CONCRETE TECHNOLOGY ASSOCIATES

TECHNICAL BULLETIN 78-B1

APRIL 1978

1123 PORT OF TACOMA ROAD / TACOMA, WASHINGTON 98421 / (206) 383-3545

SHEAR STRENGTH OF HOLLOW CORE MEMBERS

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SYNOPSIS

SHEAR STRENGTH OF HOLLOW CORE MEMBERS

Producers of prestressed hollow core units often penalize themselves by adopting unnecessarily conservative procedures for calculating the shear capacities of their products. This bulletin presents methods which permit an engineer to more accurately predict the shear capacity of hollow core units and hence to more fully utilize their true shear capacity in design.

The design recommendations outlined in the USERS GUIDE conform to the philosophy and format of Section 11.4 of ACI 318-77, which deals with the shear strength provided by the concrete for prestressed members. The recommendations are substantiated by tests of factory produced hollow core units of varying design and prestress level, as reported in PART II.

It is anticipated that the recommendations of this bulletin will be most significant in applications involving members with high uniform loads or heavy concentrated moving loads, since such members are often controlled by shear in design. Substantial savings can be achieved due to lowered prestress requirements or increased span-load capabilities. For this reason the bulletin should be particularly useful to marketing personnel in bidding jobs or in convincing engineers and building officials of the adequacy of specific designs.

CONTENTS

USERS GUIDE

INTRODUCTION

This bulletin presents recommendations for calculating the shear strength of hollow core members. These recommendations conform to procedures for calculating the shear strength provided by the concrete in prestressed members contained in Section 11.4 of ACI 318-77 which has been reproduced below. The purpose of the bulletin is to present methods for determing certain parameters in the design equations.

11.4 – Shear strength provided by concrete for prestressed members

11.4.1 – For members with effective prestress force not less than 40 percent of the tensile strength of flexural reinforcement, unless a more detailed calculation is made in accordance with Section 11.4.2,

$$V_c = \left(0.6\sqrt{f_c'} + 700\frac{V_u d}{M_u}\right) b_w d \qquad (11\text{-}10)$$

but V_c need not be taken less than $2\sqrt{f_c'}b_w d$ nor shall V_c be taken greater than $5\sqrt{f_c'}b_w d$ nor the value given in Section 11.4.3. The quantity $V_u d/M_u$ shall not be taken greater than 1.0, where M_u is factored moment occurring simultaneously with V_u at section considered. When applying Eq. (11-10), d in the term V_{ud}/M_u shall be the distance from extreme compression fiber to centroid of prestressed reinforcement.

11.4.2 – Shear strength V_c may be computed in accordance with Sections 11.4.2.1 and 11.4.2.2, where V_c shall be the lesser of V_{ci} or V_{cw}.

11.4.2.1 – Shear strength V_{ci} shall be computed by

$$V_{ci} = 0.6\sqrt{f_c'}b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \qquad (11\text{-}11)$$

but V_{ci} need not be taken less than $1.7\sqrt{f_c'}b_w d$, where

$$M_{cr} = (I/y_t)(6\sqrt{f_c'} + f_{pe} - f_d) \qquad (11\text{-}12)$$

and values of M_{max} and V_i shall be computed from the load combination causing maximum moment to occur at the section.

11.4.2.2 – Shear strength V_{cw} shall be computed by

$$V_{cw} = (3.5\sqrt{f_c'} + 0.3f_{pc})b_w d + V_p \qquad (11\text{-}13)$$

Alternatively, V_{cw} may be computed as the shear force corresponding to dead load plus live load that results in a principal tensile stress of $4\sqrt{f_c'}$ at centroidal axis of member, or at intersection of flange and web when centroidal axis is in the flange. In composite members, principal tensile stress shall be computed using the cross section that resists live load.

11.4.2.3 – In Eq. (11-11) and (11-13), d shall be the distance from extreme compression fiber to centroid of prestressed reinforcement or $0.8h$, whichever is greater.

11.4.3 – In a pretensioned member in which the section at a distance $h/2$ from face of support is closer to end of member than the transfer length of the prestressing tendons, the reduced prestress shall be considered when computing V_{cw}. This value of V_{cw} shall also be taken as the maximum limit for Eq. (11-10). Prestress force may be assumed to vary linearly from zero at end of tendon to a maximum at a distance from end of tendon equal to the transfer length, assumed to be 50 diameters for strand and 100 diameters for single wire.

NOTE: See NOTATION at the end of this bulletin for definitions of variables in the above equations.

DESIGN RECOMMENDATIONS

The value of b_w, the web width, should be taken as the minimum value, b^*_w, in all expressions and equations in Section 11.4 except in Equation 11-11. Equation 11-11 can be rewritten as follows:

$$V_{ci} = K\sqrt{f'_c}\, A_E + V_d + \frac{V_i M_{cr}}{M_{max}}$$

where

K = 0.6 unless a higher value is justified by tests

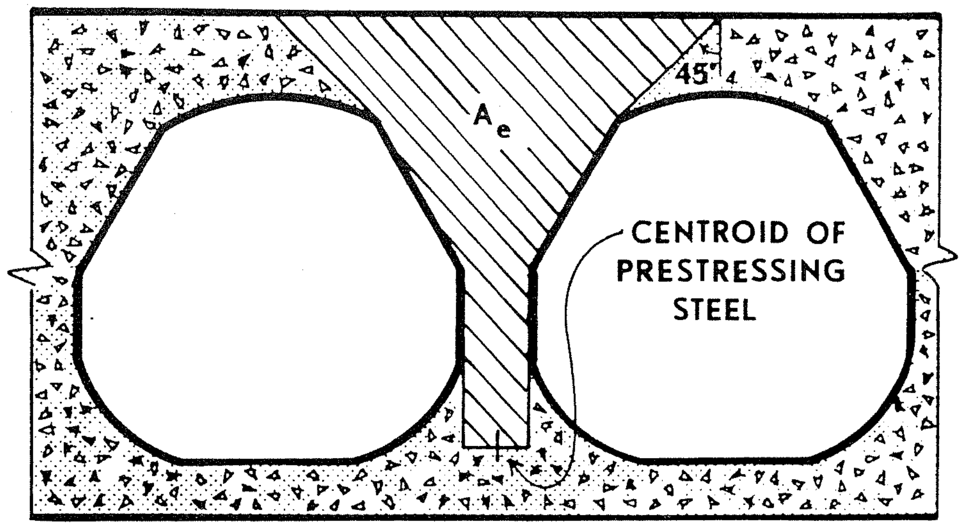
A_E = the effective shear area defined as follows:
A_E is the portion of the cross-section above the centroid of the prestressing steel enclosed by lines that follow the contour of the cross-section but never exceed a 45° angle with respect to the vertical.

The definition for A_E is illustrated in Figure 1 for 8 in. Spiroll and 12 ½ in. DyCore. Table 1 gives the effective shear areas for the products tested.

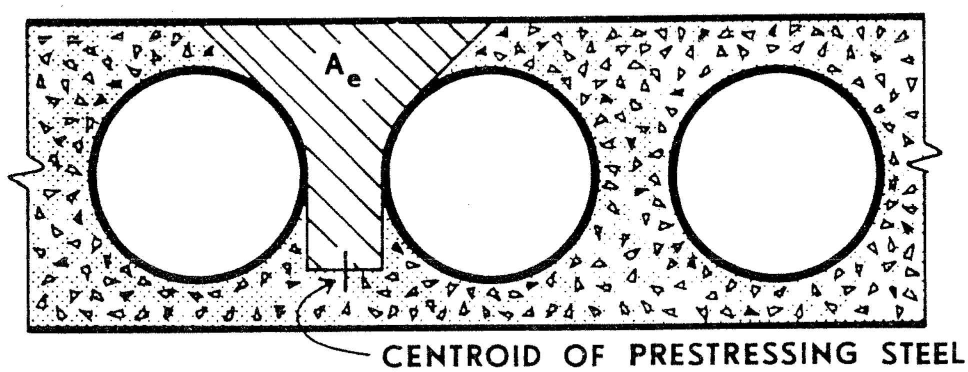
TABLE 1

EFFECTIVE SHEAR AREAS OF HOLLOW CORE PRODUCTS
(For 4 ft Wide Sections)

PRODUCT	DEPTH (inches)	$b^*_w d$	A_E	$\frac{A_E}{b^*_w d}$
DyCore	14 ½	108	268	2.47
	12 ½	91	178	1.96
	12	87	158	1.82
Spiroll	8	88	138	1.58
	6	64	103	1.62



12 1/2" DYCORE



8" SPIROLL

FIG. 1 – EXAMPLES OF EFFECTIVE SHEAR AREAS.

For Spiroll and DyCore where $h \leq 14\frac{1}{2}$ in., the following values of K have been established by tests and may be used in design:

$$K = 0.75 \qquad \frac{M_{max}}{V_i d} > 10.0$$

$$K = 1.0 \qquad 0 \leq \frac{M_{max}}{V_i d} \leq 10.0$$

As a service to its membership, the CTA laboratory will test other hollow core products at no expense to its member companies provided the interested company will supply the test specimens. The purpose of such tests would be to determine more favorable values of K. The results would then be furnished to all CTA members. In addition, the staff will upon request furnish information for hollow core produced in metric or SI units of measurement.

DESIGN EXAMPLE (in U.S. units)

Concrete Technology Corporation is currently designing DyCore planks for a processing facility for Washington Fish & Oyster Co. of Seattle, Washington. Because of the high live load requirements, shear is likely to control.

Loading	- live load	- 500 lb/ft^2
	- $2\frac{1}{2}$ in. non-structural topping	- 50 lb/ft^2
	- 6 in. insulation	- 12 lb/ft^2
Span	- 22 ft 6 in.	

Use 13 in. DyCore - $f'_c = 8000$ psi

SECTION PROPERTIES

$I = 6430\ in^4$	$Y_t = 7.25$ in.
$A = 308\ in^2$	$Z_t = 887\ in^3$
$d = 11.25$ in.	$A_E = 200\ in^2$
$b_w^* = 8.5$ in.	$W_d = 340$ lb/ft
$b = 48$ in.	

Design for Ultimate Moment

$$W_u = 1.4[4(50 + 12) + 340] + 1.7[4(500)]$$

$$= 4223 \text{ lb/ft} = 4.22 \text{ kips/ft}$$

$$M = \frac{4.22(22.5)^2}{8} = 267 \text{ k·ft}$$

or the required moment capacity is 3204 k·in.

Try 8 Strands

Using strain compatibility, $\phi M_u = 3208$ k·in. O.K.

NOTE: See CTA Technical Bulletin 75-B4 for discussion of ultimate moment based on strain compatibility.

Check Flexure-Shear Strength

Since the dead load and live load are both uniform loads, V_i/M_{max} is independent of the magnitude of the live load. At a point x-distance from either support,

$$V = W(\frac{\ell}{2} - x)$$

$$M = \frac{Wx(\ell - x)}{2}$$

$$\frac{V}{M} = \frac{\ell - 2x}{x(\ell - x)}$$

Substituting V/M for V_i/M_{max}:

$$V_{ci} = K\sqrt{f'_c}\, A_E + V_d + M_{cr}\frac{\ell - 2x}{x(\ell - x)}$$

where

$$K = 1.0 \quad \text{when} \quad \frac{M}{Vd} = \frac{\ell - 2x}{x(\ell - x)} \leq 10.0$$

$$K = 0.75 \quad \text{otherwise}$$

However, V_{ci} need not be less than $1.7\sqrt{f'_c}\, b_w^* d$.

For purposes of comparison let

$$V^*_{ci} = 0.6\sqrt{f'_c}\, b^*_w\, d + V_d + M_{cr} \frac{\ell - 2x}{x(\ell - x)}$$

x (ft)	1.0	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
M/Vd	1.12	2.36	3.78	5.44	7.47	10.06	13.62	19.00	28.80	53.30
V_{ci} (kips)	181.2	95.1	66.1	51.4	42.3	31.5	26.8	19.2	19.1	16.8
V^*_{ci} (kips)	168.5	82.4	53.1	38.7	29.6	23.4	18.5	14.7	14.4	14.4
V_u (kips)	50.3	45.4	40.5	35.6	30.7	25.8	20.9	15.9	11.0	6.13

For every value of x, V_{ci} is larger than V_u, therefore the design is adequate for shear. If, however, the shear is checked on the basis of V^*_{ci} it can be shown that two more strands would be necessary to satisfy the flexure shear constraint.

Check Web-Shear Strength

$$V_{cw} = (3.5\sqrt{f'_c} + 0.3\, f_{pc})\, b^*_w\, d + V_p$$

$$= \left[\frac{3.5\sqrt{8000}}{1000} + \frac{0.3 \times 8 \times 154 \times 0.153}{308}\right] 11.25 \times 8.5$$

$$= 47.5 \text{ kips}$$

Applying a strength reduction factor of 0.85,

$$V_{cw} = 40.4 \text{ kips}$$

At x = 3 ft, V_u = 40.5 kips. Therefore it is necessary to fill the voids with concrete to 3 ft from the support.

DESIGN EXAMPLE (in S.I. units)

Concrete Technology Corporation is currently designing DyCore planks for a processing facility for Washington Fish & Oyster Co. of Seattle, Washington. Because of the high live load requirements, shear is likely to control.

Loading	- live load	-	23940 Pa
	- 63.5 mm non-structural topping	-	2394 Pa
	- 152.4 mm insulation	-	574.6 Pa
Span	- 6.86 m		

Use 330 mm DyCore - $f'_c = 55.2$ MPa

SECTION PROPERTIES

$I = 2.68 \times 10^9$ mm^4	$Y_t = 184.15$ mm
$A = 198700$ mm^2	$Z_t = 1.45 \times 10^7$ mm^3
$d = 285.75$ mm	$A_E = 129030$ mm^2
$b^*_w = 215.9$ mm	$W_d = 4961$ N/m
$b = 1219.2$ mm	

Design for Ultimate Moment

$$W_u = 1.4[1.219(2394 + 574.6) + 4961] + 1.7[1.219(23940)]$$

$$= 61627 \text{ N/m}$$

$$M = \frac{61627(6.86)^2}{8} = 362518 \text{ N}\cdot\text{m}$$

or the required moment capapcity is 362518 N•m.

Try 8 Strands

Using strain compatibility, $\phi M_u = 362970$ N•m. O.K.

Note: See CTA Technical Bulletin 75-B4 for discussion of ultimate moment based on strain compatibility.

Check Flexure-Shear Strength

Since the dead load and live load are both uniform loads, V_i/M_{max} is independent of the magnitude of the live load. At a point x-distance from either support,

$$V = W(\frac{\ell}{2} - x)$$

$$M = \frac{Wx(\ell - x)}{2}$$

$$\frac{V}{M} = \frac{\ell - 2x}{x(\ell - x)}$$

Substituting V/M for V_i/M_{max}:

$$V_{ci} = K\sqrt{f'_c}\, A_E + V_d + M_{cr} \frac{\ell - 2x}{x(\ell - x)}$$

where

$$K = 0.083 \quad \text{when} \quad \frac{M}{Vd} = \frac{\ell - 2x}{x(\ell - x)} \leq 10.0$$

$$K = 0.062 \quad \text{otherwise}$$

However, V_{ci} need not be less than $0.141\sqrt{f'_c}\, b_w^* d$.

For purposes of comparison let

$$V_{ci}^* = 0.05\sqrt{f'_c}\, b_w^* d + V_d + M_{cr} \frac{\ell - 2x}{x(\ell - x)}$$

x (m)	0.305	0.610	0.914	1.219	1.524	1.829	2.134	2.438	2.743	3.048
M/Vd	1.12	2.36	3.78	5.44	7.47	10.06	13.62	19.00	28.80	53.30
V_{ci} (kN)	806.0	423.0	294.0	228.6	188.2	140.1	119.2	85.4	85.0	74.7
V_{ci}^* (kN)	749.5	366.5	236.2	172.1	131.7	104.1	82.3	65.4	64.1	64.1
V_u (kN)	223.7	201.9	180.1	158.3	136.6	114.8	93.0	70.7	48.9	27.3

For every value of x, V_{ci} is larger than V_u, therefore the design is adequate for shear. If, however, the shear is checked on the basis of V_{ci}^* it can be shown that two more strands would be necessary to satisfy the flexural shear constraint.

Check Web-Shear Strength

$$V_{cw} = (0.291\sqrt{f'_c} + 0.3\ f_{pc})\ b^*_w\ d + V_p$$

$$= \left[0.291\sqrt{55.14} + \frac{0.3 \times 8 \times 1062 \times 98.7}{198690}\right] 285.75 \times 215.9$$

$$= 211\ \text{kN}$$

Applying a strength reduction factor of 0.85,

$$V_{cw} = 179.7\ \text{kN}$$

At x = 0.914 m, V_u = 180.1 kN. Therefore it is necessary to fill the voids with concrete to 0.91 m from the support.

PART I
THEORETICAL DISCUSSION

INTRODUCTION

Equation 11-11 of ACI 318-77 determines the flexure shear capacity of prestressed concrete beams. In order to apply this equation to hollow core, a value of the web width, b_w, must be determined. Since the width of the webs of most hollow core products is not constant, it is not clear what the appropriate value of b_w should be. As was demonstrated in CTA Technical Bulletin 76-B11/12, the shear strength of a concrete member is underestimated when the minimum web width, b_w^*, is used for b_w. The purpose of the bulletin is to develop appropriate parameters for use in Equation 11-11, allowing the full shear capacity of hollow core members to be used in design.

APPLICATION

Although in many cases hollow core design is controlled by moment, there are important applications where shear controls. In general, shear may control under heavy uniform loads when the plank must be highly prestressed. Examples of structures where these conditions are met are warehouses, docks, and reservoir covers subjected to earth loading. Another situation where shear may control is that of moving loads. Examples again include warehouses and docks as well as short span bridges. Shear is unlikely to control in such applications as floor planks for offices and apartments or for parking decks.

EFFECT OF COMPRESSION ZONE ON SHEAR CAPACITY

Shear failure is usually associated with diagonal cracking. For a prestressed concrete beam without stirrups, failure is assumed to occur when diagonal cracks first appear. The ACI Code distinguishes between two types of diagonal cracks.

1. Web Shear Cracks

Web shear cracking occurs when the principal tension at the centroid exceeds $4\sqrt{f'_c}$. Equation 11-13 of ACI 318-77 is a simplified expression of this fact. The shape of the compression zone has relatively little effect on the principal tension at the centroid. Therefore it is recommended that the minimum web width be used in calculating the shear strength by Equation 11-13.

2. Flexure Shear Cracks

Flexure-shear cracking occurs after a nearly vertical flexural crack has formed. Additional shear is required to transform the flexural crack into a diagonal or flexure-shear crack. Subsequent to formation of a diagonal crack, failure usually occurs due to shear compression, shear tension or diagonal tension. Shear tension is unlikely in prestressed members because the strands are flexible enough so that there is little tendency for dowel action to split the concrete along the prestress steel. Shear compression is unlikely in members with wide flanges. Finally diagonal tension is unlikely in beams with wide thick flanges. A fourth type of failure is also possible. If enough flexural cracks bend over and join one another, the top flange may buckle or local compressive stresses in the web may cause web crushing. However, the load must be increased beyond the load that causes the first diagonal crack before failure can occur. For these reasons it is expected that failure due to flexure-shear will be delayed in hollow core members.

FLEXURE-SHEAR DESIGN EQUATION

Equation 11-11 (reproduced below) defines the flexure-shear capacity of prestressed concrete beams without stirrups and the contribution of the concrete to the shear capacity of beams with stirrups.

$$V_{ci} = 0.6\sqrt{f'_c}\, b_w\, d + V_d + \frac{V_i M_{cr}}{M_{max}} \qquad (1)$$

The physical model on which Equation (1) is based states that the shear at failure equals the shear that exists at flexural cracking plus an amount that depends only on the properties of the cross-section and the concrete strength. The sum of the last two terms of Equation (1) is the shear at flexural cracking. The first term is independent of the loading.

In order to obtain better correlation with test results, Equation (1) was rewritten as follows:

$$V_{ci} = K\sqrt{f'_c}\,A_E + V_{cr} \tag{2}$$

where

A_E is the effective shear area.

K is a constant relating the ratio of the shear strength to the square root of the compressive strength.

V_{cr} is the shear that exists when flexural cracking first occurs.

The procedure for computing A_E is described in the USERS GUIDE. Based on the discussion of the previous section, the effective shear area depends on both the width and the thickness of the flange. The procedure of extending the effective shear area at a 45° angle into the flange recognizes this fact. The 45° angle was chosen somewhat arbitrarily (a larger angle is probably still safe); however, in most cases A_E is fairly insensitive to this angle.

The ACI Code establishes a lower bound to K of 0.6 for all cases, but it is reasonable to expect K to depend on a number of factors. In shallow beams, cracks tend to be narrower so that aggregate interlock does not break down as soon. The aggregate size also has a bearing on K. For these reasons it was decided to treat K as a variable to be determined from test results.

PART II
TEST PROGRAM

APPROACH

In order to verify Equation (2), tests were conducted on hollow core planks of varying depths, cross-sections, and prestress levels. The majority of the tests did not end in shear failure even though the load often exceeded 1.5 times the shear capacity calculated on the basis of the minimum web width. Thus the test loads represent lower bounds to the shear capacity. In order to prove that Equation (2) gives conservative predictions of shear capacity and because of the difficulty of obtaining a shear failure, it was decided to test the critical shear span (defined as the shear span with the largest difference between the calculated flexure shear capacity and the capacity controlled by all other constraints, in most cases ultimate moment) for each specimen. In addition as many other shear spans as practical were tested. Equation (2) was considered verified if for each specimen the load predicted by Equation (2) was reached at the critical shear span and in addition the "ultimate load" or the load predicted by Equation (2) was reached for all other shear spans tested. The ultimate load was computed based on assumptions listed below.

ULTIMATE STRENGTH

For each specimen, the ultimate strength of the plank under a single concentrated load was calculated for all shear spans up to one half the length of the specimen. For each shear span tested, the load was increased until either the calculated ultimate load was reached or until failure occurred. The assumptions which form the basis of the ultimate strength calculations are as follows:

1. The capacity of each section depends only on the shear, moment, and "bond" at that section. Conditions at other sections do not affect the capacity of the section under consideration.

2. The capacity of the plank as a whole under any particular loading condition depends only on the capacity of the critical section. This assumption and the first assumption are the basis for extending the results of concentrated load tests to other loading conditions, such as uniform loads.

3. All ϕ factors are set to 1.0.

4. Flexure-shear capacity is calculated by Equation (1) with $b_w = b_w^*$, but is not considered as a constraint on the ultimate capacity.

5. Web shear capacity is computed by Equation 11-13 of ACI 318-77 reproduced below:

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

6. Moment capacity is based on strain compatibility. The assumed stress-strain relationships for the concrete and prestressing steel are shown in Figures 2 and 3 respectively.

7. The effect of bond on moment capacity near the ends of prestressed beams is calculated by the procedure outlined in CTA Technical Bulletin 77-B10/11 which is based on the provisions of Section 12.11.1 of ACI 318-71. The following equation is used to calculate the stress in the prestressing steel:

$$f_{ps} = \frac{\ell_d}{d_b} + \frac{2}{3} f_{se} \leq f_{pu}$$

Figure 4 shows the assumed relationship between f_{se} and distance from the end of the beam.

LOADING

The hollow core planks were tested as simply supported beams subjected to single concentrated loads. Figure 5 shows a test in progress and Figure 6 illustrates the test setup. The span, L, was the distance between the center lines of the 3 in. bearing pads. The shear span, a, was measured as the span from the center line of the loading to the center line of the nearest support. Dial gages with 1 inch travel were used to measure deflections at three points. In addition, a ruler was used to measure deflections at the load point when the 1 inch travel of the dial gage was exceeded.

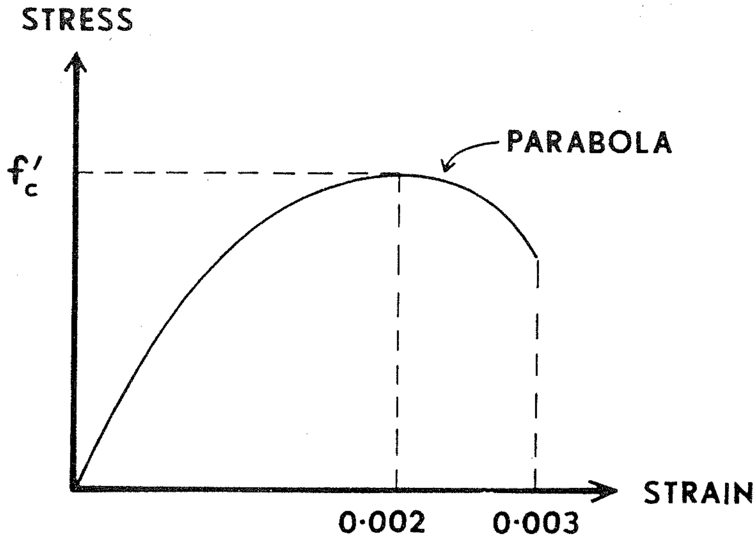


FIG. 2: ASSUMED STRESS-STRAIN RELATIONSHIP OF CONCRETE FOR USE IN ULTIMATE STRENGTH CALCULATIONS ONLY.

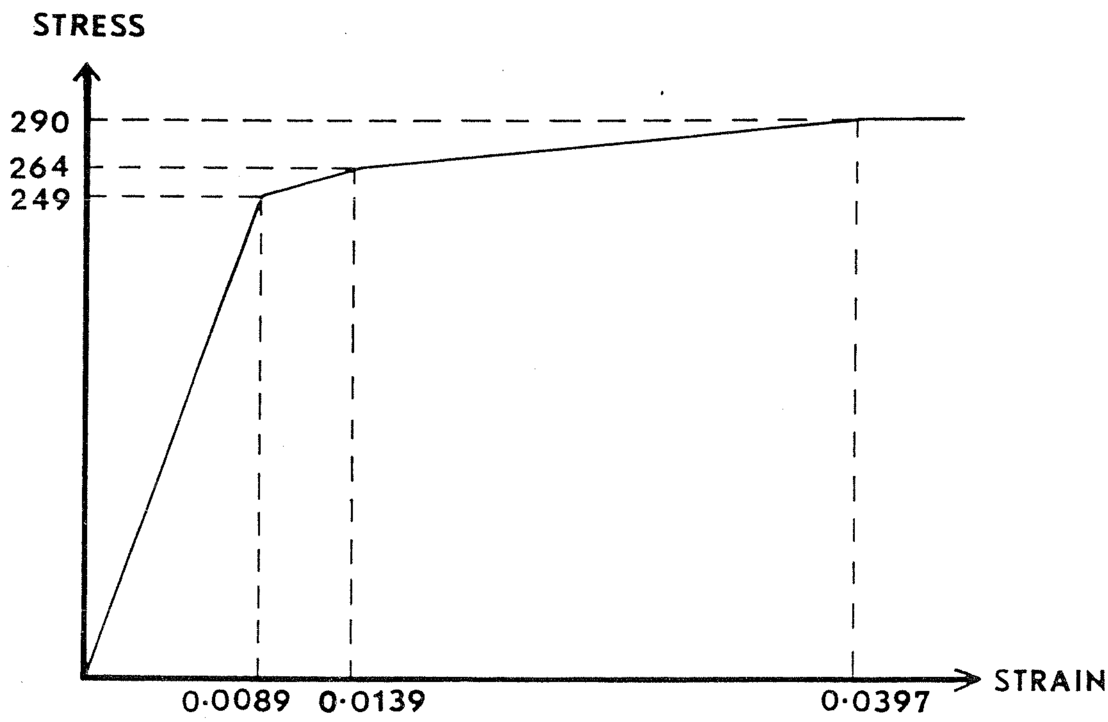


FIG. 3: ASSUMED STRESS-STRAIN RELATIONSHIP FOR 1/2"Ø 270^k STRANDS.
THIS GRAPH WAS FITTED TO TEST DATA FROM THE FOLLOWING SOURCES:

1. CTA TESTS.
2. SHENKO WIRE CO. TESTS 3/12/76.

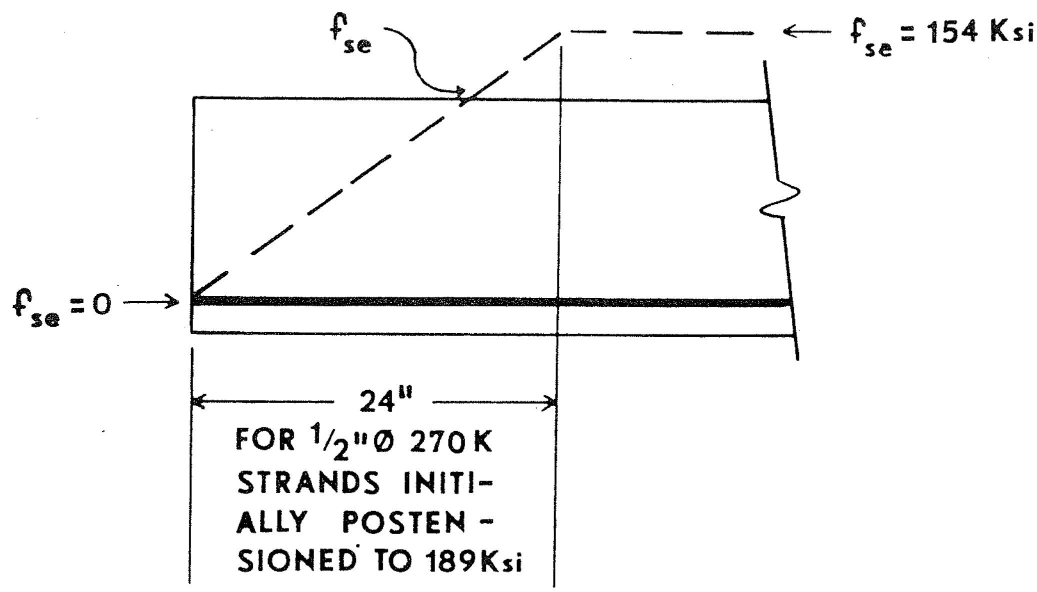


FIG. 4 – VARIATION OF f_{se} WITH DISTANCE FROM END OF THE BEAM.

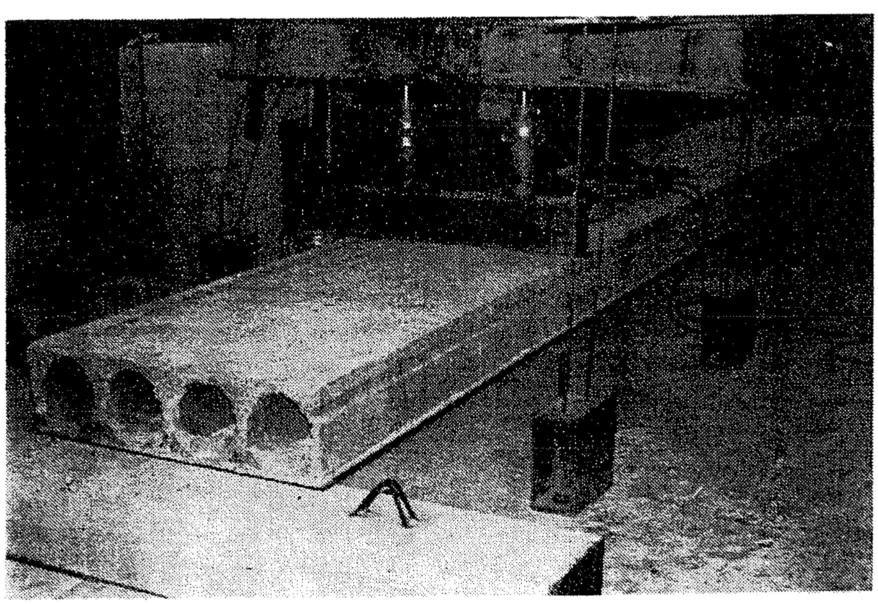


FIG. 5 - TEST IN PROGRESS

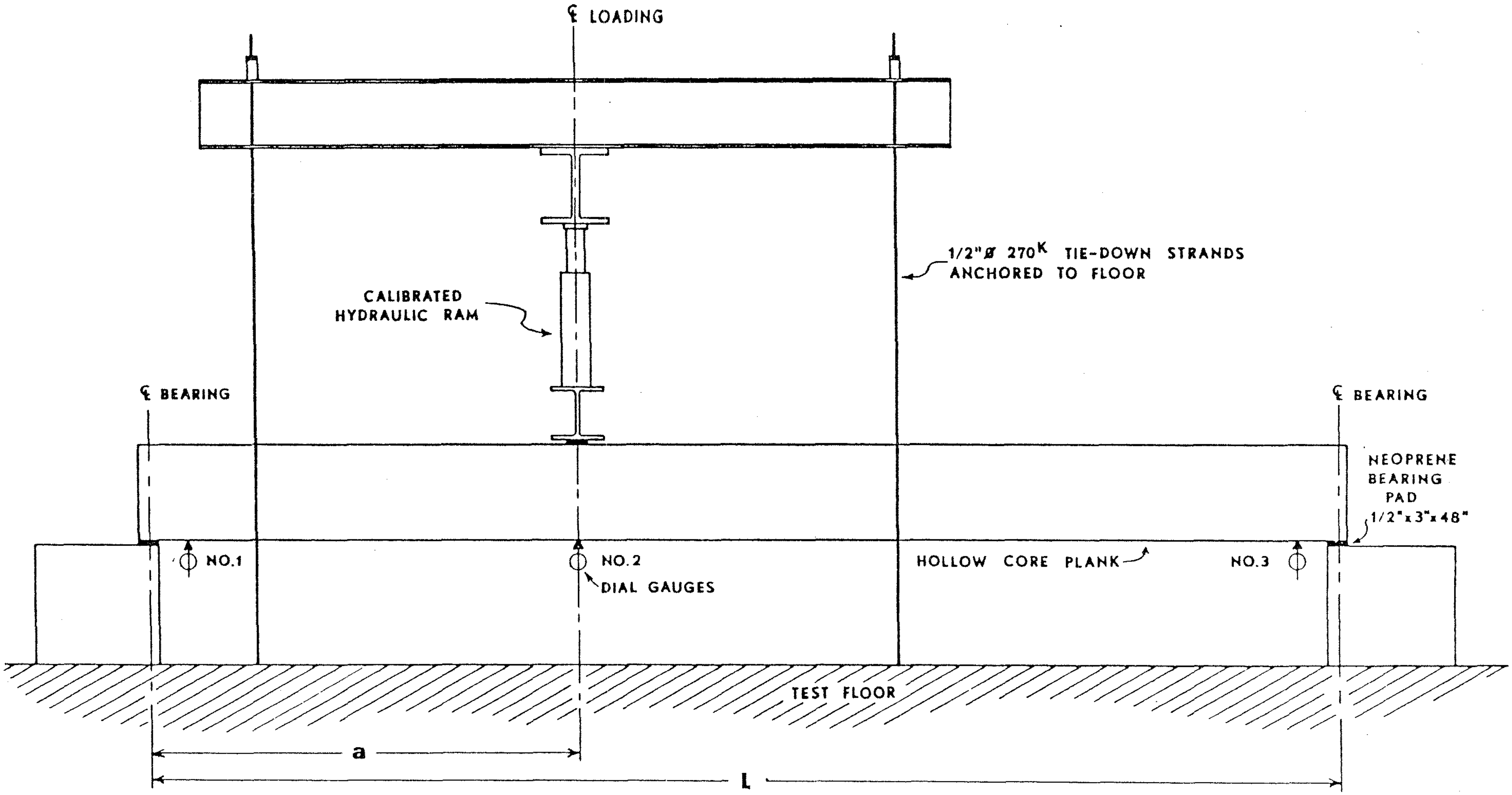


FIGURE 6 - SUPPORT AND LOADING APPARATUS

SPECIMENS

The following variables were considered to be the most significant:

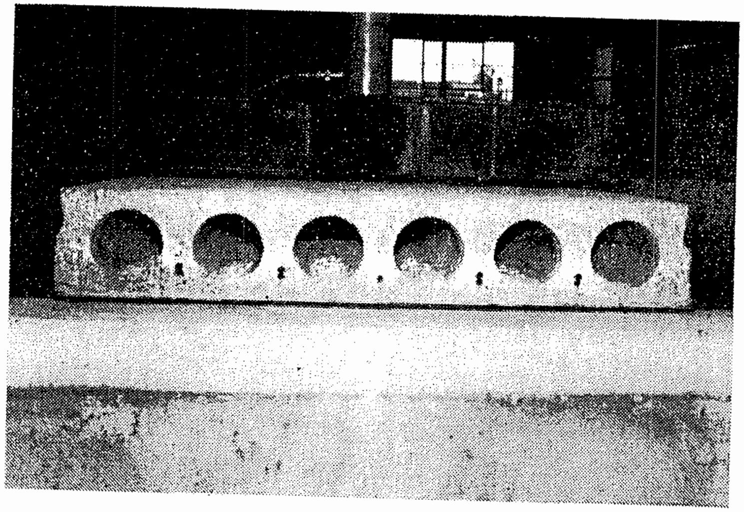
1. Cross-section
2. Prestress
3. Moment to shear ratio

Different hollow core products were tested in order to vary the cross-sectional properties. The properties of interest were the ratio of A_E to $b_w^* d$ and the depth of the section. The range of prestress levels that could be tested was limited by the difficulty of obtaining shear failures for lightly prestressed beams, however, an extensive range of moment to shear ratios was tested by varying the shear span. The lengths of the test specimens was somewhat random because the specimens were chosen from stock in the Concrete Technology Corporation yard. Nine specimens were tested, whose properties are listed in Table 2. The concrete compressive strengths were determined from Schmidt Hammer readings (see the supplement to CTA Technical Bulletin 76-B3). Figure 7 shows four of the test specimens.

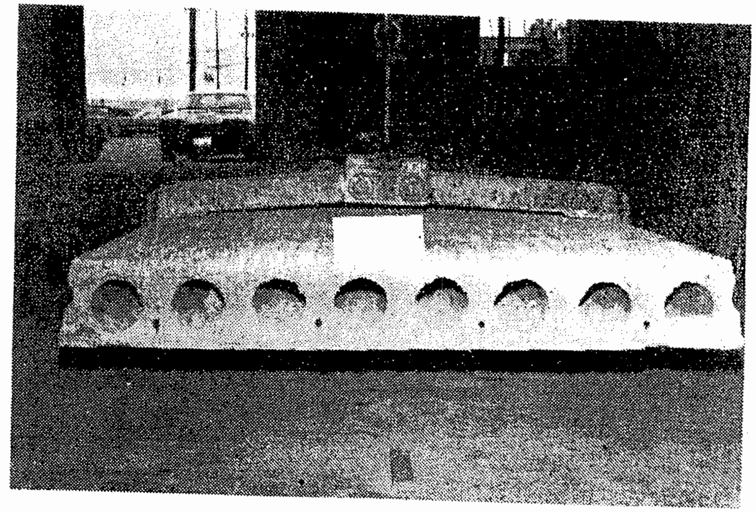
TABLE 2

SPECIMEN LIST

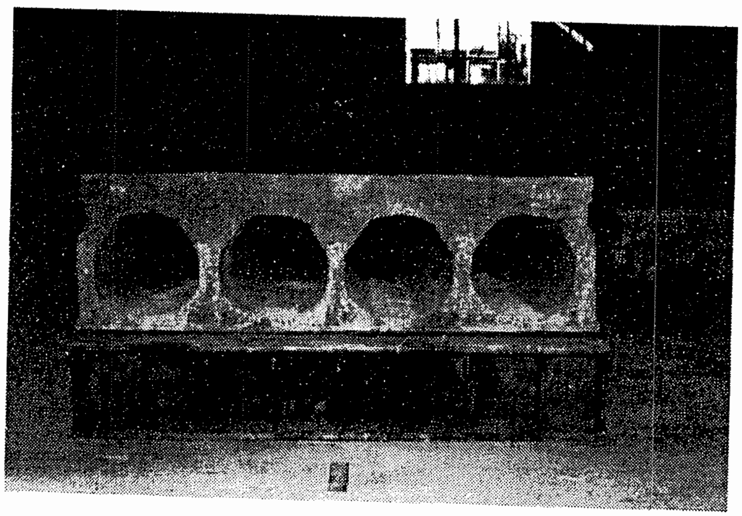
PRODUCT	DEPTH (inches)	NUMBER OF STRANDS	d	f'_c (ksi)
Spiroll	6	4	4.25	11.4
	8	9	6.00	11.5
DyCore	12	6	10.25	10.7
	12	6	10.25	10.6
	12 ½	11	11.75	10.1
	14 ½	8	12.75	11.0
	14 ½	8	12.75	11.1
	14 ½	8	12.75	11.1
	14 ½	8	12.75	11.1



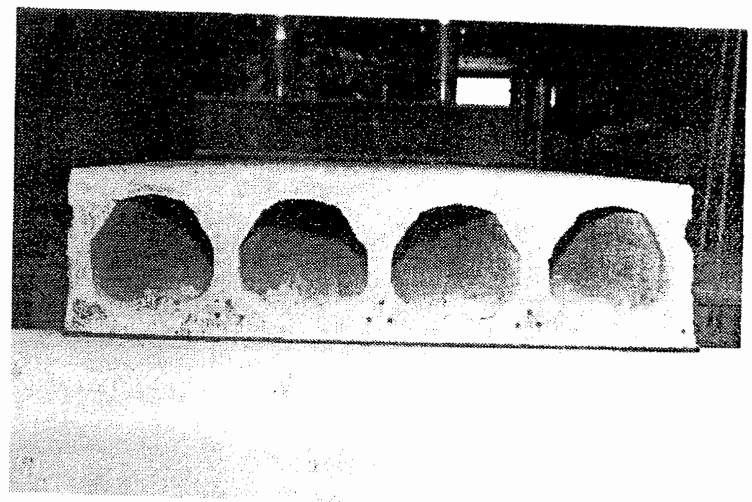
A) 8 IN. SPIROLL, 9 STRANDS



B) 6 IN. SPIROLL, 4 STRANDS



C) 14 1/2 IN. DYCORE, 8 STRANDS



D) 12 1/2 IN. DYCORE, 11 STRANDS

FIG. 7 - TEST SPECIMENS

TEST PROCEDURE

Several preliminary tests of damaged hollow core planks were made at the beginning of work on this bulletin. It was immediately clear from the results that hollow core has a very high flexure shear capacity so that it would be difficult to obtain shear failures. For this reason the following test procedure was adopted. Each plank was first tested with the concentrated load at or near midspan. The load was increased until the calculated ultimate load was reached. Then the load point was moved several feet toward one support and retested. The process was repeated until failure occurred. The undamaged end was then tested, but with a shorter overall span. In this manner as many as six tests were performed on a single plank. Results for a large number of moment to shear ratios were obtained.

RESULTS

The test results are summarized in Table 3. The specimens are designated by two numbers and a letter. The first number gives the total depth of the plank, the letter defines the product (S means Spiroll and D means DyCore) and the last number gives the number of strands. The test number, the total span (L), the shear span (a) and the embedment length (ℓ_d) are given in the next three columns. The remainder of the table is divided into calculated values and test results. The individual headings are explained below:

$M_{max}/V_i d$ - Shear to moment ratio at the critical section.

P_{cr} - Value of the concentrated load which causes the first flexural crack to occur. Cracking is assumed to occur when tensile stress first exceeds $6\sqrt{f'_c}$.

P_{ci} - Load that causes flexure-shear failure according to Equation 11-11 of ACI 318-77 with $b_w = b_w^*$.

P_M - Load that causes a flexural failure where the moment capacity is based on strain compatibility.

P_B - Load that causes a "flexural bond" failure. This value is given only when it is less than P_M.

P_{cw} - Load that causes a web shear failure according to Equation 11-12, assuming full prestress at the critical section.

P'_{cw} - Same as P_{cw}, but using the value of the effective prestress that exists at a distance of d/2 from support.

V_{ci} - Shear at the load point when the load equals P_{ci}.

P_{max} - Maximum load reached during the test.

P^t_{cr} - Load when first flexural crack was observed. This value is not always recorded because in some cases the beam was cracked prior to testing.

K - Computed by the following formula

$$K = \frac{V_{max} - V_{cr}}{A_E\sqrt{f'_c}}$$

where

V_{max} = shear at the load point when $P = P_{max}$

V_{cr} = shear at the load point when $P = P_{cr}$

Δ_{max} - Maximum recorded deflection

Δ_{res} - Residual deflection

The column headed "failure code" gives a code for the condition of the beam when the test ended.

None of the tests resulted in flexure-shear failures, even though all but five of the tests were predicted to result in flexure-shear failures according to Equation 11-11, based on the minimum web thickness. This can be seen by comparing P_{ci} with P_u where P_u = min $(P_M, P_B, P_{cw}, P'_{cw})$.

Only two of the tests actually ended in shear failures, but they were web shear failures. The rest of the failures were due to bond except for the tests of 6 inch Spiroll which ended when the steel yielded. Probably those tests would also have resulted in bond failure except that the end of the beam was extended 4 feet past the support point in order to avoid bond failure.

TABLE 3

TEST RESULTS

SPEC. CODE	VARIABLES: TEST NO.	VARIABLES: L (ft)	VARIABLES: a (ft)	VARIABLES: ℓ_d (ft)	CALCULATED: $\frac{M}{Vd}$	CALCULATED: P_{cr} (kips)	CALCULATED: P_{ci} (kips)	CALCULATED: P_M (kips)	CALCULATED: P_B (kips)	CALCULATED: P_{cw} (kips)	CALCULATED: P'_{cw} (kips)	CALCULATED: V_{ci} (kips)	TEST RESULTS: P_{max} (kips)	TEST RESULTS: P^t_{cr} (kips)	TEST RESULTS: K	TEST RESULTS: FAILURE CODE*	TEST RESULTS: Δ_{max} (in.)	TEST RESULTS: Δ_{res} (in.)
6S4	1	12.25	2.13	6.0	6.0	18.1	23.1		28.2	38.8		19.8	34.0	24.4	1.19	4	1.94	0.30
	2	12.25	1.63	5.0	4.6	22.9	27.6		33.7	34.0		24.8	44.5	30.1	1.70	4	1.30	0.30
8S9	1	25.00	10.13	10.25	22.0	11.9	21.4	26.0				13.3	23.6	14.1	0.47	6	4.25	0.00
	2	25.00	7.63	7.75	16.0	14.0	22.1	30.0				16.6	27.6	20.7	0.64	6	3.90	0.13
	3	25.00	5.63	5.75	11.5	17.7	24.9		32.6			21.1	35.5	22.1	0.94	6	4.63	0.25
	4	25.00	3.83	4.00	7.8	24.8	31.5		36.8			28.9	45.0	29.6	1.16	5		
	5	19.25	2.63	2.75	5.3	35.5	41.8		44.3	61.4	38.6	40.0	39.1		0.22	5		
	6	17.90	6.83	6.92	14.1	19.3	28.5		38.9			18.1	40.2	24.3	0.87	5		
12.5D11	1	31.25	11.13	11.25	13.0	25.2	33.7	50.8				23.0	50.2	26.0	0.90	6	3.00	0.38
	2	31.25	15.63	15.75	19.3	22.7	33.7	46.2				16.8	43.3		0.58	6	5.75	-
	3	31.25	7.13	7.25	8.1	34.2	41.3	67.3		66.1	46.0	34.5	62.4		1.22	5	4.88	0.38
	4	28.25	4.63	4.75	5.2	51.1	57.6		76.8	61.5	43.0	51.1	62.8	56.4	0.55	2		
	5	26.25	9.13	9.25	10.5	32.0	40.4	62.9		79.7	55.8	27.6	58.0		0.95	6	5.00	0.75
	6	26.25	2.88	3.00	3.2	79.7	85.8		95.6	58.3	40.9	79.5	60.5		-0.95	2		
12D6	1	28.25	14.13	14.25	19.1	14.3	25.1	28.0				12.5	25.7	16.7	0.35	6	2.90	0.30
	2	28.25	12.13	12.29	15.8	14.7	24.1	28.6				14.3	27.0		0.43	6	3.50	0.13
	3	28.25	9.13	9.29	11.3	16.9	24.8	32.5				18.2	30.8		0.58	6		
	4	28.25	6.88	7.00	8.3	20.8	27.9		35.8			23.2	38.3	28.1	0.81	6	2.75	0.00
	5	28.25	4.42	4.54	5.3	30.6	37.0		39.7			33.9	43.8		0.68	5		
	6	21.25	5.67	5.79	6.8	28.0	35.4		41.6			27.3	46.5	34.4	0.83	5		
12D6	1	8.00	2.88	3.00	3.4	68.8	77.2		72.0	69.1	55.7	49.7	69.8		0.04	5		
14.5D8	1	18.25	9.13	9.29	9.1	41.0	54.7	77.7				27.2	70.3	46.9	0.52	6	2.90	0.30
14.5D8	1	18.10	6.63	6.71	6.3	45.0	55.8		75.9	83.0	67.4	36.5	86.1	54.3	0.93	5		
14.5D8	1	18.75	6.63	6.71	6.3	43.9	54.4		74.2	82.4	66.3	36.5	84.7	54.0	0.94	5	3.25	0.75
	2	11.25	4.13	4.29	4.0	75.5	86.4		97.5	90.6	70.3	55.0	114.0	93.2	0.86	5		
14.5D8	1	11.25	5.63	7.75	5.4	70.3	83.9		127.2	107.1	90.8	42.0	116.8	83.0	0.82	6	1.75	0.10
	2	14.25	5.13	6.00	4.9	59.3	69.9		92.3	84.2	68.0	45.6	109.5	85.8	1.14	5	1.00	0.25

* FAILURE CODE - 1 = Flexure Shear
2 = Web Shear
3 = Moment-Concrete Crushing
4 = Moment-Steel Yielding
5 = Moment-Strands Slipping
6 = No Failure

VARIATION OF LOAD CAPACITY WITH SHEAR SPAN

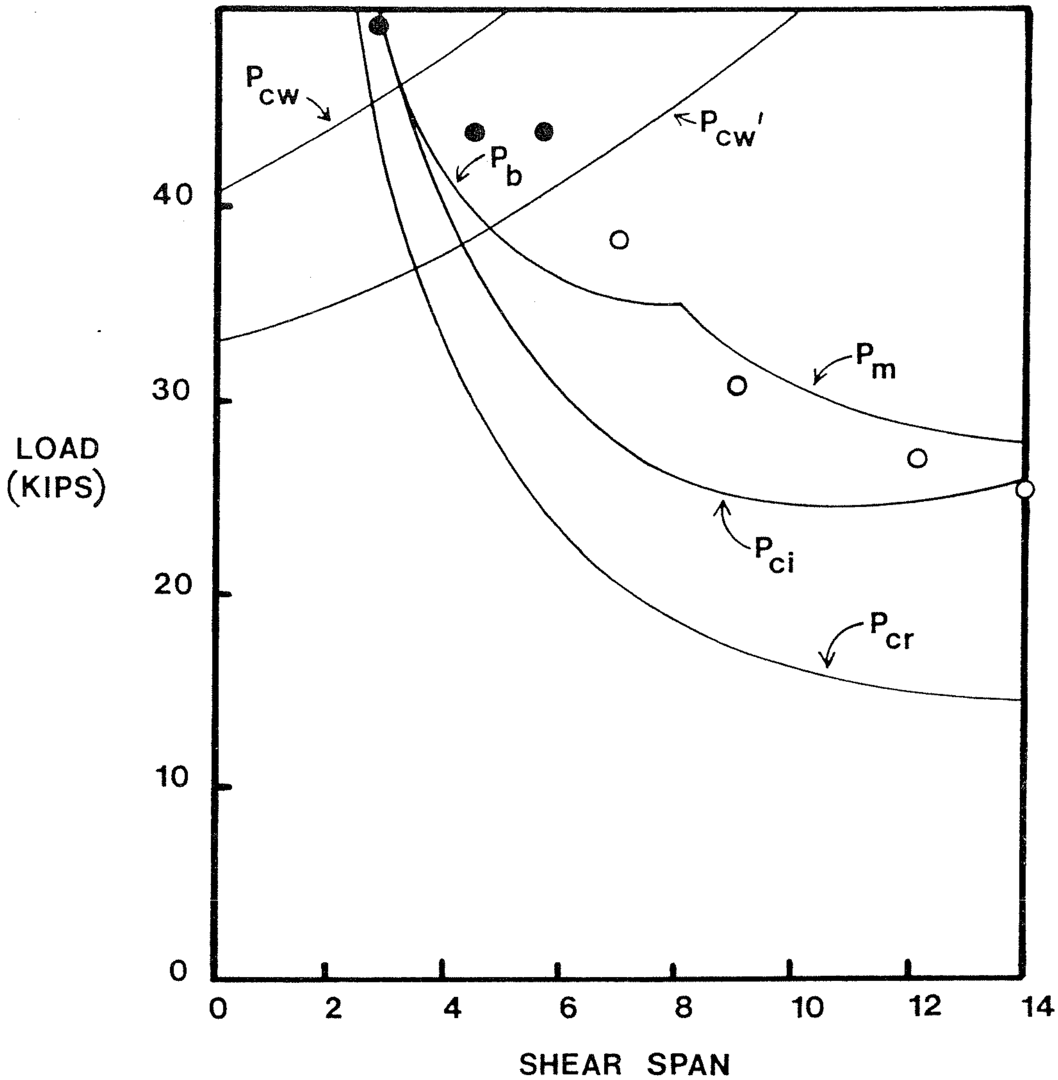
A chart of load capacity versus shear span was made for each specimen tested where the shear span varied from zero to one half of the span of the original plank. For each value of the shear span, six values of the load were plotted - P_{cr}, P_{ci}, P_M, P_{cw}, P'_{cw}, and P_B. Figures 8 through 11 show such plots for four of the test specimens. The theoretical ultimate load envelope (ignoring P_{ci}) is given by the lowest value of the constraints P_M, P_B, P_{cw}, and P'_{cw}. P_{max} and P_{cr} for each test are also plotted in the figures at the shear span of the test. For those tests where the total span was reduced, P_{max} and P_{cr} are also reduced to the load that would cause the same moment at the load point with the original span.

The test cracking load was always higher than the calculated cracking load even though a very high value of the concrete compressive strength was used in the calculations.

P_{max} appeared to follow the P_M constraint fairly closely. Although only two tests actually failed due to moment outside of the region controlled by bond, many of the tests were carried to P_M or just slightly above P_M. For most of these tests, the planks appeared to be near failure at the time the test was ended.

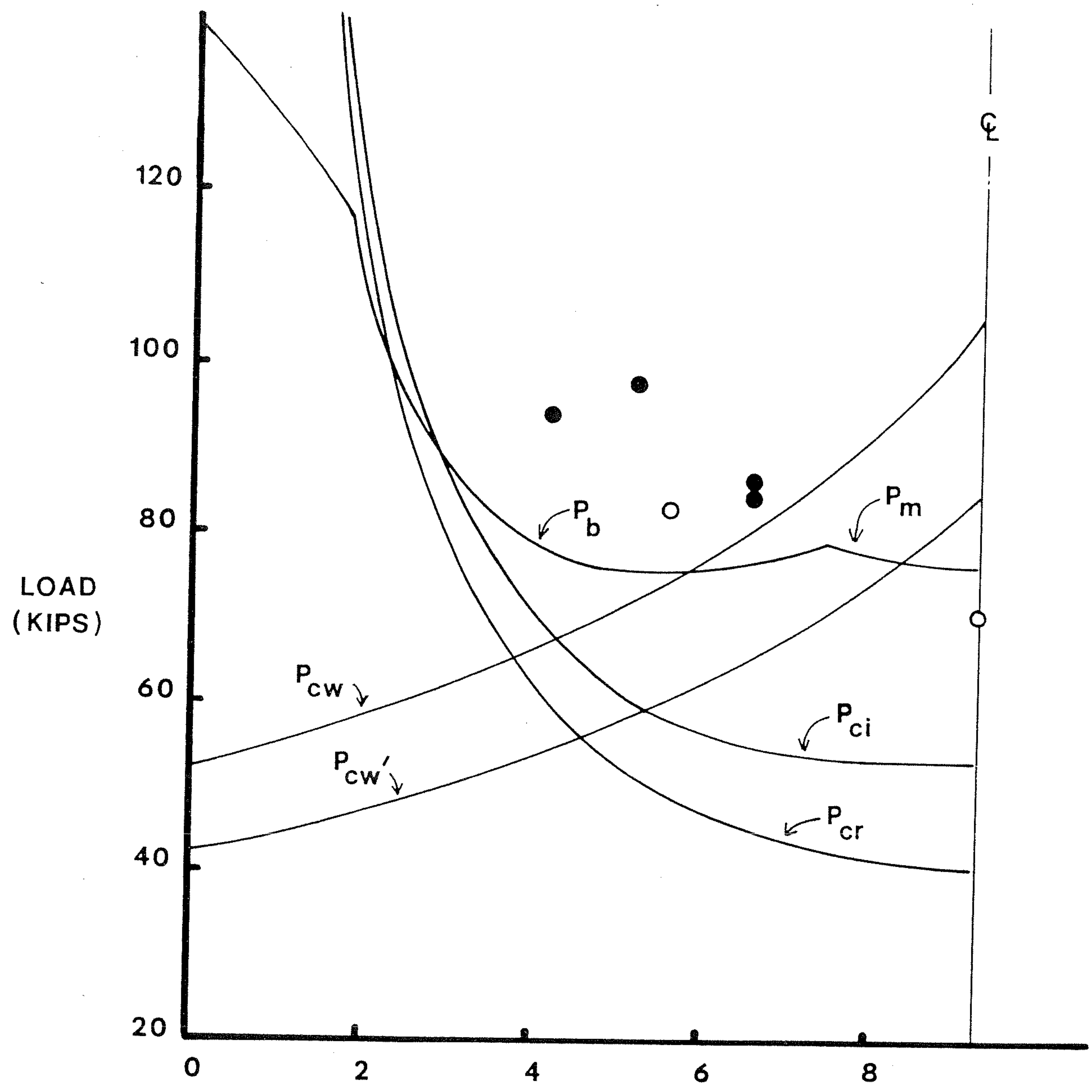
When bond controlled, P_{max} was sometimes very close to P_B and sometimes substantially above P_B. For the 12 inch DyCore with 6 strands, P_B was followed quite closely as can be seen from Figure 8. For the 8 inch Spiroll with 9 strands, P_B was exceeded in three tests. For the 12½ inch DyCore and 6 inch Spiroll, bond never controlled. For the 14½ inch DyCore, bond did not theoretically control, however, the tests ended in bond with P_{max} exceeding P_B. Although these tests are not exhaustive, the results strongly support the contention that the present ACI provisions for bond are adequate for hollow core.

When web shear controlled, P_{max} tended to ignore P'_{cw}. Instead failure usually occurred at a load slightly above P_{cw}. In some cases, even P_{cw} was exceeded by a considerable amount.



NO. STRANDS = 6, L = 28'
● – FAILURE
○ – NO FAILURE

FIG.8– TEST RESULTS FOR 12" DYCORE

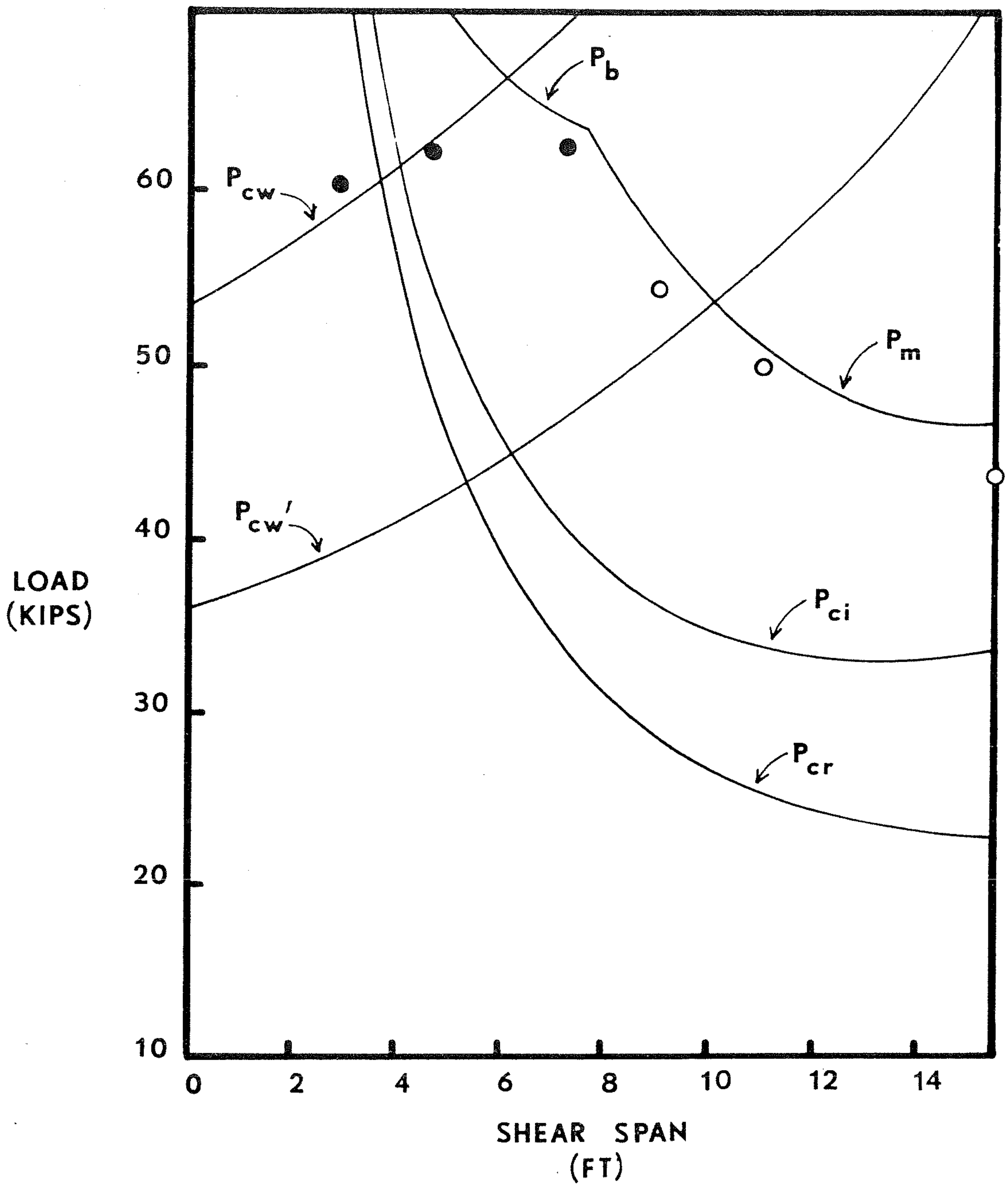


L = 18'-6", NO. STRANDS = 11

● – FAILURE

○ – NO FAILURE

FIG. 9 – TEST RESULTS FOR 14½" DYCORE



NO. STRANDS = 11, L = 31'

● — FAILURE

○ — NO FAILURE

FIG. 10 - TEST RESULTS FOR 12½" DYCORE

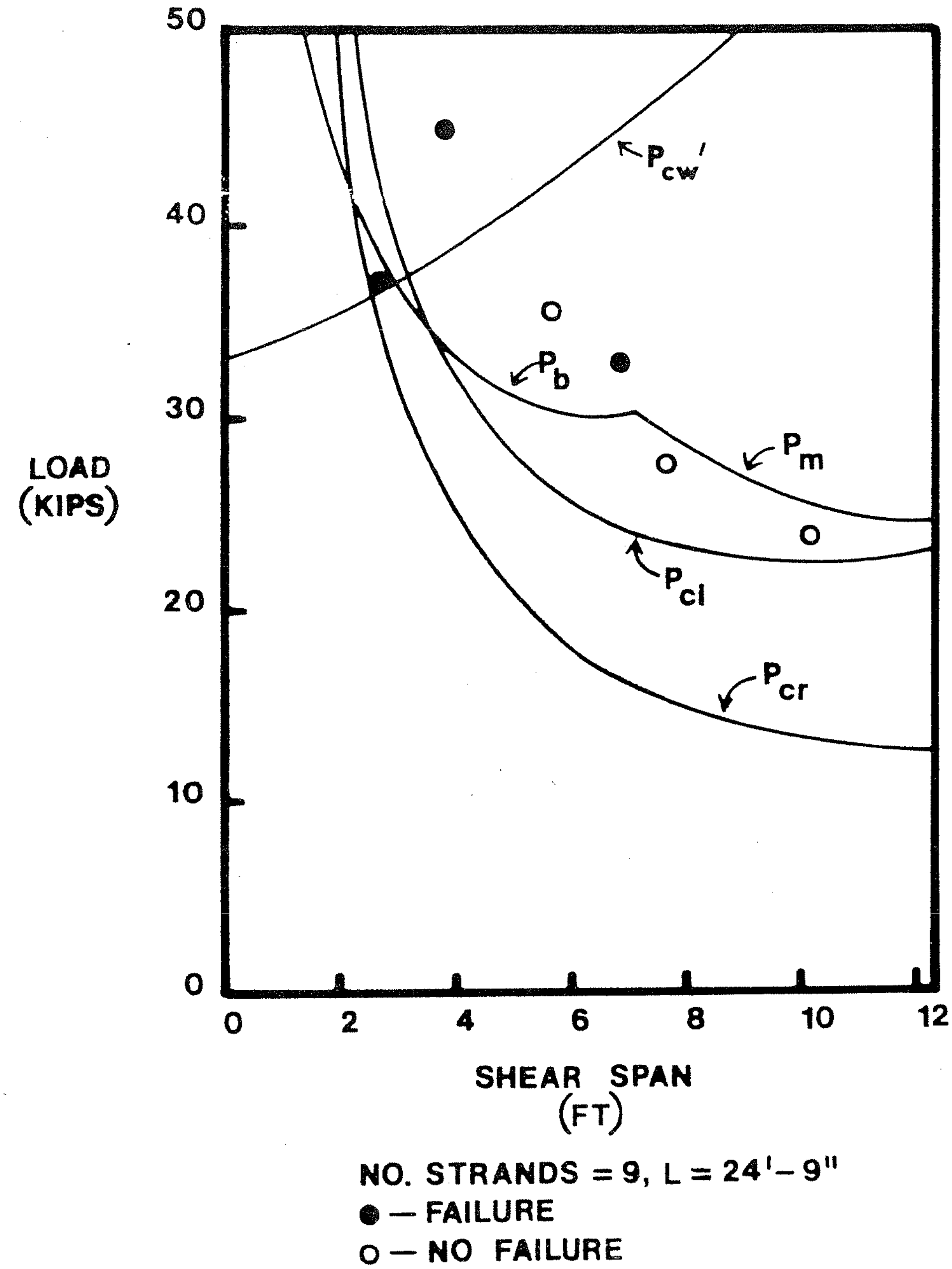


FIG.11– TEST RESULTS FOR 8" SPIROLL

The planks totally ignored the flexure-shear constraint as can be seen in all four graphs. The shear span that was considered to be most critical in flexure-shear was the span where the difference between P_{ci} and all other constraints was maximum. This point occurs where the P_M line first intersects P_{cw} or P_B. Each specimen was tested at least once with the load at or near this shear span. The fact that flexure-shear did not occur even at this critical span meant that only a lower bound to the flexure-shear capacity could be found for the products tested.

DISCUSSION OF TEST RESULTS

Figure 12 shows a plot of K versus the ratio of the maximum load reached during each test to the calculated ultimate load for that test. From Figure 12, it can be seen that in no case did the plank fail before P_u was reached. Thus none of the tests represent upper bounds to K because none of the tests ended either in flexure-shear failures or at loads less than P_u. Since only lower bounds to K can be determined from the test data, the recommended design values of K will be less than the "true" value of K until sufficient data are available to obtain upper bounds to K.

For every type of specimen tested, a lower bound to K exceeding 0.6 was obtained (see Table 3). A total of seventeen tests yielded K values greater than 0.6 as can be seen in Figure 12. For the specimens tested, A_E/b_w^*d varied from 1.58 to 2.47 and d varied from 4.25 inches to 12.75 inches. This range is large enough to justify replacing b_w^*d by A_E in Equation (1) for any type of hollow core product.

Five of the tests representing four different specimens yielded K values greater than 1.0. For 14½ inch DyCore, the largest K value was 1.14, while for 6 inch Spiroll the largest value obtained was 1.70. As discussed previously, K is expected to vary inversely with the effective depth. Thus a value of K=1.0 is justified for Spiroll and DyCore planks which are 14½ inch deep or less. Although a higher value of K could be justified for shallower sections, not enough results are available at this time to develop a relationship between K and depth.

The remaining variables are the prestress level and the moment to shear ratio. The prestress at the centroid of the concrete ranged from 0.494 to 0.876 ksi, which represents a reasonable range of values. The moment to shear ratio may be non-dimensionalized by dividing M_{max}/V_i by the effective depth, d. The K values are plotted against $M_{max}/V_i d$ in Figure 13. The range of $M_{max}/V_i d$ was 3.2 to 22.0. A value of K = 1.0 can be justified for $M_{max}/V_i d$ ratios less than or equal to 10.0 because the five tests with K values greater than 1.0 occurred in that range. For $M_{max}/V_i d$ ratios greater than 10.0, K = 0.75 appears to be justified by the data. There is no reason to expect K to be smaller for high $M_{max}/V_i d$ ratios, although until additional data can be obtained it is prudent to use the value K = 0.75 in design.

SUMMARY

Based on the results of tests described in PART II, the following conclusions were made:

1. It is nearly impossible to obtain a flexure-shear failure of a hollow core product.

2. Equation (2) may be used to obtain conservative estimates of flexure shear capacity.

3. For any hollow core product, A_E may be computed by the procedure outlines in the USERS GUIDE.

4. For any hollow core product, K = 0.6 may be used in design.

5. Higher values of K for specific products may be justified by testing.

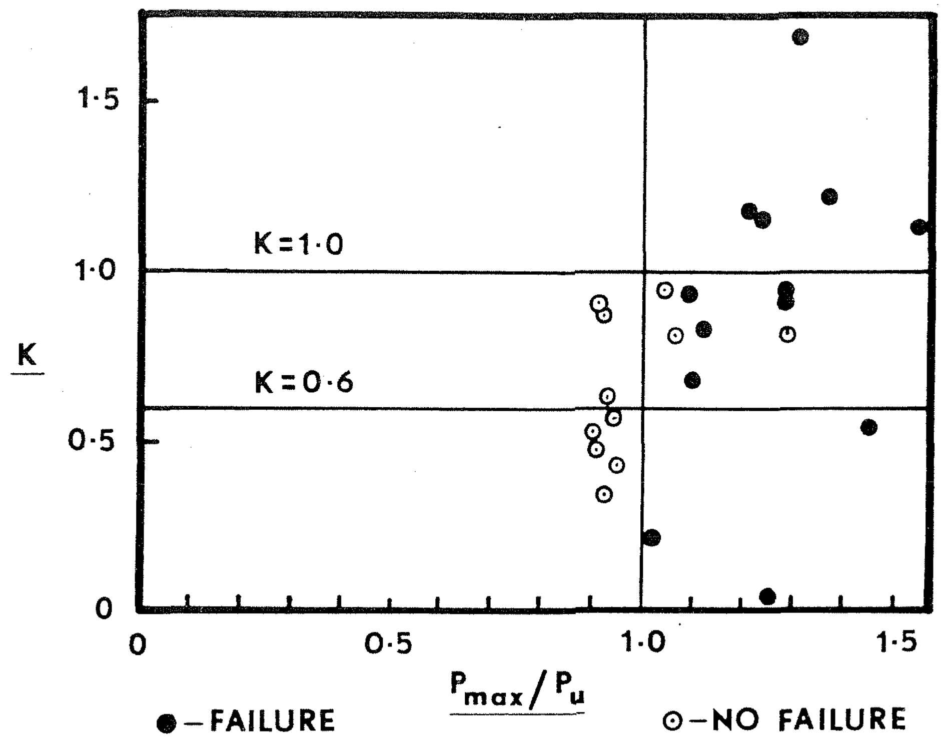


FIG. 12 – PLOT OF K vs P_{max}/P_u

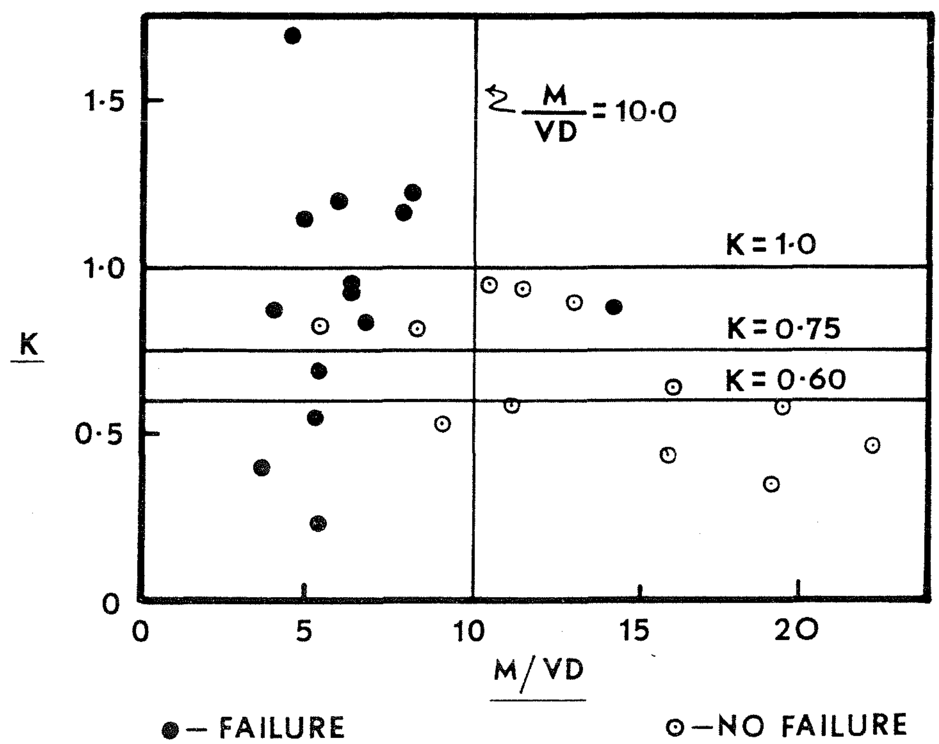


FIG. 13 – PLOT OF K vs MOMENT TO SHEAR RATIO.

NOTATION

a = Shear span, distance between concentrated load and center support

A = Gross area of concrete cross-section

A_E = Effective shear area

b = Width of compression flange

b_w^* = Web width

b_w = Minimum web width

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

d_b = Nominal diameter of prestressing strand

f'_c = Specified compressive strength of concrete, psi

$\sqrt{f'_c}$ = Square root of specified compressive strength of concrete, psi

f_d = Stress due to unfactored dead load, at extreme fiber of section where tensile stress is caused by externally applied load, psi

f_{pc} = Compressive stress in concrete (after allowance for all prestress losses) at centroid of cross section resisting externally applied loads or at junction of web and flange when the centroid lies within the flange, psi.

f_{pe} = Compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied load, psi.

f_{ps} = Stress in prestressing at design moment

f_{se} = Effective stress in prestressing steel in uncracked section

f_{pu} = Specified tensile strength of prestressing tendons, psi

h = Overall thickness of member, inches

I = Moment of inertia of section

K = Constant determined by tests

L = Span measured center line to center line of bearing

ℓ_d = Embedded length of strand from end of member to critical section

M_{cr} = Moment causing flexural cracking at section due to externally applied loads.

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

V_{ci} = Nominal shear strength provided by concrete when diagonal cracking results from combined shear and moment

V_{cw} = Nominal shear strength provided by concrete when diagonal cracking results from excessive principal tensile stress in web

V_d = Shear force at section due to unfactored dead load

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

V_p = Vertical component of effective prestress force at section

W_d = Dead load per foot

W = Uniform load per foot

W_u = Factored total ultimate load per foot

Y_t = Distance from centroidal axis of the gross section, neglecting reinforcement to extreme fiber in tension

Z_t = Section modulus

ϕ = Strength reduction factor