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Shear and Moment Transfer Between Concrete Slabs and Columns

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SYNOPSIS

The strength of flat slabs near columns was investigated by 17 tests involving combined shear and unbalanced moment loadings. Sixteen reinforced concrete slab–column specimens containing square or rectangular interior columns and one specimen with a square edge column were tested. Narrow rectangular holes were located adjacent to the columns in eight of the specimens. Reversals of loading simulating earthquake effects were applied to three of the specimens. Four design methods for shear strength, presented by Di Stasio and Van Buren, Johannes Moe, ACI-ASCE Committee 326, and the Commentary on the 1963 ACI Building Code, are evaluated in terms of Moe's data and test data reported herein. A modification of the Committee 326 method is recommended for practical design.

KEY WORDS: columns (structural); concrete slabs; cyclic loads; flat plates (concrete); flexural tests; openings; punching shear; shear tests; structural analysis

Design of flat plate concrete slabs is often controlled by shear strength of the slab. Earthquake and wind loadings may cause substantial unbalanced moments to be transferred between the slab and the column. This makes shear even more critical than for gravity load alone.

The shear strength near columns of symmetrically loaded slabs has been extensively investigated. Previous work on this subject was summarized by ACI-ASCE Committee 326 (now 426) on Shear and Diagonal Tension.(1)*

Only limited analytical and experimental study has been devoted to slab–column junctions subjected to both shear and unbalanced moment. This paper reports tests of specimens under such loading. Four methods for shear strength computation of these junctions are evaluated.

The 17 tests reported in this paper include square and rectangular interior columns, square edge columns, and square interior columns with adjacent slab openings. Reversals of loading were applied to three interior column test specimens.

*Numbers in parentheses designate references at end of paper.

BACKGROUND

Previous Tests

In 1959 Rosenthal(2) reported the results of tests on simply supported circular reinforced concrete slabs. These tests included three specimens loaded eccentrically through a centrally located circular stub column.

The following year Tsuboi and Kawaguchi(3) reported nine tests on square mortar slabs. Three of these slabs were made of plain mortar. The distribution of the reinforcement in the other six slabs varied, although all six contained an equal total amount of reinforcement. These specimens were loaded by applying a couple through centrally located square column stubs.

Kreps and Reese(4) have reported the results of six tests carried out by Frederick and Pollauf(5) on square flat plate specimens. These tests were carried out to determine the effective width of slab resisting moment. Distribution of reinforcement was included as an important variable.

In 1961, Johannes Moe reported, in BULLETIN D47,(6)* 12 tests on 6-ft-square, 6-in.-thick slabs. The test specimens were simply supported, and the corners were free to lift. Load was applied at different eccentricities through a centrally located square stub column. The results of 10 of these tests are presented in Table 1. Specimens M4 and M5 are not included here because they failed in flexure rather than shear. All the specimens except M7 and M9 had a 2-in.-diameter hole through the slab located along the line of action of the applied load.

*PCA DEVELOPMENT DEPARTMENT BULLETINS will be identified in the text primarily by the BULLETIN number. BULLETINS are available on request in the United States and Canada.

Methods for Predicting Strength

Di Stasio and Van Buren.(7) A working stress method of analysis was recommended for the strength of the slab-column junction under combined shear and unbalanced moment loadings. The major criterion of their method is limitation of the vertical shear stress on critical sections located at specified distances outside of the column periphery. The vertical shear stress is assumed to be distributed as shown in Fig. 1 for typical interior and exterior flat plate-column connections. The vertical shear stress, v_1, is calculated from

$$v_1 = \frac{8t}{7d}\left[\frac{V}{A_c} + \frac{(M - m_{AB} - m_{CD})c}{J_c}\right]C_1 \quad \text{..(1)}$$

where

$$C_1 = \frac{1}{1 + (n-1)p}$$

and where, for an interior column

$$A_c = 2t\,(x + y)$$

$$J_c = \frac{x^3 t}{6} + \frac{xt^3}{6} + \frac{x^2 yt}{2}$$

In Eq. (1), V and M are the resultant shear and unbalanced moment acting at the centroidal axis, $c-c$, of the critical section. The flexural moments on sides AB and CD of the critical section defined by x and y are m_{AB} and m_{CD}. They are assumed equal to the moment that produces the maximum working stress in either the concrete or tension steel, including the effect of any compression steel. A_c and J_c are properties of

TABLE 1—SLAB TESTS BY MOE(6)

Specimen No.	Effective Depth, d (in.)	Column Size (in.)	Cylinder Strength, f'_c (psi)	Reinforcement Ratio, p		Ultimate Moment Transferred (in. kips)	Ultimate Shear Transferred (kips)
				Bottom Face	Top Face		
M1A	4.88	12x12	3020	0.0129	0	0	97.3
M2A	4.88	12x12	2250	0.0129	0	349	47.8
M4A	4.88	12x12	2560	0.0129	0	553	32.3
M2	4.88	12x12	3730	0.0129	0	506	65.7
M3	4.88	12x12	3295	0.0129	0	621	46.6
M6	4.81	10x10	3840	0.0117	0	356	53.8
M7	4.81	10x10	3620	0.0117	0	168	70.0
M8	4.81	10x10	3570	0.0117	0.0054	578	33.6
M9	4.81	10x10	3370	0.0117	0	300	60.0
M10	4.81	10x10	3060	0.0117	0.0054	485	40.0

the area of the critical section ABCD. The total slab thickness and effective slab depth are equal to t and d, respectively. C_1 is a constant that modifies A_c and J_c to account for the doweling action of the reinforcement. The reinforcement ratio and the modular ratio between the steel and concrete are equal to p and n, respectively. The factor 8t/7d is used to conform to the design practice current in 1960 of calculating shear stress by V/bjd and assuming j equal to $\frac{7}{8}$.

In consideration of punching shear, Di Stasio and Van Buren limited the maximum vertical shear stress to $0.0625f'_c$ on a critical section directly adjacent to the column periphery. For stress from shear and moment, the diagonal tension recommendations of the 1956 ACI Building Code[8] were followed. The critical section was assumed to be located at a distance equal to the effective slab depth outside of the column periphery. The maximum vertical shear stress on this section was limited to $0.03f'_c$ when at least 50 percent of the reinforcement required for bending in the column strip passed through the critical section.

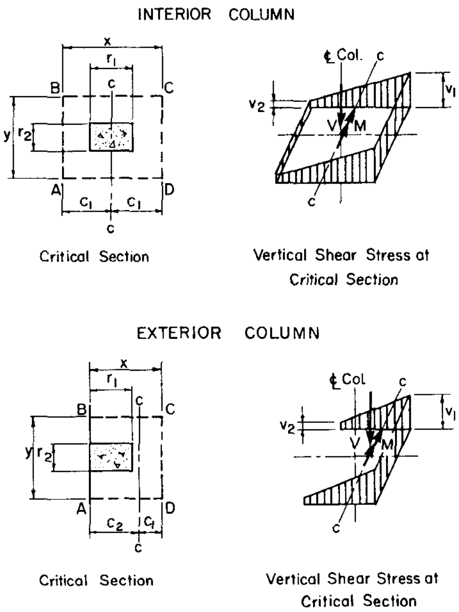


Fig. 1 — Assumed Vertical Shear Stress.

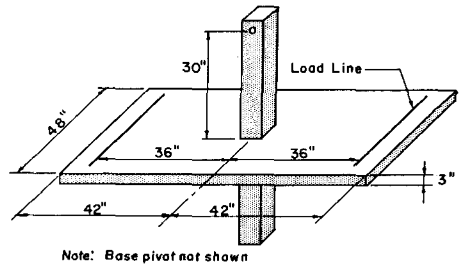


Fig. 2 — Test Specimen.

Johannes Moe.[6] An ultimate strength analysis was developed considering that the critical section is directly adjacent to the periphery of the column. For a slab-column junction subjected to combined bending and shear, the vertical shear stress, v_1, as shown in Fig. 1 is calculated from

$$v_1 = \frac{V}{\bar{A}_c} + \frac{K M c}{\bar{J}_c} \quad \cdots\cdots\cdots (2)$$

where, for a square interior column of side length r

$$\bar{A}_c = 4rd, \text{ and } \bar{J}_c = \frac{2r^3 d}{3}$$

In Eq. (2), K is a moment reduction factor which accounts for the fact that part of the moment is resisted by bending moments acting on faces AB and CD, and part by torsional moments due to horizontal shear stresses acting on faces BC and AD. All other terms are as previously defined.

Moe determined K experimentally by assuming that the maximum vertical shear stress at failure was equal to the shear stress at failure if the slab were loaded concentrically. He found that the ultimate shear strength of all of his slabs could be predicted with a standard deviation of 0.103 when K was taken equal to $\frac{1}{3}$. Moe recommended a limiting vertical shear stress of $(9.23 - 1.12\, r/d)\sqrt{f'_c}$ for r/d ratios less than 3 and $(2.5 + 10 d/r)\sqrt{f'_c}$ for r/d ratios greater than 3. These were conservative limits, intended to cause flexure rather than shear to govern ultimate strength.

Committee 326.[1] After reviewing Moe's work, ASCE-ACI Committee 326 selected a limiting shear stress of $4(d/r + 1)\sqrt{f'_c}$ on a critical section which follows the periphery of the column. It is significant that this is equivalent to requiring that the shear stress be limited to $4\sqrt{f'_c}$ on the critical section used in the 1963 ACI Building Code[10] and located at a distance d/2 outside of the column periphery. Committee 326 used the following expression to evaluate the results of 25 tests:

$$v_1 = \frac{V}{\dot{A}_c} + \frac{K M c}{\dot{J}_c} \dots\dots\dots (3)$$

where, for an interior column

$$\dot{A}_c = 2d\,(x + y)$$

$$\dot{J}_c = \frac{x^3 d}{6} + \frac{x d^3}{6} + \frac{x^2 y d}{2}$$

All other terms are as previously defined. Based on this evaluation, Committee 326 recommended limiting the shear stress to $4\sqrt{f'_c}$ on a design critical section located a distance d/2 from the column and assuming $K = 0.2$.

Commentary[9] *on 1963 ACI Building Code.* The Commentary also included a working stress method for evaluating the strength of the slab–column junction. This method uses the equation

$$v_1 = \frac{V}{\dot{A}_c} + \frac{(M - m_{AB} - m_{CD})\,c}{\dot{J}_c} \dots (4)$$

to calculate the maximum shear stress on a critical section defined in Fig. 1 by x and y equal to $(r_1 + d)$ and $(r_2 + 3t)$, respectively. All terms are as defined previously. This calculated shear stress is limited to allowable values specified in the 1963 ACI *Building Code.*[10]

EXPERIMENTAL INVESTIGATION

Specimen Description

Seventeen specimens were tested. Each was intended to represent, in reduced scale, an isolated portion of slab surrounding a column as shown in Fig. 2. The column had hinged reaction points 30 in. above and below the surfaces of the 3-in.-thick slab. Loads were applied to the slab along lines 36 in. from the centerline of the column.

Four different column and slab configurations were tested. These were designated as A, B, C, and D as shown in Fig. 3. The columns in configurations A, B, and C were located in the center of a 48 x 84-in. slab. These included 12 specimens with 6-in.-square columns located as in A, two specimens with a 6 x 12-in. column located as in B, and two specimens with a 12 x 6-in. column located as in C. The remaining specimen was made with a 6-in.-square column located adjacent to and centered along the long edge of a 48 x 45-in. slab, as shown in D.

The slabs were reinforced with two mats of No. 3 deformed bars spaced 3 in. center-to-center in each direction. The mats were placed so that the bars parallel to the long side of the slab were covered by 3/8 in. of concrete.

Pairs of 1 x 6-in. holes, as shown in Fig. 4, were blocked out of the slab in eight of

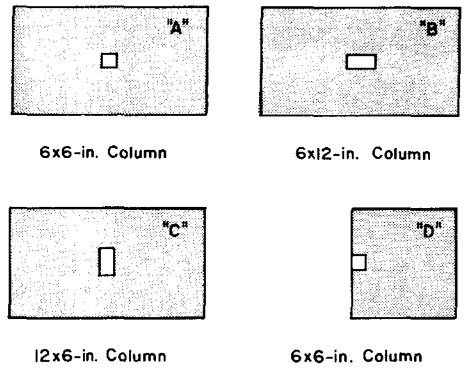


Fig. 3 — Slab-and-Column Configurations.

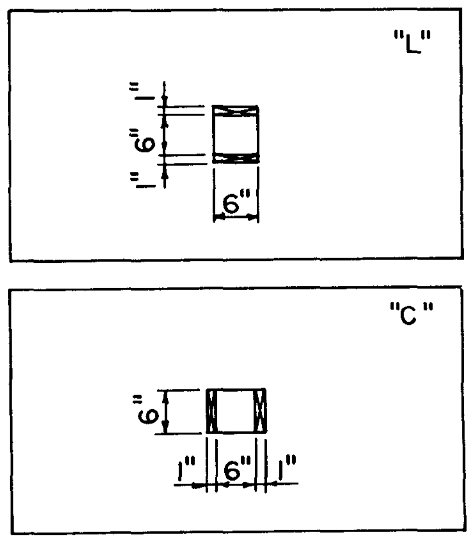


Fig. 4 — Holes Through Slabs.

TABLE 2—MATERIAL PROPERTIES

Specimen No.*	f'_c (psi)	f_y (ksi)
A1	4390	53.0
A2	4540	54.5
A3L	5370	52.8
A4L	4850	54.2
A5C	5080	53.9
A6C	5060	53.4
B7	4780	51.4
C8	4760	59.6
A9	5040	53.5
A10L	4480	51.4
A11C	4850	50.5
A12	4820	54.0
A13L	4760	53.7
A14C	5160	54.0
D15	4510	53.0
B16	4410	49.4
C17	5220	49.5

*The first letter indicates the specimen configuration, as shown in Fig. 3. The last letter, when used, indicates the specimens with holes, as shown in Fig. 4.

the specimens with square columns. These holes were located adjacent to the column and either parallel to the long side or the short side of the slab, indicated by L and C, respectively. The slab reinforcement was run through the holes.

The columns were reinforced with four No. 6 deformed bars. These bars were placed in the corners of the column with ½ in. of concrete cover on each side. Ties made from No. 2 smooth bars, spaced at 4 in. center-to-center, were used in the column outside of the slab region.

Materials

The concrete was made with Type I portland cement, Elgin sand and ⅜-in. maximum size gravel. Measured slumps ranged from one to three inches. Concrete strengths at the time of test are listed in Table 2. Each of the strengths listed is the average from tests on six 6 x 12-in. cylinders. Three of these cylinders were taken from each of the two batches of concrete used for each specimen.

The reinforcement, with the exception of the smooth No. 2 bars used as column ties, met the requirements of ASTM A 15-66[11] intermediate grade steel with deformations conforming to ASTM A305-65.[12] The yield strength of the slab reinforcement in each specimen is listed in Table 2.

Fabrication

The form for the test specimens was made of ¾-in. plywood suitably stiffened with 2 x 4-in. lumber, and it was constructed in such a manner that the entire specimen could be cast at one time. The form was oiled with a light form oil prior to casting. The reinforcing mats were tied together with iron wire and were supported on wire chairs.

Two 6-cu-ft batches of concrete were required to cast each specimen and its 6 x 12-in. test cylinders. The lower column and the slab were placed several hours before the upper column was cast. An internal vibrator was used to consolidate the concrete. The slab was screeded and the surface finished with a wooden float.

About eighteen hours after the concrete was placed, the slab surface was covered with wet burlap and plastic sheets. This moist curing continued for seven days. The specimens were then removed from the form and stored for another seven days in the laboratory at 73 F and 50 percent relative humidity prior to testing. The test cylinders were cast in steel molds. They were vibrated internally and were cured in the same way as the slab-column specimens.

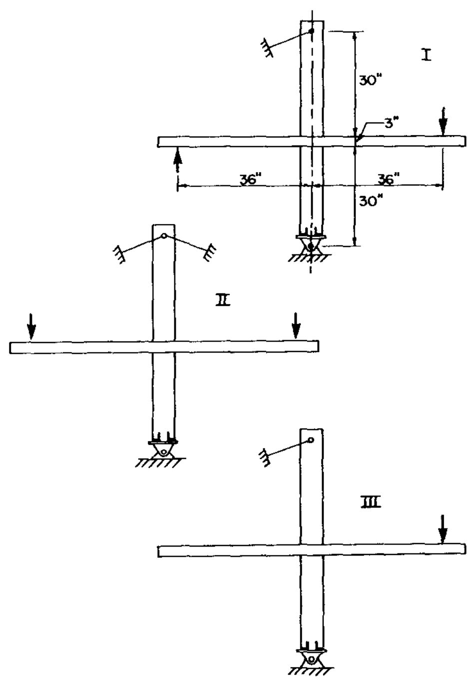


Fig. 5 — Loading Methods.

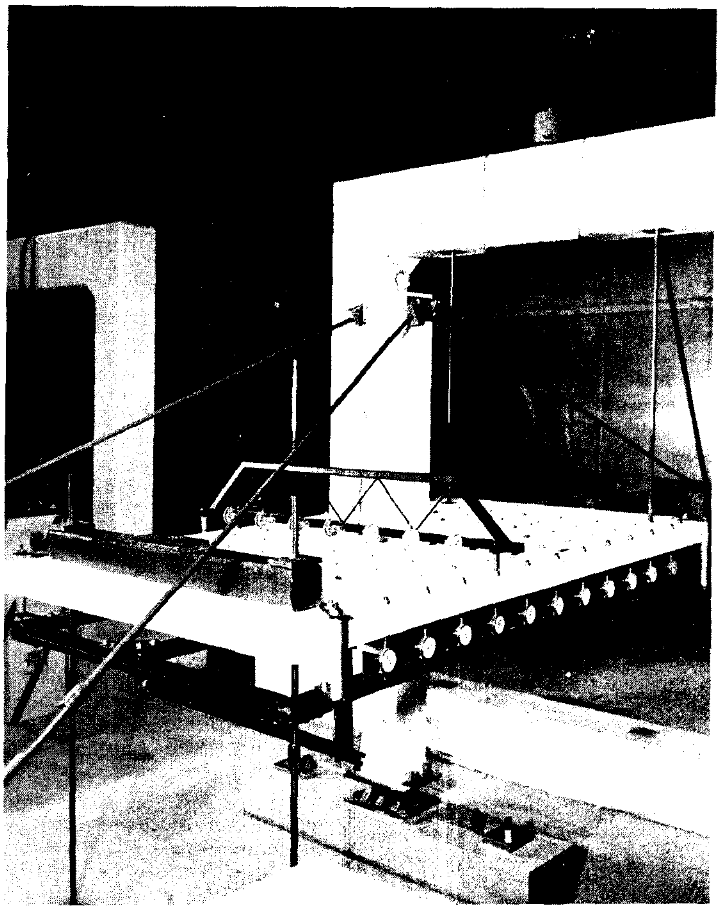


Fig. 6 — Type I Loading Method.

Test Procedure

The testing of the specimens was carried out by methods and procedures commonly used at the PCA Structural Laboratory and described in BULLETIN D33.(13) The specimen was supported by bolting the base of the column to a steel pivot that provided a hinged end condition. Steel angles were welded to the column reinforcement before casting to permit this bolted connection. The upper end of the column was connected to the test floor by inclined steel rods. A pivot arrangement provided a hinged end condition at the connection with the upper column.

Three different methods used to load the test specimens are illustrated in Fig. 5. The first method, designated as Type I, was intended to represent the case where the loads acting on a building are due primarily to lateral load caused by wind or earthquake. A downward line load was applied at one end of the slab, while an equal upward load acted at the other end so that moment, but no vertical load, was transferred from the slab to the column. The test setup for this method of loading is shown in Fig. 6.

The second method, designated as Type II, represents gravity loading. Equal downward line loads were applied at each end of the slab. This produced moment and shear in the slab without any transfer of moment into the column.

The *third method*, Type III, represents loading caused by a combination of lateral and gravity loads. A downward line load was applied at only one end of the slab. This produced combined moment and shear at the slab–column junction.

Three of the specimens subjected to the Type I loading were also subjected to reversal of loading. In these tests, the direction of the applied loads was reversed after reaching 25, 50 and 75 percent of the expected failure load.

Downward line loads on the slab were applied through a 4 x 4-in. steel tube crosshead 39 in. long as shown in Fig. 6. Each steel tube was seated in plaster on the slab surface. Two steel rods 36 in. apart connected the steel tube to a similar crosshead beneath the test floor. A 10-ton hydraulic ram at the center of the lower crosshead reacted against the underside of the test floor in the manner described in BULLETIN D33.[13] The hydraulic ram was connected by a flexible hydraulic hose to a pump and measuring unit.

To produce upward line loads, the steel tube was placed against the underside of the slab. The hydraulic ram reacted against an overhead concrete frame attached to the test floor, as shown in Fig. 6. The loads were measured by means of load cells placed between the hydraulic rams and their reaction.

Test loads were applied in increments of approximately 5 percent of the expected failure load. To apply a load increment,

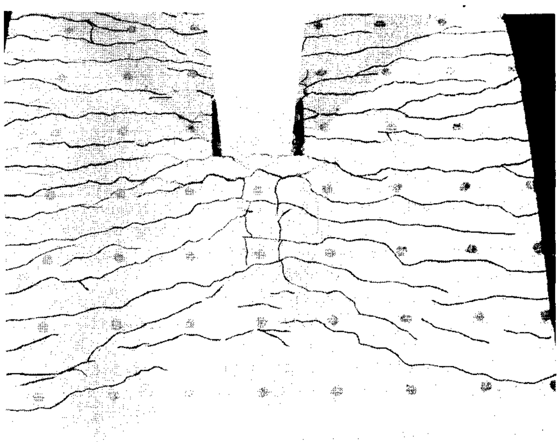


Fig. 7 — Tension Surface of Specimen A10L.

TABLE 3—TEST RESULTS

Specimen No.	Load Method[a]	Ultimate Moment Trransferred (in.-kips)	Ultimate Shear Transferred (kips)	Mode of Failure
A1	I	197.6	1.29	Shear
A2	I[b]	215.0	1.08	"
A3L	I	213.3	0.92	"
A4L	I[b]	210.7	1.08	"
A5C	I	139.6	1.14	"
A6C	I[b]	150.6	1.04	"
B7	I	316.0	1.10	"
C8	I	277.9	1.26	"
A9	II	6.2	14.13	Flexure
A10L	II	5.1	12.93	"
A11C	II	3.8	12.97	"
A12	III	181.4	6.04	Shear
A13L	III	175.9	5.88	"
A14C	III	118.9	4.30	"
D15	III	87.9	2.71	"
B16	III	242.0	7.73	"
C17	III	218.7	7.08	"

[a]See Fig. 5.
[b]Direction of loads reversed after 25%, 50%, and 75% of estimated ultimate load.

the hydraulic pressure in the loading ram was raised to a desired value. For the tests on specimens with rectangular columns, the load was then held constant for approximately three minutes. During this time data on rotation, cracking and load were recorded. About one hour was required to conduct these tests.

For the tests on specimens with square columns, the load was maintained until the deflection stabilized. At this time the hydraulic system was cut off from the pump. By following this procedure, the slab deflection pattern was held for the approximately 25 minutes required to measure deflections at 6-in. intervals over the slab surface. Deflections were measured with dial gages attached to steel frames, as shown in Fig. 6. After all data were recorded and all cracks were marked, the pump pressure was again raised to the previous cut-off point. The ram system was then reconnected to the hydraulic pump and another increment of load was applied. The average time to conduct each of these tests was about 5 hours.

Compressive strains were measured in the long direction of the slab at four locations along lines parallel to the short side of the slab and three inches from the column face. These measurements were made with 1-in. electrical resistance strain gages placed on the concrete surface.

Test Results

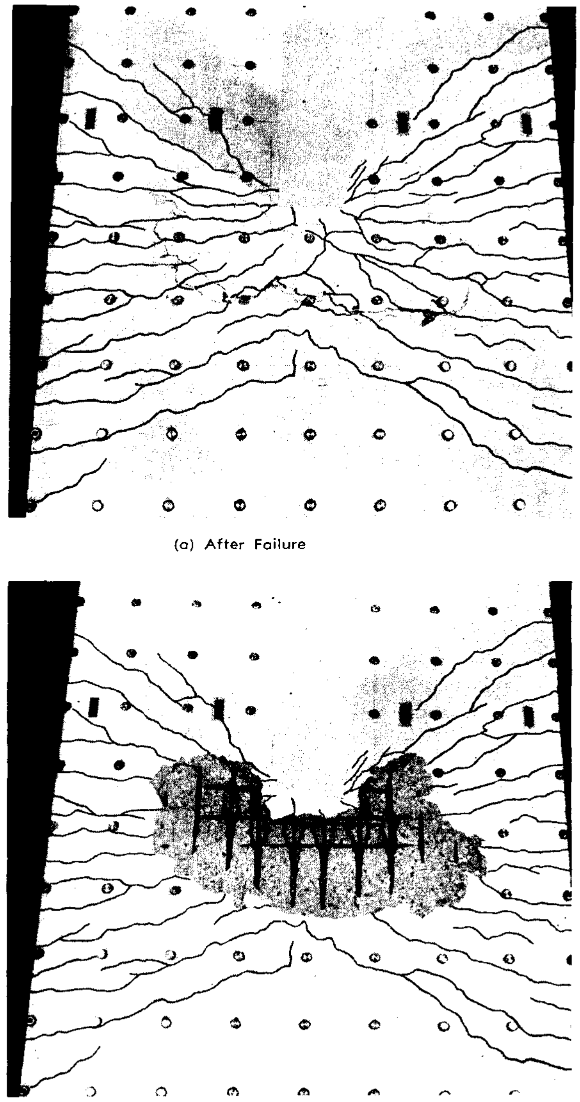
The ultimate moments and shears transferred from the slab to the column at the failure load of each test specimen are given in Table 3. The values include applied load and weight of the slab and loading equipment.

It may be noted that the ultimate moments transferred by Specimens A1 and A2, A3L and A4L, and A5C and A6C indicate a small spread of test results of 10 percent or less for similar conditions. The second of each of these pairs was subjected to the same type of loading as the first, except for reversal of loading. In two of the three duplications, the ultimate moment after reversals was higher than without reversals.

All of the test specimens except A9, A10L, and A11C failed in shear. These three specimens were subjected to the Type II loading and failed in flexure, as if the slab were acting as a wide beam. In all three tests the ultimate moment sustained by the slab was greater than the flexural capacity computed according to the 1963 ACI Building Code[10] assuming the full width of the slab along a section adjacent to the column face to be effective. The ultimate flexural strengths of Specimens A10L and A11C, with holes parallel to the long and short sides of the column, respectively, were very nearly equal and only about 10 percent less than that of the comparable slab without holes, Specimen A9. A photograph of the tension side of the slab of Specimen A10L after failure is shown in Fig. 7.

Shear failures occurred along inclined cracks that formed in the slab around the column. The failures were sudden, occurring when a truncated pyramid of concrete around the column "punched" through the slab. However, in two specimens, B16 and C17, the computed ultimate flexural capacity of the slab was exceeded before the shear failure occurred. A photograph of the tension surface of Specimen A12 after failure is shown in Fig. 8(a). Fig. 8(b) shows the same specimen after the broken concrete was removed. Other photographs of Specimens A3L, D15, and B7 are shown in Figs. 9(a), (b), and (c), respectively.

Holes located adjacent to the column and parallel to the long side of the slab had very little effect on the shear strength of the specimens. As may be observed from Table 3, the ultimate moment transferred by Specimens A1 and A2, compared to A3L and A4L, and A12 compared to A13L, are very nearly equal. In contrast, the strength of Specimens A5C and A6C, and A14C, with holes parallel to the short side of the slab, was reduced by 30 to 35 percent.

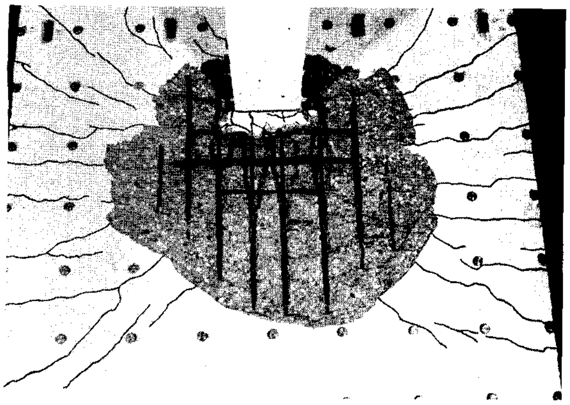


(a) After Failure

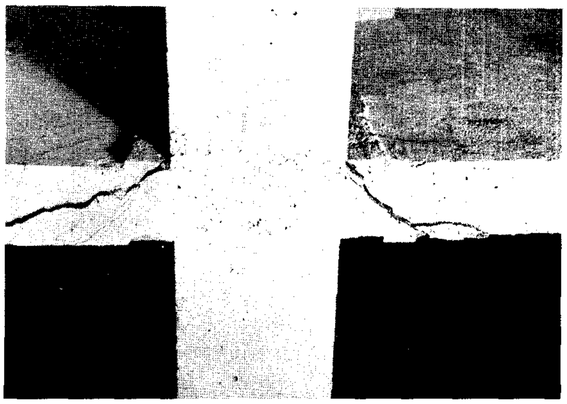
(b) Broken Concrete Removed

Fig. 8 — Tension Surface of Specimen A12.

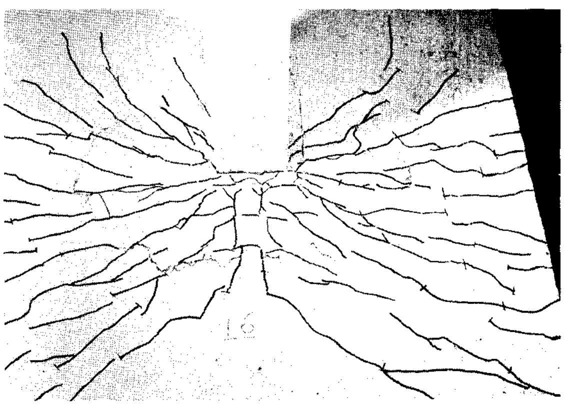
When the size of the column was increased, the ultimate moment transferred by the specimens also increased. Comparing



(a) Specimen A3L



(b) Specimen D15



(c) Specimen B7

Fig. 9—Views of Specimens A3L, D15, and B7 After Failure.

Specimen C8 to A1, and C17 to A12, it may be seen that doubling the width of the face of the column resisting the moment increased the strength by 20 to 40 percent. Similarly, Specimens B7 and B16, compared to A1 and A12, respectively, showed an increase in strength of 33 to 60 percent when the length of the face parallel to the long side of the slab was doubled.

Specimen D15 represented a column at the edge of a flat slab. Comparing Specimen D15 to A12, both of which were subjected to loading at one edge of the slab, it may be seen that the strength of the specimen representing an edge column was about 50 percent below that of an interior column.

The deflection measurements on the surface of the slab were used to construct contours of the slab deflection patterns. These measurements were intended to provide information on behavior around the column, but the accuracy of the measurements and the fineness of the grid, particularly around the column, were inadequate for this purpose. The deflection measurements were also used to draw curves showing the relationship between transferred moment and rotation, as shown for Specimens B7, C8, A12, B16, and C17 in Fig. 10 and Specimen A2 in Fig. 11. In these figures the rotation shown was calculated from the deflection at the load point with respect to a line normal to the column at the joint. The load stage at which the first cracking of the slab was observed has been marked on these curves.

STRENGTH OF THE SLAB–COLUMN JUNCTION

Four related methods for predicting the strength of the slab–column junction under combined shear and moment were described earlier. Each method is based on the establishment of a critical section around the column and the calculation of a nominal limiting shear stress on that section.

Before evaluating the strength of the test specimens as predicted by these four methods, some general observations should be made about the stress distribution on the critical section. This stress distribution is necessarily very complex, because of inclined cracking in the slab around the column. However, well established design practice involves computation of a nominal shear stress by assuming that the shear force is uniformly distributed over an area defined by the width and effective depth of a concrete member. A logical extension of this practice is to assume that an unbalanced moment creates additional shearing stresses which, over part of the critical section, add to the direct shear stresses. The maximum combined shear stress thus obtained then becomes the criterion on which the strength of the junction is based.

Actually, only part of the total moment transferred at the slab–column junction creates shearing stresses that add to the direct shear stresses. Part of the moment is resisted by flexural stresses acting on sides AB and CD of the critical section shown in Fig. 1. The remainder of the moment is resisted by both vertical and horizontal shear stresses acting on sides BC and AD, and by vertical shear stresses acting on sides AB and CD. All four methods assume that the vertical shear stresses vary linearly with the distance from the centroidal axis. This assumption leads to expressions having the form of Eqs. (1), (2), (3), and (4). These equations differ primarily in location of the critical section and the portion of the moment assumed to produce the added shear stress.

Comparison of Measured and Computed Strengths

Both Moe's method and the method recommended by Committee 326 assume that the added shear stress is proportional to the total moment transferred, i.e., to KM. In Eq. (2), $\bar{J}_c$ is equal to $\int x^2 dA$ taken over the entire area of the critical section, whereas in Eq. (3), $\dot{J}_c$ is equal to $\int x^2 dA$ taken over the entire area plus $\int y^2 dA$ taken over sides BC and DA of the critical section. In these integrals x and y are horizontal and vertical distances, respectively, from the centroidal axis to an element of area dA. Thus KM in Eq. (2) is that part of the total moment which produces only vertical shear stress, while KM in Eq. (3) is that part of the total moment which produces both vertical and horizontal shearing stress. This does not explain the difference between the K-values of 0.33 recommended by Moe for use in Eq. (2) and 0.2 recommended by Committee 326 for use in Eq. (3). On the contrary, this reasoning would indicate that the K-value in Eq. (3) should be greater than in Eq. (2).

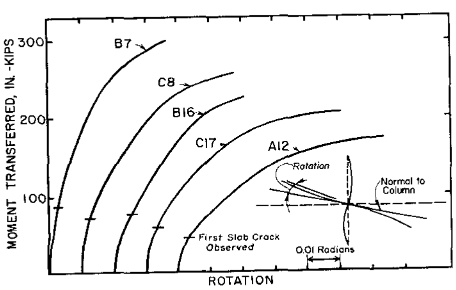


Fig. 10 — Moment–Rotation Curves.

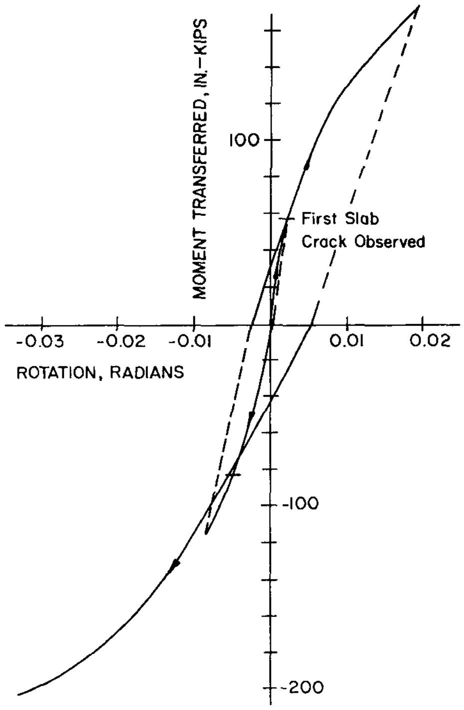


Fig. 11 — Moment–Rotation Curve for A2.

Di Stasio and Van Buren's method, and the method in the 1963 ACI Building Code Commentary, assume that the added shear stress is proportional to the total moment transferred minus the moment resisted by sides AB and CD of the critical section, i.e. to $(M - m_{AB} - m_{CD})$. Since the moment resisted by sides AB and CD is a predetermined quantity, shear stress is produced only when M exceeds $m_{AB} + m_{CD}$.

Di Stasio and Van Buren. In this evaluation of the strength of the test specimens, the Di Stasio and Van Buren method was modified to conform to the 1963 ACI Building Code. Consequently, the shear working stress was assumed to be $2\sqrt{f'_c}$, and the critical section was taken at a distance equal to half the effective slab depth outside of the column periphery. For simplicity, C_1 was assumed equal to one, thus neglecting dowel action of the reinforcement. The $\frac{7}{8}$ term for j was dropped. Furthermore, holes located at a distance from the column of less than ten times the thickness of the slab were assumed to reduce the periphery of the critical section by radial projections of the openings to the centroid of the column.

For the case of shear transfer without moment transfer, the maximum shear stress, from Eq. (1), is

$$v_1 = \frac{V}{A_c}\,\frac{t}{d} \qquad \text{(5)}$$

Therefore the working stress capacity of the connection is

$$V_w = 2\sqrt{f'_c}\,A_c\,\frac{d}{t} \qquad \text{(6)}$$

For the case of moment transfer without shear transfer, the maximum shear stress is

$$v = \left[\frac{M - m_{AB} - m_{CD}}{J_c}\right]\frac{ct}{d} \qquad \text{(7)}$$

where c is equal to c_1 for an interior column, or the greater of c_1 or c_2 for an exterior column. Letting m_r be equal to the resisting flexural moment of the slab, m_{AB} plus m_{CD}, the working load moment capacity of the connection is

$$M_w = m_r + 2\sqrt{f'_c}\,\frac{dJ_c}{ct} \qquad \text{(8)}$$

For intermediate cases where $v_1 = 2\sqrt{f'_c}$ controls, when the moment is less than M_w, the shear capacity of the connection is

$$V = A_c\left[2\sqrt{f'_c}\,\frac{d}{t} - \left(\frac{M - m_r}{J_c}\right)c_1\right] \qquad \text{(9)}$$

TABLE 4—ANALYSIS OF TEST DATA

Specimen No.	Di Stasio & Van Buren V_u/V_w	Di Stasio & Van Buren $(M_u-2m_r)/(M_w-m_r)$	Moe V_u/V_o	Moe M_u/M_o	Committee 326 V_u/V_o	Committee 326 M_u/M_o	Code Commentary V_u/V_w	Code Commentary $(M_u-2m_r)/(M_w-m_r)$
Tests Reported in This Paper								
A1	0.12	4.59	0.05	1.31	0.06	0.63	0.08	2.12
A2	0.10	5.04	0.04	1.40	0.05	0.68	0.07	2.43
A3L	0.15	6.31	0.07	1.71	0.08	0.84	0.09	2.66
A4L	0.19	6.54	0.08	1.79	0.09	0.87	0.11	2.74
A5C	0.20	11.5	0.08	3.45	0.10	1.57	0.11	2.28
A6C	0.18	12.7	0.08	3.72	0.09	1.69	0.10	2.68
B7	0.07	4.03	0.04	1.03	0.04	0.49	0.06	2.29
C8	0.08	3.76	0.04	1.28	0.04	0.56	0.06	2.15
A12	0.53	4.67	0.23	1.15	0.26	0.55	0.38	2.60
A13L	1.04	6.23	0.45	1.50	0.52	0.74	0.58	3.03
A14C	0.73	10.9	0.32	2.90	0.36	1.32	0.41	2.84
D15	0.36	9.65	0.14	0.77	0.18	0.44	0.28	4.06
B16	0.52	3.43	0.26	0.82	0.26	0.39	0.41	2.09
C17	0.44	3.32	0.22	0.97	0.22	0.42	0.34	2.14
Tests Reported by Moe in Bulletin D47[6]								
M1A	2.69	0	1.17	0	1.34	0	2.25	0
M2A	1.59	1.16	0.69	0.44	0.79	0.21	1.32	0.20
M4A	0.98	2.05	0.43	0.62	0.49	0.30	0.82	0.82
M2	1.70	1.62	0.74	0.50	0.84	0.24	1.41	0.61
M3	1.26	2.06	0.55	0.63	0.64	0.30	1.05	0.94
M6	1.58	1.42	0.68	0.48	0.79	0.22	1.28	0.40
M7	2.03	0.15	0.88	0.21	1.01	0.10	1.66	0
M8	1.00	2.24	0.43	0.75	0.50	0.35	0.82	0.70
M9	1.81	0.94	0.78	0.39	0.90	0.18	1.48	0.12
M10	1.30	1.89	0.56	0.70	0.65	0.32	1.05	0.43

Combining Eqs. (6), (8), and (9) gives the strength interaction equation

$$\frac{V}{V_w} = 1 - \frac{M - m_r}{M_w - m_r} \quad \ldots\ldots (10)$$

Now assuming that the working shear, V, and moment, M, for the test specimens are one half of the ultimate shear, V_u, and moment, M_u, respectively, Eq. (10) becomes

$$\frac{V_u}{V_w} = 2 - \frac{M_u - 2\,m_r}{M_w - m_r} \quad \ldots\ldots (11)$$

The values of V_u/V_w and $(M_u - 2\,m_r)/(M_w - m_r)$ for the test specimens listed in Tables 1 and 3 are given in Table 4. Test specimens that failed in flexure were not included. In calculating m_r for the test specimens with holes, it was assumed that the reinforcement running through the holes was stressed to its working stress limit. These values are also plotted as an interaction diagram in Fig. 12. It may be seen that all of the experimental points are outside of the dashed line that represents Eq. (11). This indicates that the working shear stress of $2\sqrt{f'_c}$ was conservative for a safety factor of 2.

Johannes Moe. In Moe's method, the critical section is assumed to be located adjacent to the column periphery. The ultimate shear stress is calculated from Eq. (2). For the case of shear transfer without moment transfer, the ultimate shear capacity of the connection is

$$V_o = v_u \bar{A}_c \quad \ldots\ldots\ldots\ldots (12)$$

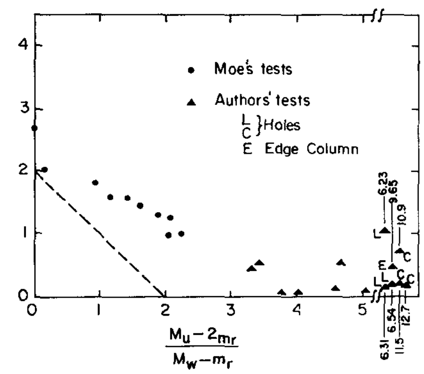


Fig. 12 — Working Stress Method Recommended by Di Stasio and Van Buren.

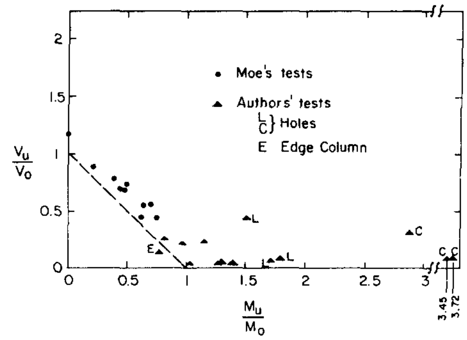


Fig. 13 — Ultimate Strength Method Recommended by Moe

where v_u was assumed equal to $(9.23 - 1.12\, r/d)\sqrt{f'_c}$ since r/d was less than 3 in all of the tests in Tables 1 and 3. Similarly, for the case of moment transfer without shear transfer, the ultimate moment capacity of the connection is

$$M_o = \frac{v_u \bar{J}_c}{K\,c} \quad \ldots\ldots\ldots\ldots (13)$$

where c is equal to c_1 for an interior column or the greater of c_1 or c_2 for an exterior column.

For intermediate cases where v_1 controls, the ultimate shear capacity of the connection is

$$V_u = \bar{A}_c \left[v_u - \frac{K M_u c}{\bar{J}_c} \right] \quad \ldots\ldots (14)$$

Combining Eqs. (12), (13), and (14) gives the strength equation

$$\frac{V_u}{V_o} = 1 - \frac{M_u}{M_o}$$

The values of V_u/V_o and M_u/M_o, assuming K equal to 1/3 for tests reported in this paper and in BULLETIN D47,[6] are given in Table 4 and plotted in Fig. 13. It may be seen that Moe's method conservatively predicted the ultimate strength of all but one of these tests.

Committee 326. The method of computing the strength of the slab–column junction recommended by Committee 326 leads to an equation similar to Moe's. The critical section is assumed to be located a distance d/2 from the face of the column. The ultimate shear stress, v_u, is $4\sqrt{f'_c}$. Values of V_u/V_o and M_u/M_o, assuming K equal to

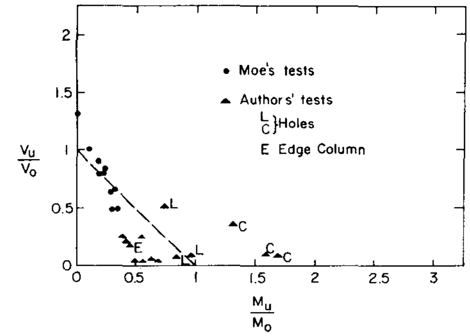


Fig. 14 — Ultimate Strength Method Recommended by Committee 326 (K = 0.2)

0.2, are given in Table 4 and plotted in Fig. 14. It is evident that this method with K = 0.2 overestimates the strength of many of the test specimens. The committee studied statistical summaries of test data rather than the type of interaction curves used here.

Commentary on 1963 ACI Building Code. The method presented in the Commentary is similar to the Di Stasio and Van Buren working stress method, except that faces AB and CD of the critical section are assumed to have a width of $r_2 + 3t$. In effect this increases the magnitude of the moment transferred by the slab to the column before any of this moment adds to the shear stress. As may be observed from Fig. 15, this method is satisfactory for the tests reported in this paper. However, it leads to a

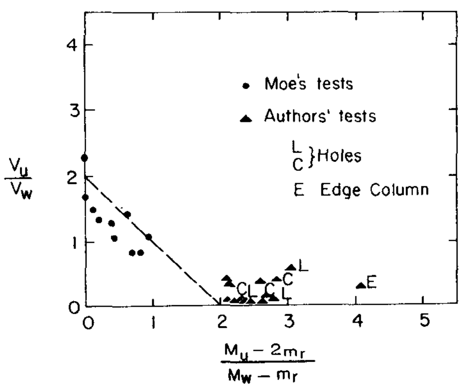


Fig. 15 — Working Stress Method from Commentary on 1963 ACI Code.

factor of safety less than 2 for all except three of Moe's tests.

Discussion of Design Methods

Several observations can be made concerning the comparison of the test data with the strength predicted by these four methods. First, the treatment of the holes according to Section 920(b) of the 1963 ACI Building Code was conservative. This observation, however, is tempered by the fact that running the reinforcement through the holes undoubtedly increased the strength of the test specimens.

Comparing the Di Stasio and Van Buren method with the method in the 1963 Code Commentary indicates that the analysis is extremely sensitive to the dimensions chosen for the critical section. Furthermore, Figs. 12 and 15 show that these two methods had a varying degree of safety that is dependent upon the magnitude of the difference between M and m_r. This indicates that calculating the vertical shear stress as a function of this difference is not in accord with the behavior of the test specimens.

Furthermore, it should be pointed out that these two methods permit a designer, by concentrating reinforcement over the column, to increase the calculated value of m_r, thus seemingly increasing the shear strength of the connection. However, Moe, in BULLETIN D47,[6] concluded from tests on centrally loaded slabs that concentration of the bending reinforcement in narrow bands across the column does not increase the shear strength. Tsuboi and Kawaguchi's tests[3] with moment alone also showed little increase in strength when the reinforcement was concentrated over the column.

Moe's method of analysis provided reasonable and yet conservative agreement with the test data for all except the specimen representing the edge column. This method correctly reflects the influence on shear strength of column size, slab thickness, and concrete tensile strength. Furthermore, Fig. 13 shows that Moe's method predicted the ultimate strength of the specimens without holes with a reasonably uniform degree of conservatism.

Finally, the Committee 326 method has two important advantages. It bases the analysis of shear strength on a critical section which is familiar to designers, and yet it retains the important principles established in Moe's work. This method is equally suited to working stress or ultimate

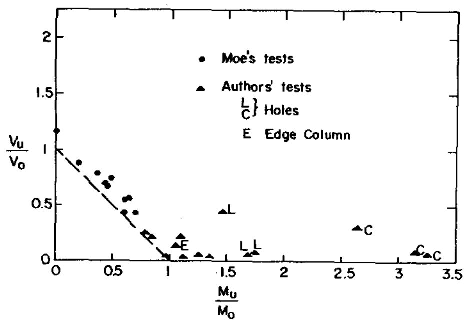


Fig. 16—Modified Committee 326 Method ($K = 0.4$).

strength design, simply by adjusting the value of the critical stress. However, Fig. 14 clearly indicates that the method needs modification. This may be accomplished by increasing the value of K to 0.4. As shown in Fig. 16, this provides a conservative prediction of the strength for all of the test specimens.

CONCLUSIONS

Examination of the tests reported in this paper and previous tests reported by Moe leads to the following conclusions:

1. The working stress method in Section 2102 of the Commentary on the 1963 ACI Building Code(9) was found to have a factor of safety less than 2 for some of the slab-column junctions under combined shear and moment.
2. The working stress method recommended by Di Stasio and Van Buren,(7) modified to agree with the 1963 ACI Code, was found to have a variable factor of safety always greater than 2. However, the method did not agree with the trend of the test data.
3. The ultimate strength design method recommended by Moe in BULLETIN D47(6) was found to be simple in application and to give good results.
4. The ultimate strength design method recommended by ACI-ASCE Committee 326(1) was found to give a good prediction of the strength of the slab–column connection only when the moment reduction factor, K, was changed from 0.2 to 0.4.

PRACTICAL APPLICATION

The investigation reported herein indicates that the following design criteria for shear strength of slabs can be recommended for use in conjunction with Section 1707 of the 1963 ACI Building Code:

"When unbalanced gravity load, wind, or earthquake cause transfer of bending moment between column and slab, 60 percent of the moment shall be considered transferred by flexure across the periphery of the critical section defined in Section 1707(b), and 40 percent by eccentricity of the shear about the centroid of the critical section. Shear stresses shall be taken as varying linearly about the centroid of the critical section and the maximum shear stress, v_u, shall not exceed $4\phi\sqrt{f'_c}$."

NOTATION

$A_c, \bar{A}_c, \dot{A}_c$ = area of concrete in assumed critical section, periphery times effective slab depth, d

A_s = area of steel passing through A_c

C_1 = constant used in Eq. (1)

c, c_1, c_2 = distance from centroidal axis to the most remote part of critical section

d = effective depth of slab

f'_c = compressive strength of concrete

f_y = yield strength of reinforcement

$J_c, \bar{J}_c, \dot{J}_c$ = property of the assumed critical section analagous to polar moment of inertia

K = moment reduction factor

M = unbalanced moment (direction indicated by right-hand rule)

M_o = ultimate strength unbalanced moment capacity when $V = 0$

M_u = ultimate unbalanced moment

M_w = working stress unbalanced moment capacity when $V = 0$

m_{AB} = flexural moment acting on face AB of critical section

m_{CD} = flexural moment acting on face CD of critical section

m_r = sum of flexural moments acting on face AB and CD of critical section

n = ratio of modulus of elasticity of concrete to modulus of elasticity of steel

p = reinforcement ratio, based on effective depth, resisting M

r = side length of square columns

r_1 = width of face of column parallel to plane of M

r_2 = width of face of column perpendicular to plane of M

t = thickness of slab

V = shear

V_o = ultimate strength shear capacity when $M = 0$

V_u = ultimate shear

V_w = working stress shear capacity when $M = 0$

v, v_1, v_2 = vertical shear stress

v_u = ultimate shear stress

x = dimension of critical section as shown in Fig. 1

y = dimension of critical section as shown in Fig. 1

ϕ = capacity reduction factor used in 1963 ACI Building Code

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"Shear and Moment Transfer Between Concrete Slabs and Columns"

KEY WORDS: columns (structural); concrete slabs; cyclic loads; flat plates (concrete); flexural tests; openings; punching shear; shear tests; structural analysis

SYNOPSIS: The strength of flat slabs near columns was investigated by 17 tests involving combined shear and unbalanced moment loadings. Sixteen reinforced concrete slab–column specimens containing square or rectangular interior columns and one specimen with a square edge column were tested. Narrow rectangular holes were located adjacent to the columns in eight of the specimens. Reversals of loading simulating earthquake effects were applied to three of the specimens. Four design methods for shear strength, presented by Di Stasio and Van Buren, Johannes Moe, ACI-ASCE Committee 326, and the Commentary on the 1963 ACI Building Code, are evaluated in terms of Moe's data and test data reported herein. A modification of the Committee 326 method is recommended for practical design.

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SYNOPSIS: The strength of flat slabs near columns was investigated by 17 tests involving combined shear and unbalanced moment loadings. Sixteen reinforced concrete slab–column specimens containing square or rectangular interior columns and one specimen with a square edge column were tested. Narrow rectangular holes were located adjacent to the columns in eight of the specimens. Reversals of loading simulating earthquake effects were applied to three of the specimens. Four design methods for shear strength, presented by Di Stasio and Van Buren, Johannes Moe, ACI-ASCE Committee 326, and the Commentary on the 1963 ACI Building Code, are evaluated in terms of Moe's data and test data reported herein. A modification of the Committee 326 method is recommended for practical design.

REFERENCE: Hanson, Norman W., and Hanson, John M., *Journal of the PCA Research and Development Laboratories* (Portland Cement Association, U.S.A.), Vol. 10, No. 1, January 1968, pp. 2-16; *PCA Development Department Bulletin D129.*

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