Shear Strength of 16 in. Thick Hollow-Core Slabs

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Abstract

Hollow-core (HC) slabs are the most dominant and economical precast prestressed concrete flooring system. HC slabs are commonly produced in the US market with thickness ranging from 8 in. to 12 in. (w/o 2 in. composite topping). Recently, deeper HC slabs (16 in. thick) have been produced more often to satisfy the growing need for longer spans and/or heavier loads. According to ACI 318-14 Section 7.6.3 "Minimum Shear Reinforcement", the minimum shear reinforcement is required where ultimate shear force is greater than 50% of the concrete shear strength (ϕV_{cw}) for precast prestressed HC slabs with un-topped thickness greater than 12.5 in. This requirement was based on previous testing and can be waived or changed if new testing proves otherwise. This paper presents additional testing conducted to evaluate the shear strength (V_n) of new 16 in. thick precast prestressed concrete HC slabs recently produced by Concrete Industries, Inc. Testing results of these new 16 in. HC slab indicated that ACI 318-14's 50% reduction factor for web shear strength may be overly conservative.

Introduction

The American Concrete Institute's (ACI) shear provisions for prestressed concrete were developed by testing prestressed concrete girder sections (i.e. deep sections). The applicability of these provisions to prestressed concrete hollow-core (HC) slabs was investigated by Anderson (1978) and Becker and Buettner (1985) to address concerns about shear strength of prestressed concrete with very low slump (almost zero slump) used in extruded HC slabs (i.e. shallow sections). Anderson (1978) performed experimental testing on HC slabs with thickness ranging from 6 in. to 14.5 in., while Becker and Buettner (1985) performed experimental testing on 8 in. and 10 in. thick HC slabs. Both researchers concluded that the actual shear strength of these slabs were in excess of the strength predicted by ACI's shear provisions. However, shear design provisions for HC slabs remained the same in the ACI design code. As of 1980s, deeper HC slabs started to be produced based on the market needs in Europe, which led the European researchers to investigate behavior of HC slabs that are 14 in. to 20 in. thick without any shear reinforcement. The European researchers concluded that the traditional design method in Eurocode 2 (EC 2) overestimates the web shear strength of HC slabs with depths greater than 12 in. (Pajari, 2005). Also, new requirements for HC shear strength design were adopted by ACI 318-08 for slabs deeper than 12 in. based on published work by Hawkins and Ghosh (2006). They reported that some of the tested HC slabs produced by three different US manufactures failed in web shear at 60 percent or less of the load predicted by ACI 318-05. The lower shear strength agrees with the European experience with deep HC slabs (Yang, 1994 and Pajari, 2005). Therefore, since 2008, ACI required the web shear strength (ϕV_{cw}) to be reduced by 50% for HC slabs with depths greater than 12.5 in. when minimum shear reinforcement is eliminated, which is the common practice due to the difficulty of placing shear reinforcement in HC slab. Web shear provisions of ACI 318-14 will be referred to throughout this paper as it is the current design code of HC slabs in the US.

This paper briefly summarizes the parameters that affect web shear strength of deep HC slabs, and presents the results of testing eight full-scale 16 in. thick HC slabs.

Parameters affecting web shear strength of HC slabs

Many parameters affect web shear strength of HC slabs. Some of these parameters are addressed by previous research (Jonnson, 1988; Kani, 1967; Collins and Kuchma, 1999; Bazant and Kazemi, 1991; Angelakos, et al., 2001; Pajari, 2005; Yang, 1994; Walraven and Mercx, 1983; Nilson, 1987; Shahawy, et al., 1992; MacGregor 1997; Bentz, 2005; Palmer and Schultz, 2010; and Palmer and Schultz, 2011). These parameters include:

- Load and support configuration
- Shear span-to-depth ratio (a/d)
- Prestressing level
- Concrete compressive strength
- Geometry of cross-section
- Overall unit thickness

Load and support configuration

Jonnson (1988) investigated the effect of load/support configuration on web shear carrying capacity. Jonnson concluded that having discrete supporting systems for HC slabs can reduce the shear strength as much as 50 % of its maximum expected shear strength. This is in agreement with the results of testing specimen 1A, which will be presented later in this paper. A premature failure of this specimen occurred as the supported width was less than the specimen width, which led to premature failure of the exterior unsupported webs.

Shear span-to-depth ratio (a/d)

Web shear strength is affected by shear span-to-depth ratio a/d, where "a" is the shear span and "d" is the member's depth. This effect is attributed to the arching action that takes place when the load is closer to the support (Jonnson, 1988; MacGregor, 1997; and Palmer and Schultz, 2010). Jonnson (1988) studied the effect of shear span-to-depth ratio ranging from 1.0 to 7.0 for 10.5 in. thick HC slabs and concluded that the shear strength can be reduced by 50%, when having a/d = 7.0 compared to the case having a/d = 1.0. Other studies showed that a/d ratio greater than 2.0 would mitigate the arching effect (Hawkins et al., 2005; and Palmer and Schultz, 2010).

Prestressing level

Axial compressive stresses due to prestressing are recognized by most shear design provisions in various codes. Prestressing enhances the web shear carrying capacity since it tends to reduce the principal tensile stress, causing an increase in the web shear strength as the web shear failure occurs when the principle tensile stress reaches its limiting concrete tensile strength. However, the critical section for design is traditionally taken at h/2 from the face of the support, which means that the section is more likely to be within the transfer length of the prestressing strands especially for shallow members. A generally accepted model for the variation of the stress in prestressing strand is a straight line from zero stress at the point where bonding commences to the effective prestress at the end of the transfer length (AASHTO, 2014). Numerous research has been done on determining transfer length of prestressing strands as it affects shear strength of prestressed members (Russell and Burns, 1993; Mitchell, et al. 1993). Current design codes propose different formulas for estimating transfer length (l_t) of 7-wire strands as a function of strand diameter (d_b): ACI 318 (2014) proposes a transfer length of $50d_b$, AASHTO (2014) proposes a transfer length of $60d_b$, and JSCE (2010) proposes a transfer length of $65d_b$. However, many other factors affect the transfer length in pretensioned member, such as the strand surface condition, concrete compressive strength at release, type of concrete mixture, center-to-center spacing between strands, concrete cover, concrete section shape and strand distribution, and de-tensioning method.

Concrete compressive strength

Concrete shear strength is directly related to its tensile strength as diagonal shear cracks start to take place when principle tension stresses reach the tensile strength. The concrete compressive strength is generally used in evaluating the tensile strength due to the difficulty associated with conducting direct tension tests and disparity of the test results comparing to compression test results. For example, contribution of concrete in shear resistance, in the traditional shear equations in ACI 318, is proportioned to the square root of the concrete compressive strength, f'_c. Similarly, in the majority of design codes, the shear strength of a member is directly proportioned to $(f'_c)^x$, where the power (x) differs from code to another to reflect the concrete tensile strength.

Geometry of cross-section

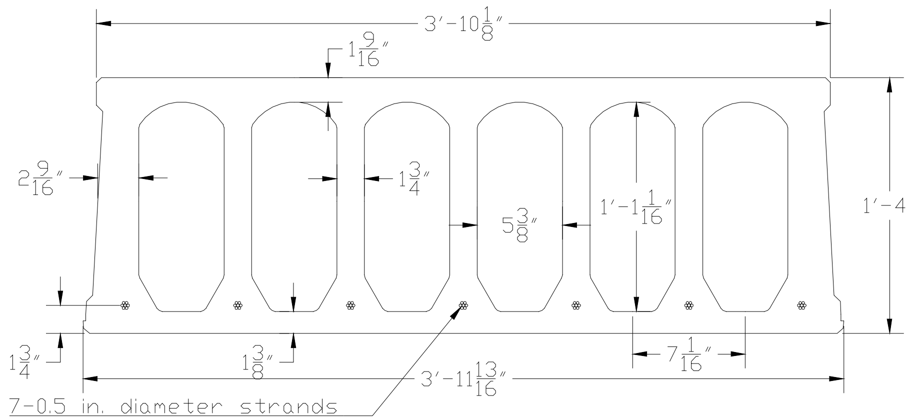
The Australian standard (AS 3600-2009) does not specify a certain equation for web shear design of prestressed members, instead, it requires the web shear strength to be calculated based on the principal tensile strength at either the centroidal axis, in the case of HC with circular voids, or the intersection of bottom flange and web, in case of HC with non-circular voids (NPCAA, 2003). Pajari (2005) reported that when using Yang's method for calculating shear strength of HC slabs, the location of the critical point is the centroidal axis for slabs with circular voids. However, for slabs with non-circular voids, the critical section is located at the junction of the web and the flange, which confirms that the geometry of the cross section plays a role in determining the shear strength of HC slabs.

Overall unit thickness

Palmer and Schultz (2010) studied the effect of HC slab thickness on the shear strength using data from five different experimental programs for a total of 198 slabs with less than 30 slabs unit greater than 12.5 in. thick. This study could not clearly indicate the thickness effect due to the disparity in the test data. Shioya et al. (1989) tested concrete members with thickness ranging from 4 in. to 18 in. without transverse reinforcement and subjected to a uniformly distributed load. Shioya et al. (1989) concluded that shear stress at failure decreases as the depth of the member increases. NCHRP report 549 indicated that members greater than 36 in. deep failed under stresses approximately one-half of the strength calculated by ACI 318 and AASHTO specifications.

Experimental program

Experimental program was conducted to investigate the web-shear and flexure-shear strength of 16 in. thick HC slabs. A total of eight full-scale HC slab specimens were fabricated by Concrete Industries Inc. in Lincoln, NE.: four slabs, 16 ft long each, were tested in web-shear at both ends using three-point loading for a total of eight tests; and four slabs, 24 ft. long each, were tested for flexure-shear using four-point loading for a total of four tests. Figure 1 shows a typical cross-section dimensions and geometric properties of the tested HC slabs. The specified concrete compressive strength of the HC slabs was 9 ksi, however, the average compressive strength at the time of testing was found to be approximately 10 ksi. All slabs were reinforced using seven 0.5 in. diameter Grade 270 low-relaxation seven-wire strands pre-tensioned to 70% of the ultimate strength.



Area = 349 in.^{2} Weight = 0.35 kip/ft.

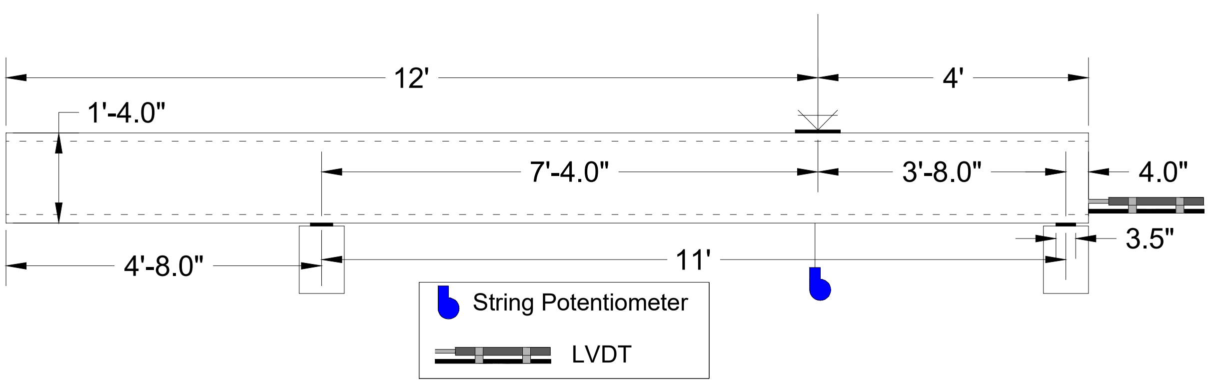
b_w = 13.875 in. d_p = 14.25 in.

Y_{bot} = 7.956 in. I_g = 10,941 in.^{4}

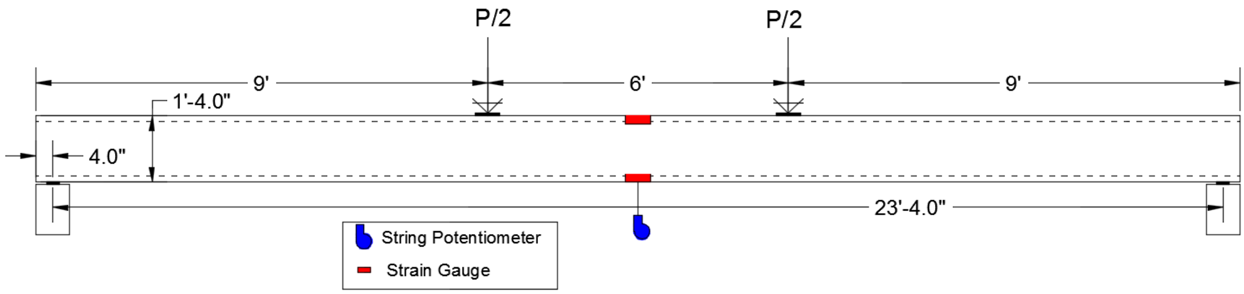
Figure 1: Cross-section dimensions and geometric properties of tested HC slabs

Test setup and instrumentation

The web-shear testing was conducted using one concentrated load applied at a shear span of 3ft-8 in. (a/d = 2.75) as shown in Figure 2a. The flexure-shear testing was conducted using two concentrated loads applied at 3 ft. from the mid-span section, as shown in Figure 2b. All loads were applied across the entire width of the specimen using steel and wood beams and 400-kip hydraulic jack supported on a steel frame anchored to the strong floor as shown in Figure 3. Loads were measured using a 450-kip load cell and deflections were measured using string potentiometer with 0.001 in. accuracy. Strand slippage was measured during the web-shear testing for two interior strands at the loaded end using two linear variable differential transformers (LVDTs) as shown in Figure 2a. Concrete strain gauges were used to measure the strain at the top and bottom fibers of the slab during flexure-shear testing. Steel rollers that are 39 in. long were initially used in the first web-shear test (#1A) and two 4x4 dimension lumber that are 48 in. long were used afterwards to support the full width of the specimen as shown in Figure 3.



a) Test setup and instrumentation for web-shear testing of 16 ft long HC specimens



b) Test setup and instrumentation for flexure-shear testing of 24 ft long HC specimens

Figure 2: Test setup and instrumentation

Figure 3: Web-shear test specimen supported by 48 in long 4x4 dimension lumber

Web-shear test results

Figure 4 shows the shear-deflection relationships of the eight shear tests (test A and test B for each of the four specimens). These relationships indicate that all specimens behaved linearly with similar slope up to 0.2 in. deflection, then non-linearly with a much lower slope up to failure. Exceptions are: test #1A that shows premature failure due to using short supports causing horizontal cracks between the exterior webs and the top flange as shown in Figure 5; and tests #2B and #4B where the specimens experienced significant strand slippage due to the excessive cracking occurred while testing the other ends (#2A and #4A) as shown in Figure 6. All other tests had resulted in typical diagonal web-shear cracks in areas adjacent to the support as shown in Figure 7. The failure in these specimens was sudden without any noticeable cracking before failure. Table 1 lists the applied load, shear, deflection at load location, and corresponding strand slippage at failure. The predicted web-shear strength (V_{cw}) using ACI 318-14 equation 22.5.8.3.2 was 83.2 kip and calculated as follows

Where

f_{pc} is the compressive stress in concrete at centroid of cross section resisting externally applied loads after allowance for all prestress losses at h/2 from support face. The transfer length is calculated using according to ACI 318-14 section 22.5.9 as 50 d_b (25 in.). Total losses are assumed to be 15 % (Pajari, 2005 and Palmer and Schultz, 2010). d_p is the effective shear depth, taken as the greater of 0.9d_p or 0.72h.

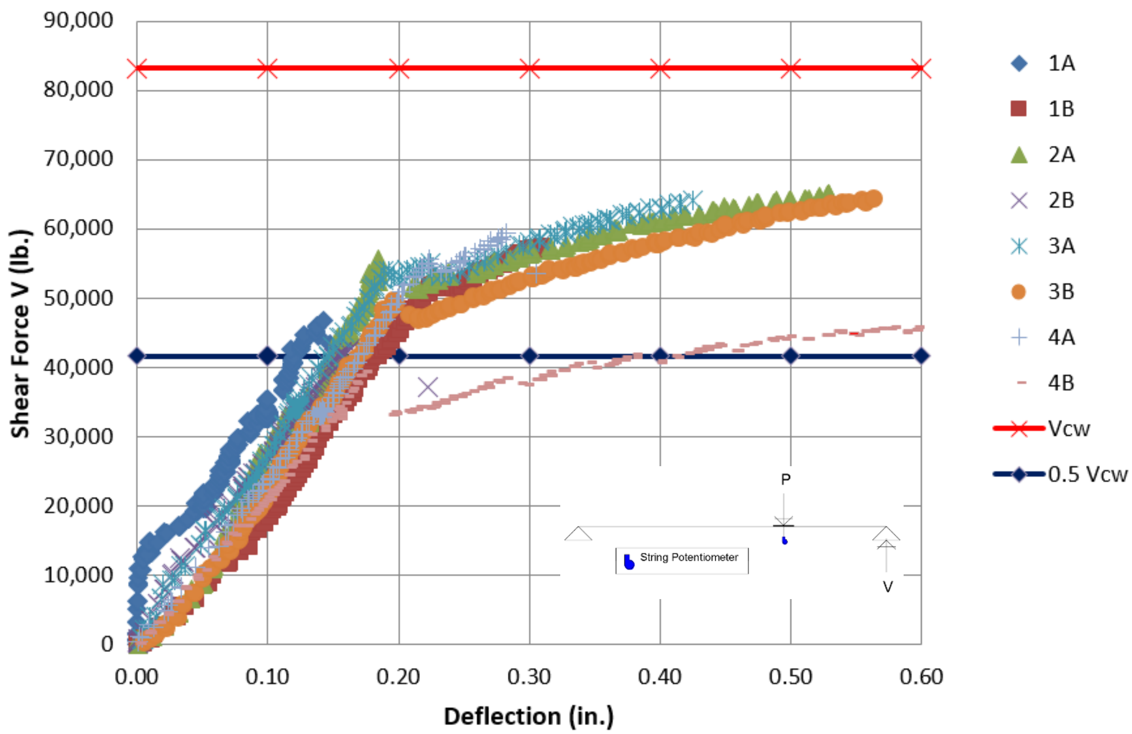


Figure 4: Shear-deflection relationships

Table 1: Shear test results of the 16 ft long HC specimens

Specimen	Ultimate Load (lb)	Ultimate Shear (kip)	V_u/V_{cw}	Corresponding Deflection (in.)	Corresponding Slippage (in.)
#1A*	70.0	48.2	0.58	0.142	0.0035
#1B	86.0	58.8	0.71	0.310	0.0124
#2A	97.7	66.6	0.80	0.529	0.0893
#2B*	64.1	44.2	0.53	0.164	-0.0001
#3A	96.2	65.7	0.79	0.425	0.0565
#3B	96.5	65.8	0.79	0.563	0.0900
#4A	89.1	60.9	0.73	0.282	0.0070
#4B*	71.7	49.3	0.59	0.698	0.2186
Average (all)	**83.9**	**57.4**	**0.69**		
COV (all)	**16.0%**	**15.6%**	**15.6%**		
Average (excl.*)	**93.1**	**63.6**	**0.76**		
COV (excl.*)	**5.6%**	**5.5%**	**5.5%**		

* These specimens did not have web-shear cracking failure at h/2

Figure 5: Failure of specimen #1A due to inadequate bearing

Figure 6: Failure of specimens #2B and #4B

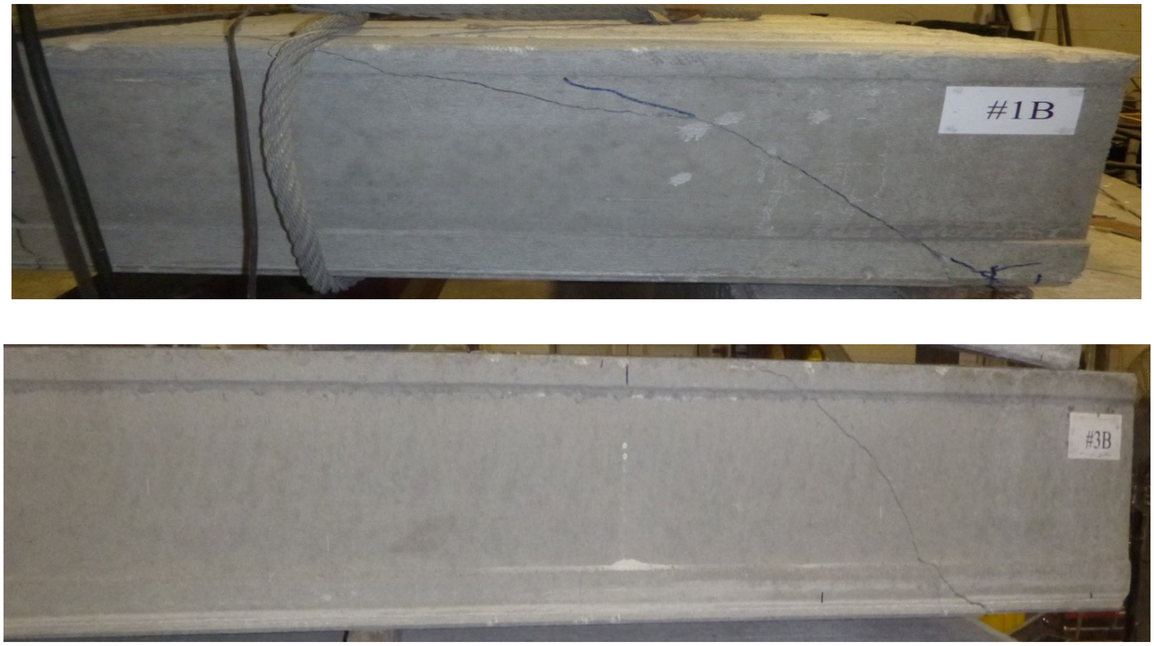


Figure 7: Web-shear failure cracks

Flexure-shear test results

Figure 8 shows the load-deflection relationships of the four 24 ft long HC slab specimens tested in flexure. These relationships indicate that the four specimens behaved linearly with approximately the same slope up to the cracking load (averaged 54.5 kips and 0.5 in. deflection), then, non-linearly up to the ultimate load (averaged 82.3 kips and 9.5 in. deflection). Table 2 lists the test results of the four specimens as well as the average and coefficient of variation. All specimens had vertical flexural cracks starting from the bottom fibers at the loading points, as shown in Figure 9, after reaching the cracking load. As the load increased, these cracks propagated upward to form inclined flexural-shear cracks as shown in Figure 10. All the specimens had demonstrated flexural strength higher than predicted by ACI 318-14 using strain compatibility procedure (M_n = 335 k.ft). Flexural-shear strength for all specimens was also slightly higher than predicted by using ACI 318-14 equation 22.5.8.3.1 (V_{ci} = 37.5 kips) and calculated as follows

Figure 8: Load-deflection relationships of the 24 ft long HC specimens

Table 2: Test results of the 24 ft long HC specimens

Specimen No.	Cracking Load (kip)	Cracking Moment (kip.ft)	Cracking M_{test}/M_{cr}	Ultimate Load (kip)	Ultimate Moment (kip.ft)	Ultimate M_{test}/M_n	V_{test}/V_{ci}	Ultimate Deflection (in.)	Ultimate Comp. Strain ($x10^{-6}$)
#1	56.0	265.0	1.14	82.1	378.2	1.13	1.09	9.86	2,288
#2	53.0	252.0	1.08	79.4	366.2	1.09	1.06	8.67	1,954
#3	55.0	260.6	1.12	82.7	380.8	1.14	1.1	10.02	1,327
#4	54.0	256.3	1.10	85.0	390.9	1.17	1.13	9.56	1,869
Averag	**54.5**	**258.5**	**1.11**	**82.3**	**379.0**	**1.13**	**1.1**		
COV	**2.4%**	**2.2%**	**2.2%**	**2.8%**	**2.7%**	**2.7%**	**3%**		

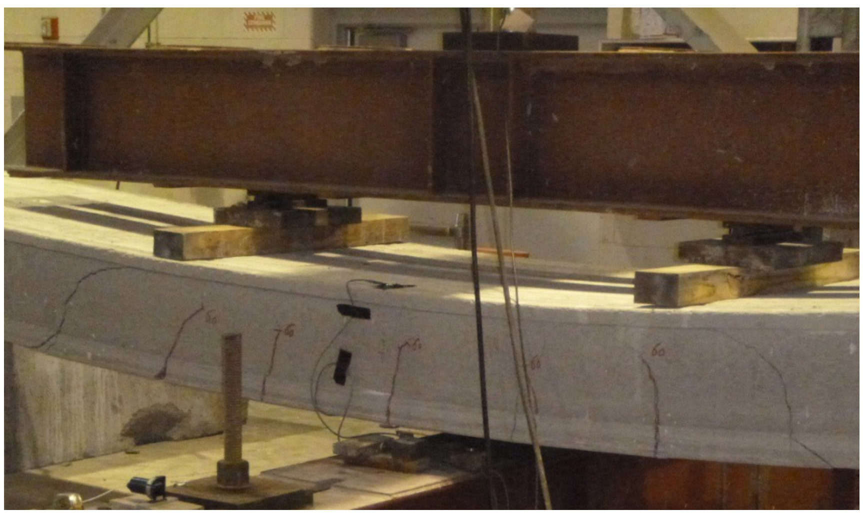


Figure 9: Flexure cracks under cracking load

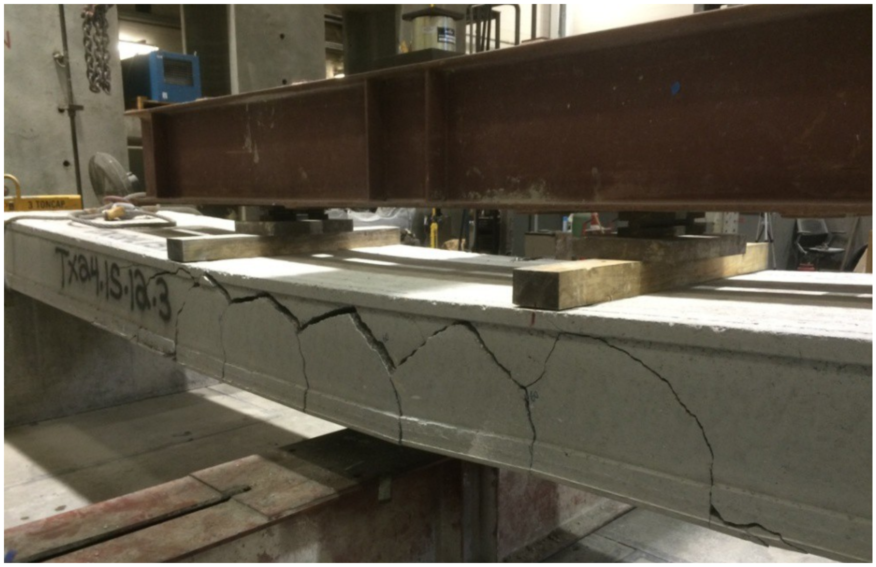


Figure 10: Flexure-shear cracks under ultimate load

Conclusions

The paper presented the shear and flexure testing of the new 16 in. thick HC slabs produced by Concrete Industry, Inc. The main conclusions of this study can be summarized as follows:

1. The measured concrete web shear strength (V_{cw}) of the 16 in. thick HC slab averaged 76% of the predicted using the ACI 318-14 equation 22.5.8.3.2 at h/2 from the face of the support (without 0.75 shear strength reduction factor) with a low 5.5% coefficient of variation. Therefore, a reduction factor higher than 50% (75% for example) can be used when calculating V_{cw} for 16 in. thick precast/prestressed HC slabs as long as a continuous support along the full width of HC is provided and strands development is considered.
2. The length of bearing supports of the HC slabs is proven to be a crucial factor to the shear strength of the member. A continuous support across the entire width of the HC slab is necessary to achieve the full strength. Although the shear strength was adversely affected by the short (39 in.) steel roller bearing support, in specimen #1A, the tested capacities do give engineers some guidance on shear strength at real situations with partially missing slab bearing, such as HC slab notching around columns on steel structures.
3. The measured flexural strength of the 16 in. thick HC slabs is 13% higher than that predicted using strain compatibility (with strength reduction factor of 1.0) and the measured cracking moment strength is 11% higher than predicted using gross section properties and prestress losses of 15%.

It should be noted that the results of this study are applicable for the HC slab and concrete mixture used in this investigation and may not be applicable to other products made by different manufacturers.

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