Shear Tests of Extruded Hollow-Core Slabs

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Through the years, hollow-core slabs have undergone extensive tests to verify their compliance with the provisions of the ACI Building Code. The advent of the 1971 ACI Code brought with it a number of provisions which were markedly different from prior codes to the extent that an updated, comprehensive testing program for hollow-core slabs seemed in order.

The specific code area containing the most ambiguous requirement was the section dealing with minimum web reinforcement. Also, since several hollow-core extruded slab systems are made with zero slump concrete, there is an additional reason to examine their characteristics experimentally since the shear strength provisions of the ACI Code were developed from tests of wet cast concrete.

While this investigation was originally developed to provide proprietary information, certain observations were deemed appropriate for publication.

First, dry cast, extruded hollow-core slabs depend on compaction of a zero slump concrete to achieve a monolithic cross section. Too little data on full scale tests of such slabs is available to the profession.

Secondly, shear tests also provide information on the bond strength of the prestressing strands. The Commentary to the ACI Code[1] specifically cautions the user on the applicability of development length equations for no-slump concrete.

Considerable discussion[2-7] has also been generated on development length in general, including some recommendations for increased development length requirements. The tests described herein provide data relative to development length.

Because of the differences in production techniques of various extruded hollow-core slabs, it is necessary to state that the test specimens were Spancrete, produced in the Waukesha, Wisconsin plant owned by Spancrete Industries. Shear and bond strength are dependent on the degree of concrete compaction and concrete mixes which in turn relate to the machine used to produce the slab. The applicability of these results to extruded slabs produced by other systems is dependent on the similarity of those factors.

Synopsis

The provisions in the ACI Code for shear in prestressed concrete were developed from tests on prestressed girder sections. Their applicability to prestressed hollow-core slabs has been inferred.

This paper reports the observations from a study of shear in dry cast, extruded hollow-core slabs. The basic conclusion was that these slabs do indeed meet the criteria of the ACI Code.

Additionally, in the study of web shear, the bond length of prestressing strands was severely tested with the conclusion that the present ACI Code (318-83) development length provisions are conservative.

OBJECTIVE

The objective of this investigation was to determine the applicability of the ACI Code provisions for prestressed concrete to the design of hollow-core slabs and specifically, to determine whether the web reinforcement exemption for slabs is valid for zero slump, extruded, hollow-core slabs.

FACTORS AFFECTING SHEAR STRENGTH

Prior to developing a detailed discussion of shear strength of hollow-core slabs and the factors relative thereto, it is essential to define certain basic criteria on failures.

Any slab which is overloaded to failure in an actual structure would ideally fail in a ductile flexural mode. Such a failure would be preceded by cracking and deflection; thus, giving ample warning of collapse.

Clearly, not all failures will be of this mode due to variations in loading conditions. While a flexural failure is most desirable, certain loading conditions may force failure into a shear mode. This type of failure is not as desirable because it can be a brittle type of failure and give little warning of imminent collapse.

However, in prestressed concrete slabs, other modes of failure could occur prior to either flexural or shear failures. Examples might include premature bond failure of the strands and transverse splitting due to nonuniform bearing. Such failures are simply not tolerable. Extremely low ultimate loads will result and the philosophy of design assumes that these types of failures will not exist.

As presented in Section 11.4 of the ACI Code (318-83), the shear equations for prestressed concrete are:

$$V_c = \left(0.6\sqrt{f'_c} + \frac{700\,V_u d}{M_u} \right) b_w \mathrm{d} \quad (11\text{-}10)$$

$$V_{ci} = \left(0.6\sqrt{f'_c}\, b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \right) \quad (11\text{-}11)$$

$$V_{cw} = (3.5\sqrt{f'_c} + 0.3 f_{pc})\, b_w d + V_p \quad (11\text{-}13)$$

Actual shear forces must not exceed ϕV_c.

Eq. (11-10) may be called an "approximate" equation. Its use is limited to sections in which the prestress force is greater than 40 percent of the tensile strength of the prestressing steel and it is intended to give conservative results.

When more accuracy is desired, Eq. (11-10) may be replaced by Eqs. (11-11) and (11-13). Both equations must be evaluated and the shear strength is the lesser value obtained from these equations. V_{ci} [Eq. (11-11)] is the ultimate shear strength based on the failure mode in which a flexural crack ultimately becomes inclined and results in a somewhat ductile flexure-shear failure.

V_{cw} [Eq. (11-13)] is the ultimate shear strength based on a sudden diagonal crack in the web of the member, generally occurring near the support. Web shear failures should be avoided because of their brittle nature. In general, web shear will control a design near a support where the strands are still being developed in bond while flexure-shear becomes more critical at sections some distance from the support.

In order to develop a comprehensive research program, it is essential to recognize the variables which affect the behavior of the element being tested. The ACI shear equations clearly show those parameters which principally affect shear strength. The important variables in these equations are the amount of prestress force, concrete strength, loading condition, and span length.

TEST PROGRAM

The variables studied in this program were the loading conditions, span length and amount of prestress. Due to the fact that extruded hollow-core slabs are manufactured under controlled conditions with a standard mix design, concrete strength was not considered as a variable. In order to obtain a shear failure, concentrated loads were placed near the supports with the distance from the support being varied to check the variation in shear strength for different load positions along the span.

Even though the predicted ultimate concrete shear strength for these tests was insignificantly affected by the span length with two-point loading, two slabs were tested on a longer span to check

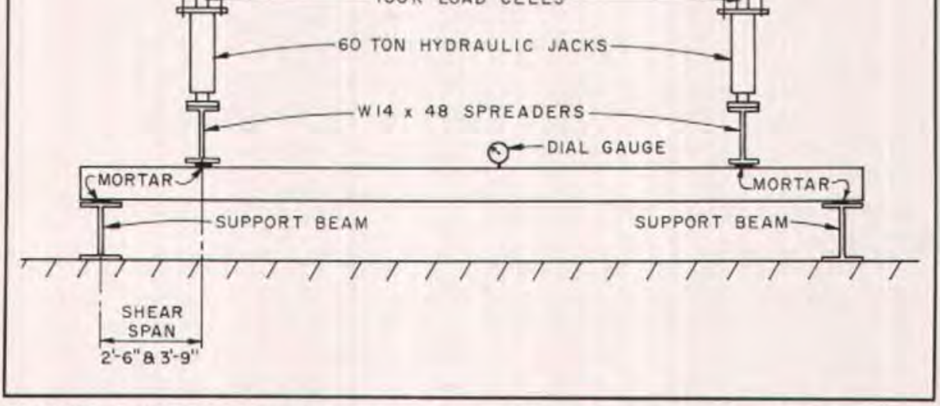


Fig. 1. Test setup for determining shear strength of hollow-core slab specimens.

Table 1. Summary of test specimen prestress levels.

Designation	Stranding	Slab thickness, in.	Calculated loss, percent
8414	14 - ¼ in., 250 ksi	8	17.2
8512	12 - 5⁄16 in., 250 ksi	8	17.7
8614	14 - ⅜ in., 250 ksi	8	21.1
10414	14 - ¼ in., 250 ksi	10	16.7
10512	12 - 5⁄16 in., 250 ksi	10	17
10614	14 - ⅜ in., 250 ksi	10	19.9
10620	20 - ⅜ in., 250 ksi*	10	24

*Ten strands are debonded 2 ft from each end.
Note: 1 ft = 0.305 m, 1 in. = 25.4 mm, 1 ksi = 6.89 MPa.

the effect of the span length. One slab was also tested with six concentrated loads (simulating a uniform load) to check the effects of this type of loading condition.

Finally, the amount of prestress was varied to determine if the contribution of the prestress force in increasing the concrete shear capacity would corroborate with the ACI Code equations.

MATERIALS AND TEST SETUP

Three series of 8 in. (200 mm) slabs and four series of 10 in. (254 mm) slabs were tested for shear strength characteristics. All slabs were expected to result in a shear failure prior to a flexural failure. The levels of prestressing used are summarized in Table 1. In all cases, the initial prestress was 65 percent of ultimate, the standard level for the slab manufacturer.

The tests on the three 8 in. (200 mm) series and three of the 10 in. (254 mm) series were conducted on a 15 ft (4.57 m) span loaded with two concentrated loads as shown in Fig. 1. The shear spans used were 2 ft 6 in. (0.76 m) (an arbitrary length) and 3 ft 9 in. (1.14 m) (the maximum length for which shear would still control the mode of failure).

Testing of the fourth 10 in. (254 mm) series was conducted on a 34 ft (10.4 m) span. Two of the tests had the loading arrangement shown in Fig. 1 except that 3 and 6 ft (0.91 and 1.83 m) shear spans were used and one test had six equally spaced loads, simulating a uniform load.

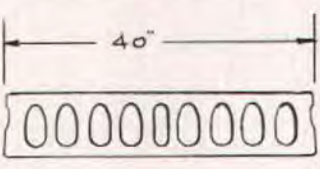
Line loads were applied with 60 ton hydraulic jacks and steel distributing beams. Loads were measured using 100 kip (450 kN) strain gage calibrated load cells and deflections were measured with 0.001 in. (0.025 mm) dial indicators. The supports were steel wide flange sections with a 3 in. (76 mm) wide bed of grout used to provide a uniform bearing surface for the plank.

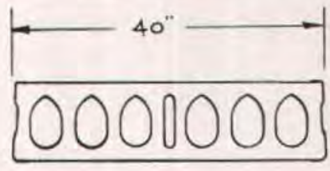
Section properties of the series tested are given in Table 2. Cylinder tests showed the actual concrete strength to be about 6000 psi (41.4 MPa) and this value was used in all theoretical calculations.

TYPES OF FAILURES

Two distinct types of shear failures were observed in this test program. Fig. 2 shows an example of flexure-shear cracking which was the most common failure observed. The crack started as a vertical flexure crack and then became inclined as the shear stresses increased.

Table 2. Cross section characteristics of hollow-core slabs tested.





8 in. Slab	10 in. Slab
A = 218 in.^{2}	A = 257 in.^{2}
I = 1515 in.^{4}	I = 2933 in.^{4}
b_w = 17.0 in.	b_w = 13.3 in.
S_b = 380.6 in.^{3}	S_b = 565.2 in.^{3}
Weight = 64 psf	Weight = 75 psf

Note: 1 in. = 25.4 mm, 1 in.^{2} = 645 mm^2,
1 in.^{4} = 416000 mm^4, 1 in.^{3} = 16400 mm^3, 1 psf = 47.9 Pa.

The section shown was loaded 3 ft 9 in. (1.14 m) from each support and was prestressed with 12 - 5/16 in. (8 mm) strands.

The second type of shear failure was the web shear failure which was generally sudden and destructive as shown in Fig. 3. The section shown in Fig. 3 was loaded 2 ft 6 in. (0.76 m) from the support and was prestressed with 14 - 3/8 in. (9.5 mm) strands.

The definition of ultimate capacity used in this test program was collapse. While web shear ultimate capacity can only be defined by collapse because of the type of failure, ultimate flexure-shear capacity is defined by the ACI Code as the formation of the inclined crack. However, sufficient warning of failure is given with the flexure-shear failure and the excess capacity beyond inclined cracking was found to be up to 29 percent.

TEST RESULTS

The test results are summarized in Tables 3 and 4 and shown graphically in relation to the ACI Code shear equations in Figs. 4, 5, and 6. One important point to note is that ACI Eq. (11-10) allows higher shear capacities than the more "exact" equations for the 8414 and 10414 series.

Even though all the tests were successful in terms of Eq. (11-10), judgment should be used when applying this equation since it is not conservative in all cases. The second important point to note is that only one slab has a ratio of "tested" capacity to "predicted" capacity of less than 1.0.

The ACI Code shear equations are set up to be used in conjunction with a ϕ factor. This ϕ factor (0.85) is an understrength factor used to insure that all shear test data, which was the basis for the Code equations, would be applicable. Not all of the original ACI tests satisfied the basic ACI equations, but every test satisfied 0.85 times the ACI equations. Since no ϕ factor can be used in a test program, any test with a ratio of tested strength to predicated strength greater than 0.85 would strictly satisfy the ACI Code shear equations. One test in this program is in this category.

One of the limitations of the test procedure was the fact that the test equip-

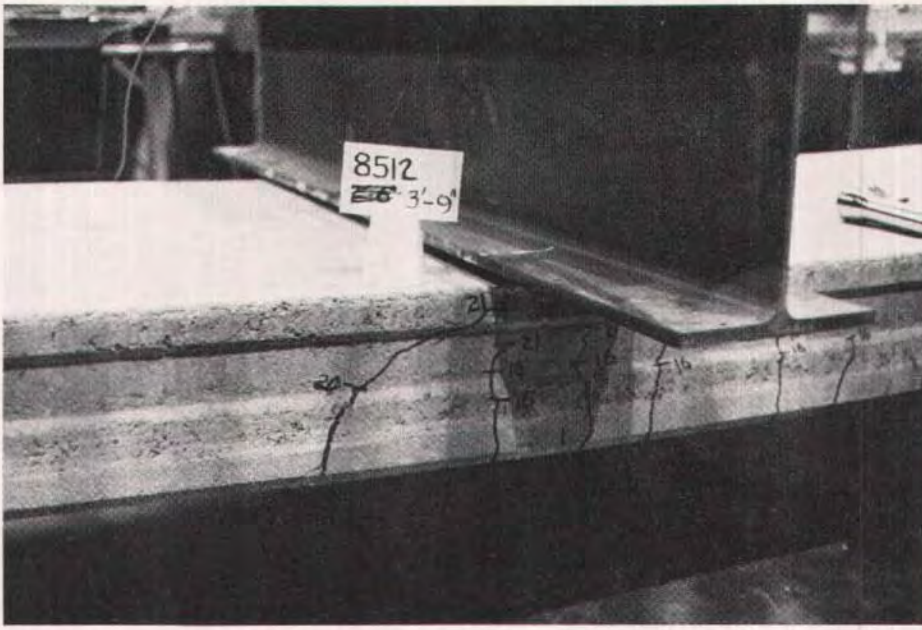


Fig. 2. Inclined shear cracking of hollow-core specimen.

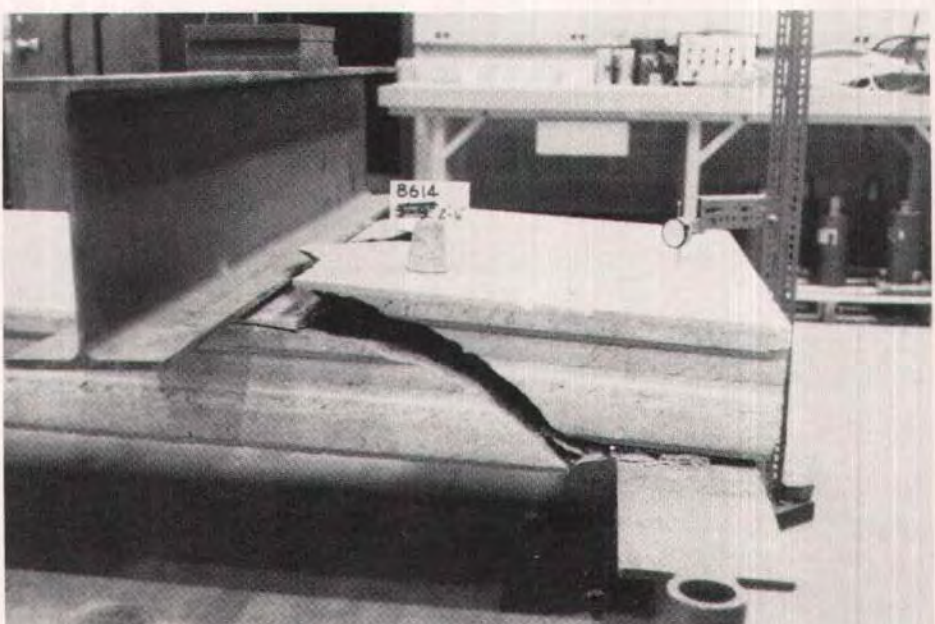


Fig. 3. Web shear failure of hollow-core specimen.

Table 3. Summary of shear test (8 in. slabs).

Series	Span length, ft	Shear span, ft — in.	Predicted shear		Test shear		Ratio test to predicted	Mode of failure
			Eq. (11-10), kips	Eq. (11-11) or (11-13), kips	Inclined cracking, kips	Ultimate, kips		
8414	15	2 — 6	26.04	21.8	—	32.4	—	*
8414	15	2 — 6	26.04	21.8	—	32	—	*
8414	15	2 — 6	26.04	21.8	24.1	27.6	1.27	Flexure Shear
8414	15	3 — 9	19.25	16.23	—	19.6	—	*
8414	15	3 — 9	19.25	16.23	18	19.8	1.22	Flexure Shear
8414	15	3 — 9	19.25	16.23	18	19.8	1.22	Flexure Shear
8414	15	3 — 9	19.25	16.23	18	25.1	1.55	Flexure Shear
8512	15	2 — 6	25.9	25.8	34	38.1	1.48	Flexure Shear
8512	15	2 — 6	25.9	25.8	—	35	1.36	Web Shear
8512	15	3 — 9	19.17	18.8	24	25.7	1.37	Flexure Shear
8512	15	3 — 9	19.17	18.8	—	25.9	1.38	Web Shear
8512	15	3 — 9	19.17	18.8	20	21.6	1.15	Flexure Shear
8512	15	3 — 9	19.17	18.8	16	21.1	1.12	Flexure Shear
8614	15	2 — 6	25.8	34.6	—	61.1	1.77	Web Shear
8614	15	2 — 6	25.8	34.6	—	56	1.62	Web Shear
8614	15	2 — 6	25.8	34.6	—	30	.85	Web Shear
8614	15	3 — 9	19.1	24.8	34	40	1.61	Flexure Shear
8614	15	3 — 9	19.1	24.8	33	38.8	1.56	Flexure Shear
8614	15	3 — 9	19.1	24.8	28.1	32.9	1.33	Flexure Shear

*Deflection limited test.

Notes: 1 — V_c based on 6000 psi concrete.

2 — Predicted shear does not include ϕ factor.

1 ft = 0.305 m, 1 in. = 25.4 mm, 1 kip = 4.5 kN, 1000 psi = 6.89 MPa.

Table 4. Summary of shear test (10. in slabs).

Series	Span length, ft	Shear span, ft — in.	Predicted shear		Test shear		Ratio test to predicted	Mode of failure
			Eq. (11-10), kips	Eq. (11-11) or (11-13), kips	Inclined cracking, kips	Ultimate, kips		
10414	15	2 — 6	32.2	28.3	29	29	1.02	Flexure Shear
10414	15	2 — 6	32.2	28.3	—	36	1.27	Flexure
10414	15	3 — 9	23.1	20.5	—	26.1	1.27	Flexure
10414	15	3 — 9	23.1	20.5	—	25.6	1.25	Flexure
10512	15	2 — 6	32.1	33.6	—	47.7	1.42	Flexure
10512	15	3 — 9	23.0	24.1	—	30.9	1.28	Flexure
10512	15	3 — 9	23.0	24.1	—	31.7	1.32	Flexure
10614	15	2 — 6	31.9	45.4	—	56	1.23	Web Shear
10614	15	2 — 6	31.9	45.4	—	57.9	1.28	Web Shear
10614	15	3 — 9	22.9	31.9	44	50.3	1.58	Flexure Shear
10614	15	3 — 9	22.9	31.9	—	41.8	1.31	Flexure
10620	34	3 — 0	27.2	37.2	—	43.1	1.16	Web Shear
10620	34	6 — 0	18.7	27.1	31.6	33.6	1.24	Flexure Shear
10620	34	†	20.2	15.9	—	11.9	—	*

*Deflection limited test.

†Six-point loading.

Notes: 1 — V_c based on 6000 psi concrete.

2 — Predicted shear does not include ϕ factor.

1 ft = 0.305 m, 1 in. = 25.4 mm, 1 kip = 4.5 kN, 1000 psi = 6.89 MPa.

ment was limited in cases where deflections were large. Where the mode of failure in Tables 3 or 4 is footnoted with an asterisk, the deflection exceeded the capability of the equipment. Where possible, the slab was loaded at least to the ACI Code shear values. The one exception was the 10620 series loaded with six concentrated loads on a 34 ft (10.4 m) span. At a load of 6 kips (27 kN) at each jack, the deflection at midspan was about 10 in. (254 mm) and the test had to be stopped.

Although the ACI Code ultimate shear stress was not reached, the performance at working loads was acceptable with a deflection of 0.922 in. (23 mm) (*L*/442) and with no flexural cracking. The load at which the test was stopped was 2.3 times the allowable working load (controlled by web shear). Since failure was not reached, no ratio of "tested" to "predicted" strength is given.

The two types of shear failures can be shown to be generally related to the amount of prestressing force and the shear span. With low amounts of prestress and a longer shear span, flexural stresses are more dominant and the flexure-shear failure results. With heavy prestress and a short shear span, the flexural stresses do not have as great an effect, hence the web shear failure occurs.

CONCLUSIONS

The objective of this investigation was to determine the applicability of the ACI Code shear equations for prestressed concrete to zero slump, ex-

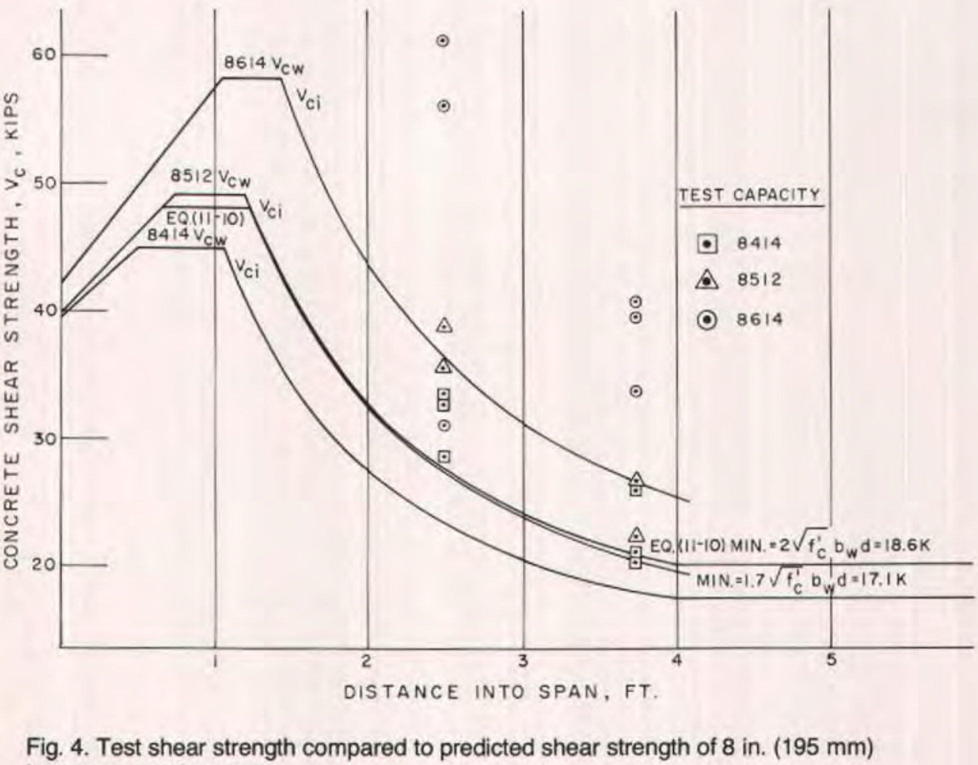


Fig. 4. Test shear strength compared to predicted shear strength of 8 in. (195 mm) hollow-core slab series.

truded hollow-core slabs. The test results did indeed show that the actual shear capacity of these slabs was in excess of the capacity predicted by the use of the ACI equations.

Even though no web reinforcement was used in the slabs, satisfactory working load and ultimate behavior were observed. Therefore, it can be concluded that the minimum shear reinforcement exemption allowed by the ACI Code for slabs is valid for these prestressed hollow-core slabs.

Finally, the loading conditions used in this test program severely tested the strand bond strength. In the worst case, the strands had only 2 ft 6 in. (0.76 m) plus the length beyond bearing to develop their capacity. In no case was a test limited by bond failure even though the theoretical ultimate flexural capacity of the section assuming full strand development was exceeded in several cases.

Table 5 was developed to compare the theoretical flexural capacity with the applied moments at the point of load application for the cases where ultimate flexural capacity was exceeded. As illustrated in that table, ultimate moment capacities were exceeded even where the available development length was about one-half that required.

DESIGN RECOMMENDATIONS

1. Even though no test specimen in this test program failed to meet the predicted shear strength based on Eq. (11-10), it was found that for the lightly prestressed members, the simplified

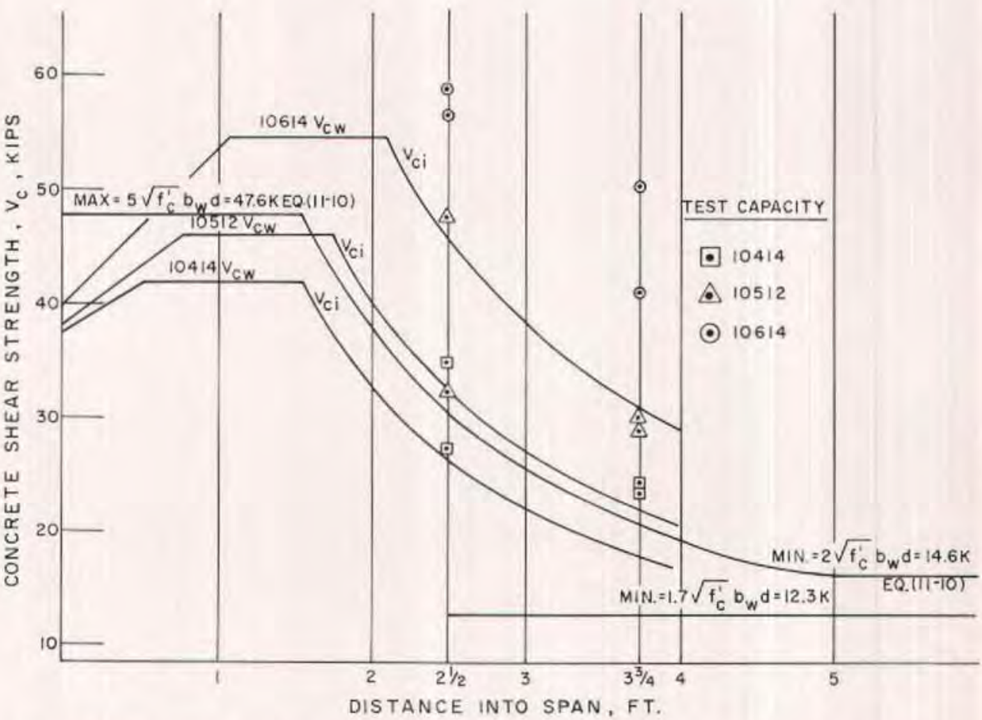


Fig. 5. Test shear strength compared to predicted shear strength of 10 in. (254 mm) hollow-core slab series.

Table 5. Comparison of theoretical flexural capacity with applied moments at point of load application.

Series	Shear span, ft — in.	Test moment in shear span, ft-kip/ft	Theoretical ultimate moment, ft-kip/ft	ACI Code l_d, in.	Available l_d, in.
8414	2 — 6	25.3	21.0	40	36
8414	2 — 6	25.0	21.0	40	36
8512	2 — 6	29.5	28.0	50	36
8614	2 — 6	46.8	38.4	61.5	36
8614	2 — 6	42.8	38.4	61.5	36
8614	3 — 9	46.4	38.4	61.5	51
8614	3 — 9	44.9	38.4	61.5	51

Note: 1 ft = 0.305 m, 1 in. = 25.4 mm, 1 ft-kip = 1.37 kN-m.

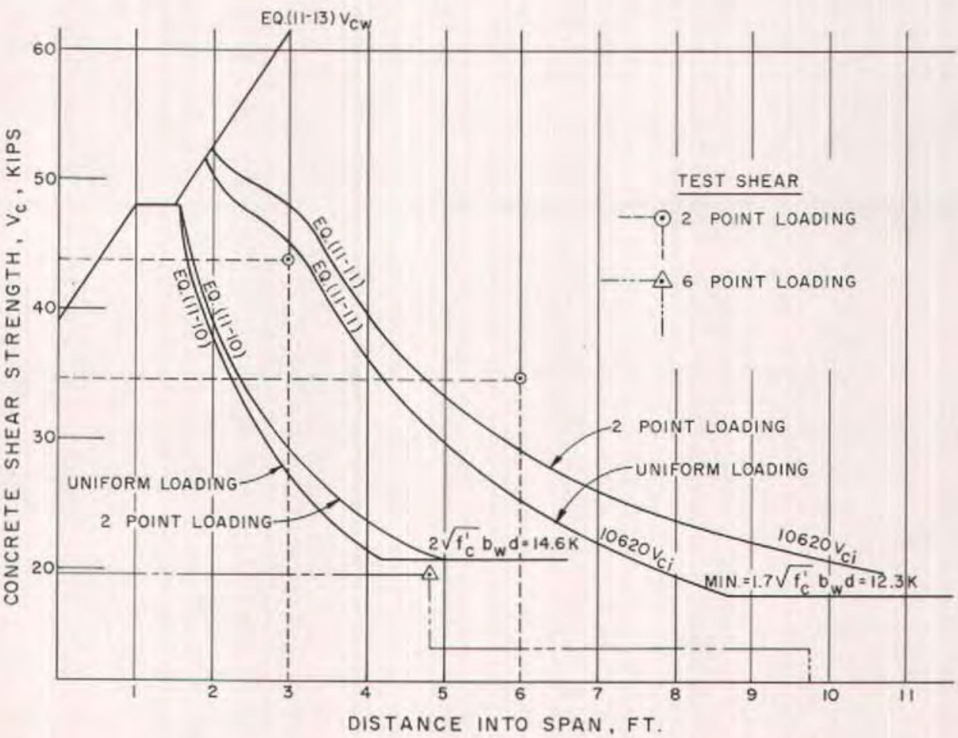


Fig. 6. Test shear strength for uniform and two-point loading of 34 ft (10 m) span hollow-core slabs.

equation predicts higher shear strength than the more exact equations. Consideration should be given to limiting the use of Eq. (11-10) to members with reinforcing ratios greater than 0.2 percent.

2. With the bond strength developed in these test specimens, the ACI Code development length criteria conservatively predicted strand stress potential. A significant increase in required development length does not seem justified based on these tests.

3. Related to Item 2, quality control procedures must assure proper bond of the strands particularly in the dry cast, extruded slab systems. The test slabs in this program exhibited free end slip of less than 1/16 in. (1.6 mm). Excessive free end slip will definitely increase the required development length and must be considered in strength evaluation. If the effects of free end slip are considered in design, again, no changes would be required in the basic ACI Code development length requirements which are based on negligible free end slip.

ACKNOWLEDGMENT

This investigation was conducted under the auspices of the Spancrete Manufacturer's Association as a part of their continuing research program. Test work was performed at the Structural Engineering Laboratory of the University of Wisconsin-Milwaukee under the direction and control of the authors.

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NOTE: Discussion of this paper is invited. Please submit your comments to PCI Headquarters by November 1, 1985.

APPENDIX A — SAMPLE CALCULATIONS

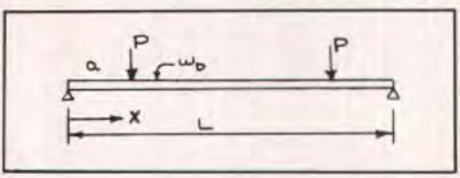
Computation of the allowable shear strength for hollow-core slabs using the ACI Code equations is not difficult, but rather tedious. The charts given in the PCI Design Handbook[8] cannot always be used because they only apply to uniformly loaded simple spans which is also the condition usually covered by the manufacturer's load tables.

A common loading condition often encountered in residential buildings involves the use of wood frame construction above the precast member. With this type of framing, the slabs are loaded with line and concentrated loads. A manual calculation of the allowable shear strength is then required.

Allowable shear strength calculations for the two-point loading used in the test program are presented here. For more complex loading conditions, the equation for V_{ci} becomes proportionately more complex and cannot be derived in general terms. It is then more reasonable to use the actual values of dead load shear, live load shear, and moment, and to use a computer program to find the critical point for shear.

If a computer program is not available and the computation of V_{ci} is required, a reasonable sequence of calculations can be made to minimize the number of points to be checked. The starting point would be at $h/2$ from the support, keeping in mind that the strands must be developed over 50 diameters. If the shear checks at that point, other critical points will be at or near concentrated load points. Because of the nature of the curve for V_{ci}, points between a concentrated load and the support should also be checked. It must be realized that there is no rapid, easy method for finding the critical point for shear.

Calculations for Determining Shear Strength



Series: 8614 (14 - ⅜ in. diameter, 250 ksi strands)
Loading: Two-point load as shown in the diagram.

Eq. (11-10)

$$V_c = \left(0.6\ \sqrt{f'_c} + 700\ \frac{V_u d}{M_u}\right) b_w d$$

$$= \left(0.6\ \sqrt{6000} + 700\,\frac{P\,d}{P\,X}\right) b_w\, d \text{ for } X < a$$

$$= \left(46.5 + \frac{(700)(7.0625)}{12\ X}\right)(17)(7.0625)$$

$$= 5580 + \frac{49463}{X}$$

Note that the symbol X is expressed in feet. Also note that self weight is ignored since in this case the effect is less than 0.5 percent.

Eq. (11-11)

$$V_{ci} = 0.6\sqrt{f'_c}\, b_w d + V_d + \frac{V_i M_{cr}}{M_{max}}$$

$$V_d = \frac{w_D L}{2} - w_D$$

$$\frac{V_i}{M_{max}} = \frac{P}{PX} = \frac{1}{X}$$

Define $M_{cr} = M'_{cr} - M_D$

$$= \left(\frac{I}{y_t}\right)\left(6\sqrt{f'_c} + f_{pe}\right) - \left(\frac{w_D L}{2} X - \frac{w_D X^2}{2}\right)$$

$$V_{ci} = 0.6\sqrt{f'_c}\, b_w d + \frac{w_D L}{2} - w_D X + \frac{M'_{cr}}{X} - \left(\frac{w_D L}{2} - \frac{w_D X}{2}\right)$$

$$= 0.6\sqrt{f'_c}\, b_w d - \frac{w_D X}{2} + \frac{M'_{cr}}{X}$$

(Note that for these tests the term $w_D X/2$ is insignificant.)

$$V_{ci} = 0.6\sqrt{6000}\,(17)(7.0625) + \frac{1}{12}\left(\frac{1515}{3.98}\right)\Bigg[6\sqrt{6000} + (0.65)(0.789)(14)(20{,}000)\left(\frac{1}{218} + \frac{(3.04)(3.98)}{1515}\right)\Bigg]$$

$$V_{ci} = 5580 + \frac{72016}{X}$$

where X is expressed in feet.

Eq. (11-13)

$$V_{cw} = (3.5\sqrt{f'_c} + 0.3 f_{pc})\, b_w d$$

$$= \left(3.5\sqrt{6000} + \frac{(0.3)(0.65)(0.789)(20{,}000)(14)}{218}\right)(17)(7.0625)$$

$$= 32550 + 23725$$

V_{cw} = 56275 lb (253 kN)
(beyond transfer length)

$$= 32550 + 23725\,\frac{12X + 6}{(50)(0.375)}$$

(within transfer length)

$$V_{cw} = 40142 + 15184X$$

Evaluating the above equations along the span results in a test shear capacity (see Table A1). In this table only the controlling values are given.

Table A1. Various controlling distances in determining shear strength.

X, ft	V_c, lb	V_{ci}, lb	V_{cw}, lb
0.333	45198		45198
0.5	46500*		47734
1.0	46500*		55326
1.5	38555	53590	
2.0	30312	41588	
2.5	25765	34586	
3.0	22068	29585	
3.5	19712	26156	

*Controlled by $5\sqrt{f'_c}\ b_w d$.
Note: 1 ft = 0.305 m, 1 lb = 4.5 N.

APPENDIX B — NOTATION

A = gross concrete area
b_w = net web width resisting shear
d = distance from extreme compression fiber to steel centroid
f'_c = concrete compressive strength
f_{pc} = compressive stress at centroid of cross section after prestress losses have occurred
f_{pe} = compressive stress in concrete due to prestress forces after losses only at the extreme fiber where tension is caused by external loads
I = moment of inertia of cross section
M_{cr} = moment causing cracking due to externally applied loads
M'_{cr} = moment causing cracking due to loads including self weight
M_d = moment due to unfactored self weight at section being considered
$M_{max} = M_u - M_d$
M_u = total factored moment at section being considered
S_b = section modulus with respect to bottom fiber
V_c = nominal concrete shear strength
V_{ci} = nominal concrete shear strength in inclined shear mode
V_{cw} = nominal concrete shear strength in web shear mode
V_d = shear force due to unfactored self weight at section being considered
$V_i = V_u - V_d$
V_p = vertical component of effective prestress force at section being considered
V_u = total factored shear force
X = controlling distance from end support in determining shear strength (expressed in feet)
w_D = unfactored self weight
y_t = distance from centroid of cross section to extreme tension fiber due to applied loads
ϕ = ACI strength reduction factor

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