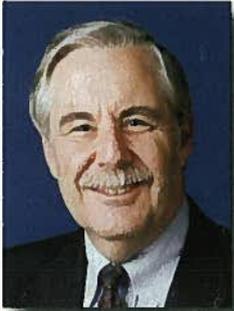


Shear Strength of Hollow-Core Slabs



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Reinforced concrete members without shear reinforcement can fail at loads less than those predicted by ACI 318-05, Eq. 11-3. Thus, there is an additional requirement for minimum shear reinforcement if V_u exceeds $0.5\phi V_c$. This requirement, however, is waived for slabs, including hollow-core units. Concern has been raised recently over the lack of shear reinforcement in deep one-way slabs, particularly those using high-strength concrete, high-strength reinforcement, or both. Several U.S. precast concrete manufacturers have chosen to conduct shear tests on hollow-core units with depths greater than 12 in. (300 mm). This paper presents the results of shear strength tests performed by three U.S. manufacturers as well as European research relating to hollow-core shear strength. The European and U.S. test data are compared and recommendations on further research are presented.

Concern has been raised recently over the lack of shear reinforcement in deep one-way slabs, particularly those using high-strength concrete, high-strength reinforcement, or both. Several U.S. precast concrete manufacturers have chosen to conduct shear tests on hollow-core units with depths greater than 12 in. (300 mm).

BACKGROUND

Hollow-core units were developed in the 1950s, when long-line prestressing techniques evolved, and for more than 30 years the type of units produced changed little; typical units in the United States were a maximum of 12 in. (300 mm) thick. There was an increase in research performed in Europe in the 1980s that related to hollow-core units, which led to technology advances allowing the economical production of units with depths greater than 12 in. (300 mm). The trend of manufacturers who are increasingly able to supply units with depths greater than 12 in. (300 mm) has now spread to the United States.

The *PCI Manual for the Design of Hollow Core Slabs*¹ notes that there are essentially two methods used to produce hollow-core units in the United States. One is a dry-cast (or extrusion) system where a very low-slump concrete is forced through the casting machine. Cores in the units are formed with augers or tubes, and the concrete is compacted around the cores. The second system uses a higher slump concrete. The sides are created by stationary forms (or by slip forming) with forms attached to the machine. Cores are typically created by pneumatic tubes attached to the form or by slip forming with long tubes attached to the casting machine. **Table 1,**

Table 1. Hollow-core systems from eight major hollow-core manufacturers used in the United States.

Manufacturer	Machine Type	Concrete Type/Slump	Core Form
Dy-Core	Extruder	Dry/Low	Tubes
Dynaspan	Slip Form	Wet/Normal	Tubes
Echo	Slip Form	Wet/Normal	Tubes
Elematic	Extruder	Dry/Low	Auger/Tube
Flexicore	Fixed Form	Wet/Normal	Pneumatic Tubes
Spancrete	Slip Form	Dry/Low	Tubes
Span Deck	Slip Form	Wet/Normal	Filler Aggregate
Ultra-Span	Extruder	Dry/Low	Augers

an update of information reported in Reference 1, shows the eight major hollow-core manufacturing systems used in the United States.

DESIGN OF HOLLOW-CORE SLABS

The design of hollow-core slabs is governed by ACI 318² requirements. Chapter 2 of the *PCI Design Handbook*³ provides information on dimensions, section properties, and load-carrying capacities of hollow-core slabs. The *Design Handbook* shows cross sections and section properties of proprietary hollow-core slabs along with load tables for uniform loading that apply for non-proprietary sections varying from 6 in. (150 mm) to 12 in. (300 mm) in depth. In addition, in the proprietary tables, properties were provided by four precast suppliers for sections greater than 12 in. (300 mm) in depth, with 16 in. (410 mm) being the maximum depth listed. The load tables in the *Design Handbook* are based on the ACI 318 requirements for flexural strength, service load flexural stresses, and shear strength. Note that for loading conditions other than uniform loading, the design professional needs to perform separate calculations.

Shear Strength

The use of shear reinforcement is generally not feasible for hollow-core slabs and, therefore, the shear strength, particularly of deep slabs, may be limited to the shear strength of the concrete. Section 11.4 of ACI 318 gives the requirements for evaluating the shear strength of concrete. The provisions of Section 11.4.3 are likely to be used if shear is a controlling factor in the design of the hollow-core unit. In Section 11.4.3, the factored shear force V_u is limited to the lesser of ϕV_{ci} and ϕV_{cw} , where V_{ci} is the flexure-shear cracking strength and V_{cw} is the web-shear cracking strength. For simply supported hollow-core units, the shear cracking strength of the web adjacent to the support usually does not control the design of the unit unless the design loading includes heavy, non-uniform loads. The ϕ value for shear calculations is 0.75.

The nominal shear strength provided by concrete (web-shear cracking strength) V_{cw} is given in ACI 318 by Eq. (11-12):

$$V_{cw} = (3.5\sqrt{f'_c} + 0.3f_{pc})b_w d_p + V_p \quad (11-12)$$

where d_p is the distance from extreme compression fiber to the centroid of prestressing steel; b_w is the web width; f_{pc} is the compressive stress in the concrete at the centroid of the

cross section; f'_c is the specified compressive strength of concrete; and V_p is the vertical component of effective prestress force. The depth d_p need not be taken as less than $0.8h$ where h is the overall thickness of the slab including any composite topping.

Sections located less than a distance $h/2$ from the face of the support can be designed for V_u (factored shear force at the section) computed at a distance $h/2$. The distance $h/2$, however, is often closer to the end of the member than the transfer length for the prestressing steel. In such cases a reduced prestressing force has to be used in calculating V_{cw} . Per Section 11.4.4, the prestress force is assumed to vary linearly from 0 at the end of the slab to f_{pc} at the end of the transfer length. That transfer length is taken as 50 diameters for strand or 100 diameters for single wire.

The shear strength of reinforced concrete members is given in ACI 318 by Eq. (11-3):

$$V_c = 2\sqrt{f'_c}b_w d \quad (11-3)$$

where d is the distance from extreme compression fiber to the centroid of longitudinal tension reinforcement; b_w is the web width; f'_c is the specified compressive strength of concrete; and V_c is the nominal shear strength provided by concrete.

There is a discontinuity in concept between the evaluation of the shear strengths of prestressed members by Eq. (11-12) and the evaluation of the shear strengths of reinforced concrete members by Eq. (11-3). As f_{pc} decreases along the transfer length, it is reasonable to assume that the strength for a prestressed concrete beam should approach that of a reinforced concrete beam. Values from the two equations are not consistent with that assumption.

EFFECT OF BEAM DEPTH ON SHEAR STRENGTH

Since the 1970s, researchers have recognized⁴ that deep reinforced concrete beams without shear reinforcement can fail in shear at loads less than those calculated using Eq. (11-3). To counter that effect, Section 11.5.6 of ACI 318 has required minimum shear reinforcement in most prestressed and non-prestressed concrete members if V_u exceeds $0.5\phi V_c$. One member type excluded from that requirement, however, is slabs, including hollow-core slabs, because most slabs in buildings are not deep and it is difficult to place shear reinforcement in such members.

Recent research⁵ has raised concerns about the minimum shear reinforcement exception for deep, lightly reinforced, one-way slabs, particularly those made with high-strength concrete and high-strength steel. In tests⁶ at the University of Toronto, a 36-in.-deep (1 m) member without web reinforcement, and made with 5300 psi (37 MPa) strength concrete, failed at a nominal shear stress of $1.3\sqrt{f'_c}$. In tests⁷ at North Carolina State University, a 36-in.-deep (1 m) member without web reinforcement made with 4650 psi (32 MPa) strength concrete developed inclined cracking at a shear stress of $1.2\sqrt{f'_c}$ and failed at a nominal shear stress of $1.56\sqrt{f'_c}$. The University of Toronto tests showed that the nominal shear stress at failure starts to drop below the strength calculated from Eq. (11-3) once the depth exceeded about 20 in. (500 mm). The use of Eq. (11-10) and (11-12) to evaluate the shear strength of hollow-core slabs with depths of 12.5 in. (320 mm) or less was validated 20 or more years ago through testing.^{8,9} The shear strength findings for deep reinforced concrete members, however, have raised concerns about the shear strength of deep hollow-core units and have led to several hollow-core manufacturers conducting shear tests on units with depths greater than 12 in. (300 mm).

RESULTS OF SHEAR STRENGTH TESTS OF HOLLOW-CORE UNITS

The authors are aware of the test results on units with depths exceeding 12 in. (300 mm) by three different hollow-core suppliers. Table 2 summarizes those test results. For all results shown in Table 2, the shear corresponding to the development of the flexural strength of the member was greater than V_{test} and was considerably greater than V_{cw} .

For the tests by Suppliers A and B, the V_{test}/V_{cw} values show considerable variation. This suggests that results are sensitive to the test setup or to variations in the characteristics of the test members. The f_{se} value at the critical section for web-shear cracking was always computed to be less than the yield strength for the prestressing strand at the computed V_{cw} value. Therefore, results were sensitive to the bond properties and transfer length for the strand. Supplier A measured

the amount of strand slip at release and calculated transfer lengths using the procedure described in Reference 1. The amount of strand slip varied widely from strand to strand. Measurements confirmed an average value of 50 strand diameters for the transfer length, however.

For units with overall depths greater than 12.5 in. (320 mm), the results in Table 2 suggest a high probability that values of V_{test}/V_{cw} are less than unity. Comparisons of the variations in the V_{test}/V_{cw} results with depth suggest, however, that the rate at which depth effects develop will differ from supplier to supplier. Variations with depth may also be more related to the effective depth to the prestressing steel than to the overall depth of the member.

The $V_{test}/b_w d$ values for the 16-in.-deep (410 mm) units provided by Supplier A averaged only $2.66\sqrt{f'_c}$. Therefore, for these members, the shear strength was more like that of a reinforced concrete member than a prestressed concrete member. The principal tensile stress in the concrete at the centroidal axis of those units was clearly less than $4\sqrt{f'_c}$ at the time of failure.

In all cases where the web-shear cracks resulted in failure, the cracks seemed to initiate in the units close to the face of the support. In the units tested by Supplier B, horizontal cracking along the junction of the webs and lower flange often appeared to be part of the shear failure mechanism. In general, the results also indicate that if bond slip of the strand occurred, it was after the formation of the web-shear crack. In particular, the end slip measurements made in the tests by Supplier A suggest that bond slip was not the cause of failure.

While the test data indicate that web-shear cracking strengths may be less than those calculated by ACI 318-05 procedures, none of the tests showed flexure-shear cracking strengths less than those calculated by ACI 318-05. In some of the tests, flexure-shear cracks developed but they did not result in failure and they occurred at shear values consistent with those predicted using Eq. (11-10) of ACI 318-05. Hence, it appears that the reduction in the web-shear cracking strength with depth is related to the spread of the prestressing force into the section and associated with a combination of effects due to shear lag, anchorage bond, prestress location, and the geometry of the cross section of the unit.

Table 2. Summary of results of hollow-core web-shear cracking tests.

Supplier	No. of Tests	h (in.)	A_{ps} (in. ²)	b_w (in.)	d_p (in.)	f_c (ksi)	f_{se} (psi)	No. of V_{cw} Failures	V_{test}/V_{cw} Avg.	Range
A	6	15	1.224	15.27	14	6*	175	6	0.76	0.61–0.83
A	6	16	1.53	15.93	15	6*	175	6	0.58	0.53–0.64
B	8	16	1.786	11.23	13.75	8–10	189	8	0.85	0.74–1.02
C	3	12.5	1.683	8.5	10.75	10	189	2†	1.47	1.46, 1.48
C	5	14.5	1.224	8.5	12.75	11	189	0‡		1.27–1.61

*Specified strength; actual strength not measured.

†Remaining failure due to strand slip at shear of $1.46 V_{cw}$ and before web-shear cracking.

‡All five failures due to strand slip before web-shear cracking.

Note: All strands were 0.5-in.-diameter 270 ksi, except for the four 0.6-in.-diameter 270 ksi strands used in combination with the eight 0.5-in.-diameter strands by Supplier B and the use of 0.5-in.-diameter 250 ksi strands for the 15-in. product by Supplier A. All units were 4 ft wide. 1 in. = 25.4 mm; 1 ksi = 6.89 MPa; 1 psi = 6.89 kPa.

SHEAR STRENGTH OF HOLLOW-CORE UNITS: EUROPEAN EXPERIENCE

Elliott¹⁰ has summarized European experience on evaluating the web-shear strength of hollow-core units. In particular, Yang¹¹ evaluated the web-shear cracking strengths observed in 59 different tests conducted by the Technical Research Center of Finland (VTT) between 1978 and 1987. Those tests used hollow-core units provided by 10 different manufacturers and made using the extrusion method. The units contained three different void geometries, with two of the geometries being similar to those used by Suppliers B and C in the U.S. tests. The overall depths of the units ranged from 7.5 in. (190 mm) to 16.0 in. (410 mm). As was the case for the U.S. tests, Yang found that the flexure-shear strengths were in reasonable agreement with code predictions but that web-shear strengths showed considerable scatter compared with code predictions. Yang also observed that the rate at which the prestressing force was transferred to the concrete within the transfer length could significantly affect the web-shear strength. Web-shear cracking occurred when the principal tensile stress in the units (at the intersection of the narrowest web and the bottom flange) reached the tensile strength of the concrete for a location on a line drawn from the centerline of the support at an angle of 35 degrees to the axis of the unit.

In additional VTT tests, Pajari and Koukkari^{12,13} examined the shear strength of hollow-core floors where units are seated on flexible supports. Tests were conducted on continuous two-span floor systems consisting of 23.6-ft-long (7.2 m) by 10.4-in.-deep (265 mm) hollow-core units, 5 units across, supported on 10 different types of central, precast concrete inverted tee and steel-plate beams. In Reference 13, it is suggested that for practical purposes, the web-shear strength of a unit should be evaluated using Yang's model to locate the distance of the critical section from the support. The shear stress and effective prestress force at the centroid of the unit are then used to determine its web-shear strength. That approach is equivalent to evaluating V_{cw} using Eq. (11-12) but at a location different from the $h/2$ distance from the face of the support specified in ACI 318-05.

Recently Pajari¹⁴ examined VTT results for 49 tests on units ranging from 7.9 in. (200 mm) to 19.7 in. (500 mm) deep and failing due to web shear. Pajari compared measured strengths with those calculated using Eq. 6.4 of EC2¹⁵ and Yang's model; the EC2 method overestimated the mean shear strength for all slab types. While the overestimation was only slight for slabs with circular voids, it was obvious for units with flat webs. For units with flat webs and depths equal to or greater than 10.4 in. (265 mm), strengths were in reasonable agreement with those calculated using Yang's model.

CORRELATION OF EUROPEAN FINDINGS AND U.S. TEST DATA

Table 3 shows the dimensions that result when Yang's model is used to determine the location of the critical section for V_{cw} evaluations for the largest unit tested by each U.S. supplier. In Table 3, the first column lists the shortest distance h_{cp} from the narrowest point on the web of the unit to the bot-

Table 3. Location of critical section according to Yang's model.

Supplier	h_{cp} (in.)	$h_{cp}/0.7$ (in.)	x_{cp} (in.)	$h_{cp}/0.7$ $-h/2-b_s$ (in.)*
A	4.82	6.88	8.34	-2.62
B	5.39	7.70	9.70	-2.30
C	7.25	10.34	11.84	+1.59

*Value indicates that critical section is closer to face of support than location specified in ACI 318-05.

Note: 1 in. = 25.4 mm.

tom of the unit; the second column lists the distance to the center of the support per Yang's model. The third column lists the distance x_{cp} from the critical section (per Yang's model) to the end of the slab (as determined in most of the shear tests by a given supplier). The fourth column lists the difference between the distance to the critical section from the face of the support (per Yang's model) and the distance of $h/2$ permitted by ACI 318-05. In the last column, the quantity b_s is half the width of the bearing pad supporting the end of the slab.

The derivation of the h_{cp} values of Table 3 requires some additional explanation. For units from both Suppliers A and B, the distance from the narrowest point on the web to the bottom of the unit differs for interior and exterior webs. Further, while the units from Supplier A had six interior webs, those from Supplier B had only three. The h_{cp} value of Table 3 is a weighted value calculated as the sum of the thickness of each web times the h_{cp} value for that web, divided by the total web thickness for that unit. For the tapered outside webs, where the least thickness is at the top of the web, the h_{cp} value was limited to half the member depth.

Correlation of the results in Table 3 with the measured shear strengths of Table 2 suggests that the products of Supplier C performed closer to ACI 318-05 web-shear strength expectations than those of Suppliers A and B. This is because the actual location of the critical section for web-shear cracking for the products of Supplier C was in closer agreement with the ACI 318 specified location. This concept can be expanded and used to examine when the products of a given supplier are likely to have critical sections, located per Yang's model, in approximately the same position as that specified by ACI 318-05. Supplier A, for example, currently provides units with 2-in.-depth (50 mm) increments up to 12 in. (300 mm). For the 12-in.-thick (300 mm) unit, the h_{cp} distance is almost the same as that for the 16-in.-deep (410 mm) unit, but the 12-in.-deep (300 mm) units are supported on 2-in.-wide (50 mm) rather than 3-in.-wide (75 mm) bearings for testing. Thus, the corresponding value in the fifth column of Table 3 for the 12-in.-deep (300 mm) unit is 0.12 in. (3 mm). Much better agreement with the V_{cw} values calculated per ACI 318-05 is to be expected for the 12-in.-deep (300 mm) unit than for the 16-in.-deep (410 mm) unit.

Adjustment of the critical section location (per the Yang's model concept) can explain some of the differences in V_{test}/V_{cw} for different suppliers. It does not explain the differences in those ratios for the products of Suppliers A and B, however. The h_{cp} values for the interior webs of the units for those two suppliers are essentially the same, but there are significant

differences in the average of the V_{test}/V_{cw} ratios for the deepest units from the two suppliers. It is reasonable to conclude that other factors can contribute to the ratio differences, such as the location of the prestressing steel within the unit and the method by which the unit is manufactured. For the units provided by Supplier A, there was one strand located below each web, except for the first two interior webs (where there were two strands). Failure of all webs simultaneously is to be expected. For the units provided by Supplier B, there was one $1/2$ -in.-diameter (13 mm) strand below each exterior web, one $1/2$ -in.-diameter (13 mm) and two 0.6-in.-diameter (15 mm) strands below the first two interior webs, and two $1/2$ -in.-diameter (13 mm) strands below the central web. In addition, there was a $1/2$ -in.-diameter (13 mm) strand in the top flange above each of the first interior webs. The more highly prestressed first interior webs are likely to have a substantially greater resistance to cracking than the exterior webs. The first interior webs are also able to provide an enhanced strength for the unit, compared with the strength calculated on average properties.

CONCLUSION

Tests of hollow-core units with depths greater than 12.5 in. (320 mm) have shown that web-shear strengths can be less than strengths computed using Eq. (11-12) of ACI 318-05 coupled with a critical section (located at $h/2$ from the face of the support per Section 11.1.3.2 of ACI 318-05). Analysis of European results for tests on units with depths to 16 in. (410 mm) has suggested that the location for the critical section for evaluation of web-shear strengths should be a function of unit geometry. While the available U.S. test data are consistent with that European finding, there also seems to be other factors affecting web-shear strengths that need to be identified before any code changes are proposed.

It is highly desirable that additional experimental investigations of the web-shear strength of hollow-core units be undertaken. A reduction in web-shear strength from the capacity predicted by ACI 318-05 seems warranted for hollow-core members deeper than 12.5 in. (320 mm). With such a reduction, web shear will control the design of deep hollow-core sections more often. Shear enhancement techniques¹⁶ are available that will still allow the hollow-core sections to be used in most applications without requiring the use of shear reinforcement.

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APPENDIX—NOTATION

- A_{ps} = area of prestressing strand
 b_w = total width of web
 d = distance from extreme compression fiber to the centroid of longitudinal tension reinforcement
 d_p = distance from extreme compression fiber to the centroid of prestressing steel
 f'_c = specified compressive strength of concrete
 f_{pc} = compressive stress in the concrete at the centroid of the cross section
 f_{ps} = steel stress before release
 h = overall thickness of the hollow-core unit including any composite topping
 V_c = nominal shear strength provided by concrete
 V_{cw} = computed shear for web-shear cracking per Eq. (11-12) of ACI 318-05 with the critical section taken at $h/2$ from the face of the support and the prestress effective at the critical section reduced to that which could be developed for a transfer length of 50 diameters from the end of the unit as placed in the test setup
 V_p = vertical component of effective prestress force
 V_{test} = shear acting on critical section when failure occurred
 V_u = factored shear force at the section