm), the predicted web-shear capacity of the hollow-core-slab concrete was 47.4 kip (210.8 kN). By subtracting the predicted hollow-core slab shear capacity from the peak shear force, 16.7 kip (74.3 kN) of shear capacity remained and was attributed to the core-fill concrete. The coefficient for the core-fill concrete shear capacity was calculated to be 3.2 by dividing the remaining shear capacity by both the square root of the core-fill concrete strength and the core-fill concrete area of 58 in.^{2} (37,419 mm^2) (**Fig. 7**).

To formulate an equation for the core-fill concrete strength that was in the form of a coefficient times the square root of the core-fill concrete strength, the hollow-core-slab concrete contribution needed to be isolated and excluded. Because all of the hollow-core slabs were of identical cross section and prestressing configuration, each specimen should have had no difference in predicted web-shear capacity when normalized for hollow-core-slab concrete strength. Therefore, the predicted hollow-core-slab concrete shear strength was subtracted from the peak applied shear force for each test and the remaining force was divided by the square root of the core-fill concrete compressive strength (Table 2). Figure 7 shows a horizontal reference line at a coefficient of 2.0 to designate unreinforced core-fill behavior.

To identify the threshold for fully prestressed concrete behavior, the terms in parentheses in Eq. (3) were modified to determine an equivalent coefficient multiplied by the square root of the core-fill concrete strength (Eq. [5]). The axial precompression and core-fill concrete compressive strength varied between specimens (Table 2), so the average respective values were used to determine the approximate coefficient of 3.9 in Eq. (5), which is specific to the core-fill concrete (not the hollow-core slab). This value is shown as a horizontal line in Figure 7 and designates the threshold of prestressed core-fill concrete behavior.

$$3.5 + \frac{0.3 f_{pc}}{\sqrt{f'_{cf}}} = 3.9 \tag{5}$$

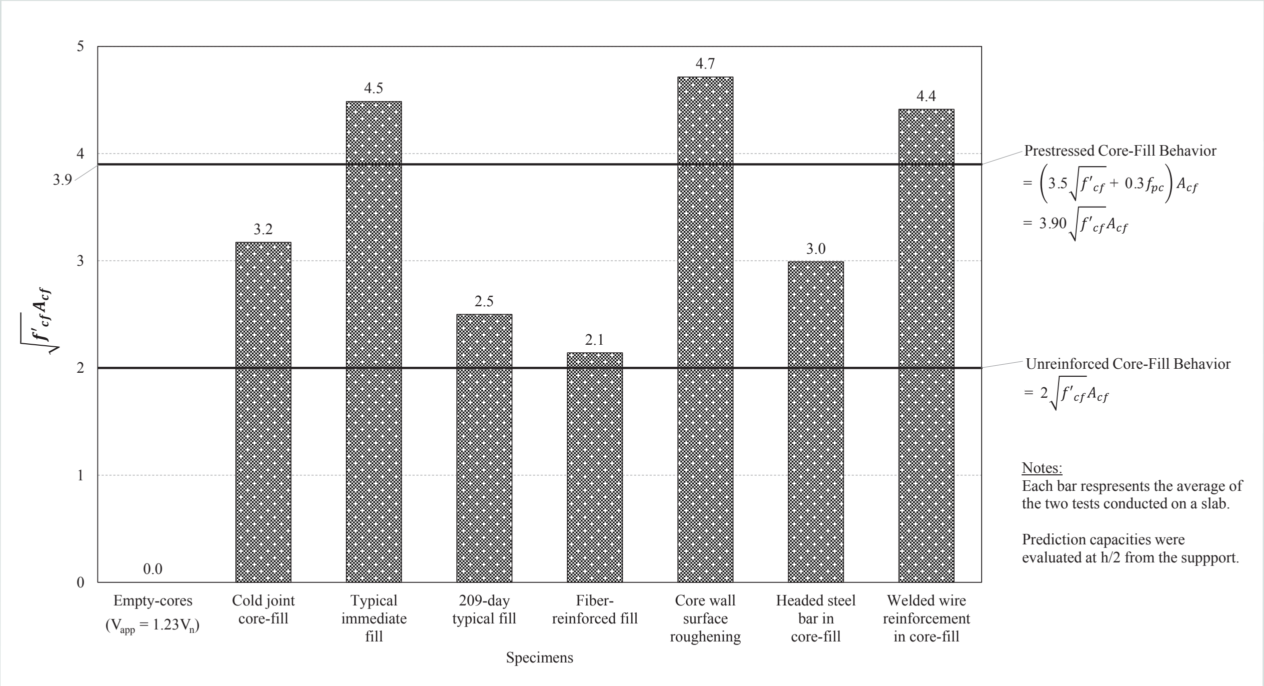


Figure 7. Coefficient of the square root of the core-fill concrete compressive strength multiplied by the core-fill area in excess of the predicted hollow-core-slab web-shear strength. Note: A_{cf} = core-fill concrete area; f'_{cf} = nominal compressive strength of core-fill concrete; f_{pc} = compressive stress in concrete, after allowance for all prestress losses, at centroid of cross section resisting externally applied loads; V_{app} = maximum shear demand applied at failure; V_n = nominal shear resistance of the section considered.

Core-fill timing specimens

Four specimens in the testing program served to investigate the difference in web-shear capacity due to core-fill timing. The typical immediate-fill specimen exceeded the web-shear capacity of the cold-joint core-fill specimen, both of which exceeded the web-shear capacity of the baseline empty-core specimen (Fig. 7). The cold-joint core-fill results exceeded the predicted web-shear capacity of a hollow-core slab with unreinforced core fill by an average of 10%, and the typical immediate-fill results exceeded the predicted web-shear capacity of a hollow-core slab with prestressed core-fill concrete by an average of 5%. These percentages were determined by dividing the average peak applied shear force by the unreinforced and prestressed concrete predicted shear capacities, respectively.

The baseline empty-core specimen test results exceeded the predicted hollow-core-slab web-shear capacity by 23%, which was likely due to the conservative nature of Eq. (3). It was not possible to explicitly measure the hollow-core-slab web-shear contribution in core-filled specimens because the contribution of the hollow-core-slab concrete and core-fill concrete to the overall hollow-core slab shear capacity cannot be differentiated. Because the hollow-core-slab web-shear conservatism was likely included in all of the core-filled tests, the web-shear contribution of the core-fill concrete presented in Fig. 7 likely attributed more web-shear capacity to the core-fill concrete than was realistic.

The aged core-filled specimen did not perform as well as the typical immediate-fill specimen. The dismantled broken hollow-core slab showed that the core-fill concrete was not composite with the hollow-core-slab concrete.

Core-fill enhancement specimens

The SFRC core-fill specimen was modified to potentially improve the core-fill shear capacity. This specimen had the lowest web-shear performance of all of the specimens in the testing program. In fact, the empty-core baseline specimen outperformed the SFRC core-filled specimen. These results were unexpected because even an unaltered core-filled specimen should have outperformed a hollow-core slab with no core fill.

The specimen with a headed steel bar in the core fill was expected to perform at least as well as the typical immediate-fill specimen because they were fabricated identically except for the headed bar. This specimen had a lower web-shear strength than the typical immediate-fill specimen (Fig. 7). Cracking of the core-fill concrete in the headed-steel-bar specimen was unlike all of the other core-filled specimens, all of which had exhibited a consistent diagonal crack through the entire core-fill concrete depth. Instead, the core-fill concrete of the headed-steel-bar specimen cracked more steeply from the hollow-core-slab top flange to the headed-steel-bar depth. At the bar level, the core-fill concrete cracks extended longitudinally following the steel bar to the member end (Fig. 5). This suggested that the steel bar promoted a failure plane along itself in the core-fill concrete that was uncharacteristic of the shear cracking observed in the other specimens.

The addition of WWR in the core fill provided transverse shear reinforcement similar to traditional stirrups, which was expected to increase the core-fill shear strength. The WWR specimen had a slightly lower web-shear strength than the typical immediate-fill specimen (Fig. 7). This result could be interpreted two ways. In one scenario, the core-fill concrete may have underperformed and the WWR provided an additional steel contribution to shear capacity so that the specimen performed similarly to the typical immediate-fill specimen. Alternatively, the WWR could have been ineffective and the core fill performed similarly to the typical immediate specimen core fill.

Roughening of the core wall before core filling was intended to improve the bond between the core-fill and hollow-core-slab concretes to promote the transfer of axial precompression to the core-fill concrete. The core-wall-surface-roughening specimen had a slightly higher web-shear strength than the typical immediate-fill specimen and exceeded the strength prediction calculated as if the core-fill concrete were prestressed (Fig. 7). In addition, the core-fill concrete remained connected to the adjacent hollow-core-slab webs.

Web-shear crack angles

The crack angle relative to the horizontal of each hollow-core-slab web and core-fill concrete indicated the orientation of the principal stresses in the concrete at web-shear failure. The angle of the core-fill crack face compared with the adjacent hollow-core-slab webs at the same depth likely indicated a difference in axial precompression. This comparison could also be used to suggest the relative level of precompression transmitted between the core-fill and hollow-core slab and the effectiveness of the core-fill enhancements.

Discussion of experimental results

Core-fill timing specimens

The empty-core, cold-joint, and typical immediate-fill specimens all slightly exceeded the predicted web-shear capacities of an unaltered hollow-core slab, a hollow-core slab with unreinforced core-fill concrete, and a hollow-core slab with prestressed core-fill concrete, respectively. This trend indicates that the core fill behaved like unreinforced concrete in the cold-joint specimen and like prestressed concrete in the typical immediate-fill specimen. The aged core-filled specimen had a lower web-shear capacity than the typical immediate-fill specimen. If no time-dependent detrimental effects had been present, the aged core-filled specimen should have performed similarly to the typical immediate-fill specimen. However, the aged core-filled specimen behaved as if the core fill were unreinforced concrete, indicating that the bond between the hollow-core-slab and core-fill concrete was poor. This was confirmed by the noncomposite core fill observed upon dismantling the specimen. The decreased web-shear performance of the aged core-filled specimen over time is critical because precast concrete structures are intended to perform as designed throughout their service lives.

Core-fill enhancement specimens

Although the low web-shear strength of the SFRC-core-filled specimen was unexpected, no explanation was available for why both SFRC core-fill tests had a much lower web-shear strength than even the empty-core specimen. The headed-steel-bar specimen also had a much lower web-shear capacity than expected. Because the only intentional difference between the headed-steel-bar specimen and typical immediate-fill specimen was the headed steel bar, the bar was suspected to have had an undesirable effect, which was counter to the predicted effect. Due to the atypical crack face in this specimen, the core-fill concrete area beneath the steel bar (approximately 15% of the core area) likely did not contribute to the core-fill shear strength, which could explain why the specimen appeared to underperform.

The WWR specimen and typical immediate-fill specimen were constructed identically except for the addition of WWR. Consequently, the web-shear capacity of the typical immediate-fill specimen was expected to be nearly identical to the WWR specimen's web-shear capacity less the contribution to shear strength possibly afforded by the added steel. However, the WWR was not anchored to any prestressing strands or supplemental longitudinal reinforcement; the typical immediate-fill and WWR specimens had very similar web-shear strengths. The estimated steel contribution to shear V_s was calculated to be 4.8 kip (21.4 kN) based on ACI 318[5] Eq. (22.5.10.5.3) (Eq. [6] in this paper). If this value had been subtracted from the average peak applied shear force of 66.8 kip (297.1 kN) (Table 2), the coefficient multiplied by the square root of the core-fill concrete strength would have been 3.3, which is less than the web-shear capacity of the typical immediate-fill specimen. This indicated that the added steel did not notably improve the web-shear strength, which was likely because the WWR was not anchored outside of the core fill. The core-fill concrete of the WWR specimen remained connected to the adjacent hollow-core-slab webs, which indicated a good bond between the hollow-core-slab and core-fill concretes. This further suggests that the core-fill concrete performed like prestressed concrete for this specimen. This axial precompression could have been maintained by the longitudinal bars of the WWR.

$$V_s = \frac{A_v f_{yt} d}{s} \quad (6)$$

where

A_v = area of shear reinforcement within spacing s

f_{yt} = specified yield strength of transverse reinforcement

s = center-to-center spacing of transverse reinforcement

Of the four core-fill enhancement specimens, only the core-wall-surface-roughening and WWR specimens had web-shear strengths similar to that of the typical immediate-fill specimen and exceeded the strength prediction calculated as if the core-fill concrete behaved like prestressed concrete. The two other enhancement strategies intended to improve the core-fill concrete strength (SFRC core fill and headed steel bar in the core fill) had web-shear strengths lower than that of the typical immediate-fill specimen and did not exceed the strength prediction calculated as if the core-fill concrete behaved like prestressed concrete.

Web-shear crack angles

Because the core-fill concrete was expected to have shear behavior that ranged between unreinforced and prestressed concrete, the crack angle of the core fill relative to the hollow-core-slab webs served as an indication of the amount of axial precompression present in the core-fill concrete. Based on Mohr's circle, concrete with no precompression will crack at approximately 45 degrees under an applied shear stress. As more axial precompression is added to a member that is also carrying an applied shear force, the angle of cracking will become shallower relative to the horizontal plane.

The typical immediate fill, core wall surface roughening, and WWR in core-filled specimens behaved the most like prestressed concrete (that is, the measured capacity of the core fill exceeded that of the predicted prestressed core-fill concrete behavior) (Fig. 7). Not coincidentally, the core-fill crack angles of these three specimens were shallow (34, 41, and 32 degrees, respectively) and near the average hollow-core-slab web crack angle of 33 degrees. This suggested that the core-fill concrete had axial precompression in these specimens, which contributed to additional web-shear capacity. Similarly, the cold-joint core-filled, SFRC-core-filled, and headed-steel-bar specimens had the steepest core-fill crack angles (60, 69, and 60 degrees, respectively) and had the lowest web-shear strengths of the hollow-core slabs, further supporting this correlation. The core-fill crack angle of 40 degrees in the 209-day typical-fill specimen was near the average crack angle of the hollow-core-slab webs (33 degrees), but the shear capacity of the hollow-core slab was lower than the predicted capacity, with the core fill treated as prestressed concrete. This was the only specimen that did not fit the correlation between core-fill crack angle and shear performance.

Composite action

After investigating the broken, deconstructed specimens, the core-fill concrete remained composite with the adjacent hollow-core-slab webs for the three specimens that had the highest web-shear strengths (typical immediate fill, core wall surface roughening, and WWR in core fill). The crack faces on these three specimens (**Fig. 8**) indicated that the core-fill concrete acted monolithically with the hollow-core-slab concrete because the crack face and angle across the hollow-core-slab webs and core-fill concrete were similar and uniform. This observation suggests that the bond between the hollow-core-slab and core-fill concrete was a critical factor in the web-shear strength of core-filled hollow-core slabs, particularly in the core-wall-roughening specimen. The other four core-filled specimens did not appear to have composite action between the core-fill concrete and hollow-core-slab concrete, as indicated by a crack around the perimeter of the core-fill concrete when viewed from the end of the hollow-core-slab and confirmed with deconstruction of the specimens after failure.

Typical immediate-fill specimen | Core-wall-surface-roughening specimen | Welded-wire reinforcement specimen

Figure 8. Core-fill concrete with a crack face angle similar to the adjacent hollow-core-slab webs.

Conclusion

Experimental web-shear tests were performed on 12 in. (304.8 mm) deep hollow-core slabs to investigate how a concrete-filled core augmented the web-shear capacity of the slab. Core-fill material was added both immediately following extrusion and following detensioning to examine the effects of core-fill timing relative to the core-fill web-shear strength. In addition, several core-fill enhancement strategies were implemented and tested to investigate practical methods to potentially improve the core-fill web-shear strength. The following conclusions were drawn based on the results and observations of the tests performed within this project.

- None of the core-fill enhancement strategies definitively demonstrated an improvement in web-shear capacity beyond the typical core-filled specimen. Rather, bond between the core-fill concrete and hollow-core-slab concrete appeared to be the most important factor for core-fill web-shear strength.

- When a core was filled with concrete within an hour of hollow-core slab extrusion and allowed to cure with the hollow-core slab before prestressing strand release, the total web-shear capacity of the hollow-core slab had the potential to perform as if both the hollow-core-slab and core-fill concretes were fully prestressed.

- When the prestressing strands of a hollow-core slab were released and the hollow-core slab was cured before core filling, a cold joint between the cured hollow-core-slab concrete and core-fill concrete often contained residual saw-cutting dust. Hollow-core slabs with core-fill concrete added in this time line performed as if the core fill was nonprestressed, unreinforced concrete.

- When a hollow-core slab with core fill added immediately after extrusion was allowed to age approximately seven months (five months more than the other specimens in this experimental study), the core-fill concrete shear strength decreased from prestressed concrete behavior to unreinforced concrete behavior. This decrease in core-fill shear strength was attributed to the additional concrete shrinkage experienced over a longer amount of curing time, which likely compromised the bond between the hollow-core-slab and core-fill concrete. As the bond weakened, the axial precompression present in the core-fill concrete decreased, resulting in a lower web-shear capacity of the hollow-core specimen.

- The core fills enhanced with steel-fiber reinforcement, a headed steel bar, and WWR did not show promise for improving the core-fill web-shear capacity.

- Vertically roughening the hollow-core-slab web walls inside the void before placing core-fill concrete improved the shear strength of the hollow-core specimen. The increase in core-fill shear capacity was attributed to a stronger bond and composite action between the slab concrete and core-fill concrete, which likely resulted in more axial precompression.

- Adequate bond between the hollow-core-slab concrete and core-fill concrete allowed the core-fill concrete to have axial precompression, which in turn increased the core-fill concrete shear capacity. This conclusion was supported by the observed crack angles of the core-fill concrete. Specifically, the three core-filled specimens with the greatest shear capacity (typical immediate fill, core wall surface roughening, and WWR) had shallow core-fill concrete crack angles relative to the horizontal plane and remained connected to at least one of the adjacent hollow-core-slab webs, which indicated the presence of axial precompression in the core-fill concrete and good bond with the slab concrete. Conversely, the three core-filled specimens with the lowest shear capacity (cold-joint core fill, SFRC core fill, and headed steel bar) had steep core-fill concrete crack angles and the core-fill concrete was not composite with the hollow-core-slab concrete. This indicated a lack of axial precompression and poor bond with the hollow-core-slab concrete.

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Notation

a = shear span

A_{cf} = core-fill concrete area

A_v = area of shear reinforcement within spacing s

b_w = width of web

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement

d_b = nominal diameter of prestressing strand

d_p = distance from extreme compression fiber to centroid of prestressing reinforcement

E_p = modulus of elasticity of prestressing steel

f'_c = nominal compressive strength of slab concrete

f'_{cf} = nominal compressive strength of core-fill concrete

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

f_{pi} = prestressing steel stress immediately before transfer

f_{pu} = specified tensile strength of prestressing steel

f_{yt} = specified yield strength of transverse reinforcement

h = section depth

l_{tr} = transfer length

s = center-to-center spacing of transverse reinforcement

V_{app} = maximum shear demand applied at failure

V_c = nominal shear capacity provided by concrete

V_{cw} = nominal shear capacity provided by concrete where diagonal cracking results from high principal tensile stress in web

V_n = nominal shear capacity of the section considered

V_p = vertical component of effective prestress force at section

V_s = nominal shear capacity provided by shear reinforcement

δ_{es} = measured end slip of strand

ϕ = shear strength reduction factor

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Abstract

Precast concrete manufacturers frequently enhance the shear capacity of hollow-core slabs by filling one or more of the cores with concrete in regions where the shear demand exceeds the slab capacity. Although routinely implemented, the core-fill web-shear performance has not been thoroughly studied and can be ambiguous in design. Sixteen experimental web-shear tests were performed on eight 12 in. (304.8 mm) deep hollow-core slabs, each slab tested twice, to investigate how a concrete-filled core augmented the web-shear capacity of the hollow-core slab. Core-fill material was added both immediately following extrusion and following detensioning to consider core-fill timing in regard to web-shear strength. In addition, several core-fill enhancement strategies were implemented and tested to investigate practical methods to potentially improve the core-fill web-shear strength. None of the core-fill enhancement strategies definitively demonstrated an improvement in web-shear capacity beyond the typical core-filled specimen. Rather, bond between the core-fill concrete and hollow-core-slab concrete appeared to be the most important factor for core-fill web-shear strength.

Keywords

Core fill concrete, crack angle, fiber-reinforced, hollow-core slab, shear capacity, web shear, welded-wire reinforcement.

Review policy

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

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