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A Study of Hooked Bar Anchorages in Beam-Column Joints

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Twenty-two specimens simulating typical exterior beam-column joints in a structure were tested to evaluate the capacity of anchored beam reinforcement subjected to varying degrees of confinement at the joint. The effects of column axial load, vertical column reinforcement, side concrete cover, and lateral reinforcement through the joint on the performance of standard hooked bars were studied. The tests were conducted using either #7 or #11 beam bars anchored in the columns. Standard 90 or 180 deg hooks conforming to ACI 318-71 specifications were used throughout.

Based on observations of the test specimens, current ACI 318-71 Code specifications are studied and design recommendations for hooked bar anchorages proposed. Comparison of the proposed design recommendations with the measured results indicated that higher stresses could be allowed for hooked anchorages provided that confinement in the form of cover or ties and straight lead embedment before the hook were sufficient.

Keywords: anchorage (structural); beam-column frame; beams (supports); hooked reinforcement; joints (junctions); reinforced concrete; structural engineering.

EXPERIMENTAL PROGRAM

■ IN THE FIRST PHASE OF THIS INVESTIGATION, [1,2] the specimens consisted of relatively short bars embedded in small concrete blocks. The anchored lengths were short to insure that bond failure would occur before the steel yielded. In addition, the side cover on the bars was sufficient to prevent fracturing the concrete block so that the bars could be considered as anchored in mass concrete. To evaluate the anchorage capacity of the hooked

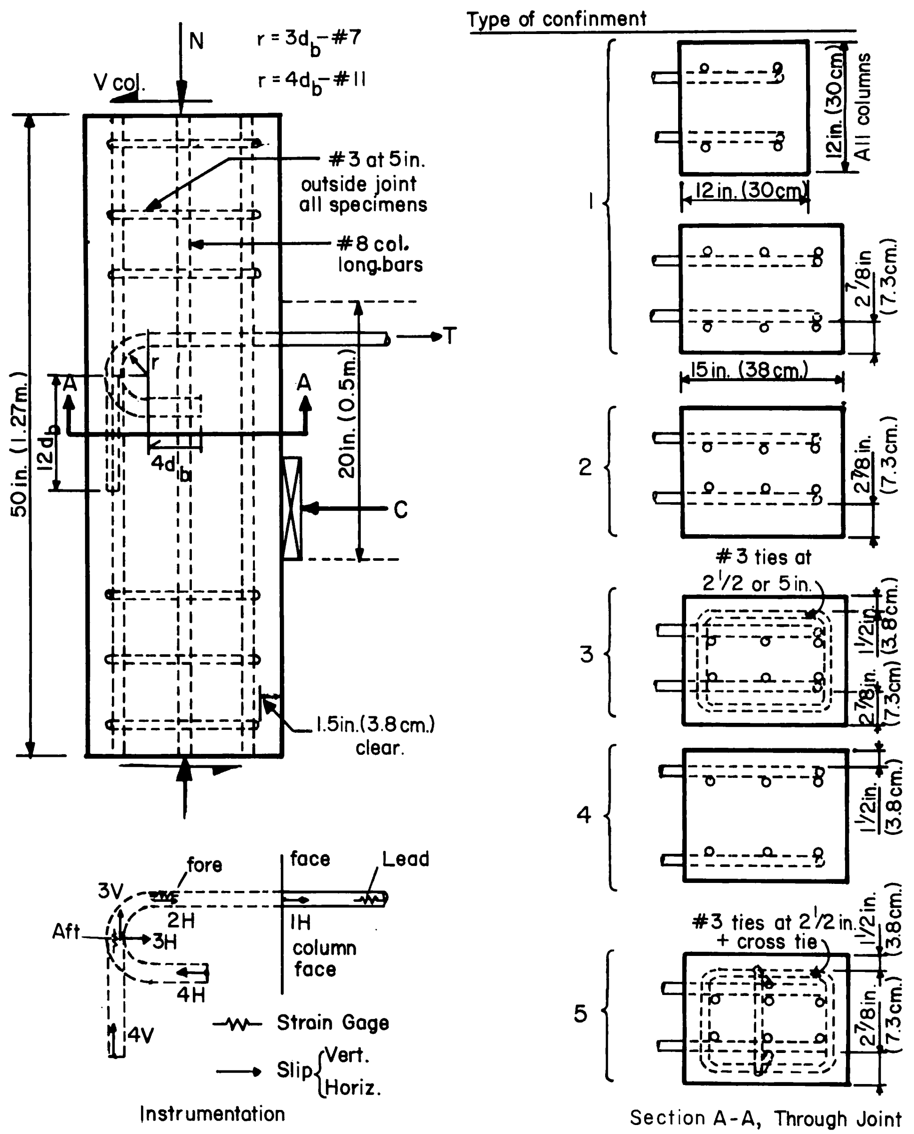

Fig. 1—Test specimens

bars under more realistic conditions, the specimens studied in this phase were full-scale models of beam-column joints in order to eliminate scale effects and to permit the use of fairly large diameter hooked bars which conformed to ACI standards for hook geometry.

Test specimens

A diagram of the specimen simulating a typical exterior beam-column joint is shown in Fig. 1. The length of the column was chosen to permit the embedment of the hooked bars and to eliminate lateral constraints at the joint region produced by the axial loading heads. To facilitate fabrication the beams were not cast with the columns. The anchored bars extended past the face of the column and were connected to threaded rods which were loaded with hydraulic rams. The compression zone of the beam was duplicated with a steel plate bearing against the face of the column over an area which approximated that of the compression zone of the assumed beam.

Variables—Table 1 summarizes the properties of the 22 specimens tested in this study. Fig. 1 shows plan and side views of the columns at the point where the beam bars are anchored in the joint. The variables considered and the range of those variables are discussed below.

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TABLE I—PROPERTIES OF TEST SPECIMENS

Specimen	Col. size, in.	Hook angle, deg.	l_l, Lead embed., in.	Lateral confinement*	N, Col. load, k	Conc. strength, f_c', ksi	
						Bottom batch	Top batch
J7-90-15-1-H	12 x 15	90	9.5	1	545	4.6	4.6
J7-90-15-1-M	12 x 15	90	9.5	1	269	5.0	5.1
J7-90-15-1-L	12 x 15	90	9.5	1	145	4.8	4.8
J7-90-12-1-H	12 x 12	90	6.5	1	420	4.4	3.9
J7-180-15-1-H	12 x 15	180	9.5	1	545	3.9	4.1
J7-180-12-1-H	12 x 12	180	6.5	1	425	4.0	4.7
J7-90-15-2-H	12 x 15	90	9.5	2	545	4.8	4.7
J7-90-15-2-M	12 x 15	90	9.5	2	274	4.7	4.8
J7-90-15-3-H	12 x 15	90	9.5	3	555	4.5	4.8
J7-90-15-3a-H	12 x 15	90	9.5	3a	535	3.9	3.6
J7-90-15-4-H	12 x 15	90	9.5	4	548	4.4	4.6
J11-90-15-1-H	12 x 15	90	6.0	1	540	4.8	5.0
J11-90-15-1-L	12 x 15	90	6.0	1	154	4.8	4.7
J11-90-12-1-H	12 x 12	90	3.0	1	437	4.5	4.7
J11-180-15-1-H	12 x 15	180	6.0	1	540	4.2	4.6
J11-180-15-1-L	12 x 15	180	6.0	1	140	4.3	4.4
J11-90-15-2-H	12 x 15	90	6.0	2	540	5.1	4.9
J11-90-15-2-L	12 x 15	90	6.0	2	125	4.6	4.4
J11-90-15-3-L	12 x 15	90	6.0	3	150	4.9	4.8
J11-90-15-3a-L	12 x 15	90	6.0	3a	175	5.0	5.0
J11-90-15-4-L	12 x 15	90	6.0	4	140	4.1	4.1
J11-90-15-5-L	12 x 15	90	6.0	5	140	5.1	4.9

*Lateral confinement:
1. Col. bars + 2⅞ in. cover
2. Only 2⅞ in. cover
3. 2⅞ in. cover + #3 ties at 5 in. through joint
3a. 2⅞ in. cover + #3 ties at 2.5 in. through joint
4. Only 1½ in. cover
5. 2⅞ in. cover + #3 ties (with a crosstie) at 2½ in. through joint.

Note: 1 in. = 2.54 cm; 1 kip = 0.454 kg; 1 ksi = 70.3 kgf/cm²

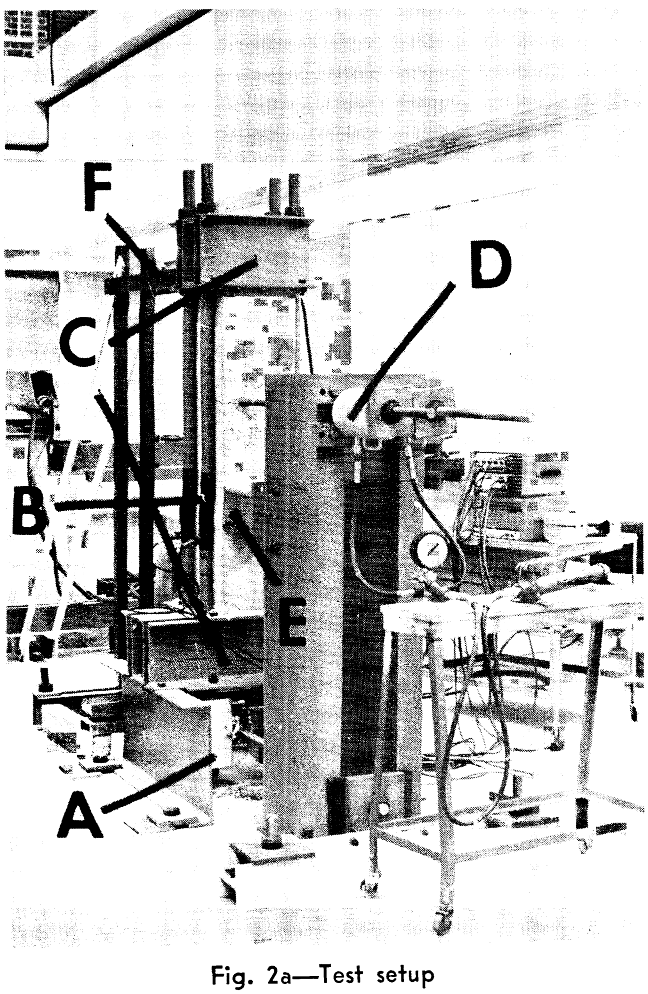


Fig. 2a—Test setup

(a) *Size of anchored bars.* The tests were conducted with either #7 (22 mm) or #11 (35 mm) beam bars anchored in the columns.

(b) *Hook geometry.* All the tests were conducted using either 90 or 180 deg hooks conforming to ACI 318-71[4] specifications (Section 7.1).

(c) *Lead embedment length.* By varying the size of the column, the lead embedment before the hook portion of the anchored bar was varied. For example, the lead embedment for a #7 (22 mm) bar anchored in a 12 x 15 in. (30 x 38 cm) column, was 9.5 in. (24 cm). For a 12 x 12 in. (30 x 30 cm) column this was reduced to 6.5 in. (16.5 cm).

(d) *Confinement.* Three types of confinement were considered. First, the influence of the longitudinal column bars; second, the influence of column ties through the joint; and third, the influence of concrete cover. To determine the influence of the location of column bars, companion tests were run with column bars placed inside or outside the anchored beam bars with concrete cover over the anchored beam bars of 2⅞ in. (7.3 cm). The effect of the ties through the joint was isolated by placing the column bars inside the beam bars, and carrying ties through the joint. In this case, the confinement consisted of 2⅞ in. concrete cover plus #3 (9.5 mm) ties at either 5 or 2½ in. (12.7 or 6.4 cm) spacing through the joint. The effect of concrete cover was determined by reducing cover from 2⅞ in. (7.3 cm) to 1½ in. (3.8 cm) and placing the column bars inside the beam bars so only clear concrete cover confined the anchored beam bars.

(e) *Column axial load.* It has been felt for some time that column axial loads would have a beneficial effect on anchorage strength of bars in joints. Some limited tests[6] indicate that normal pressure reduces the tendency for splitting the cover on an anchored bar. To determine the influence of column loads, tests were run holding all specimen dimensions and reinforcement constant and varying the level of axial load. Nominal axial loads of 135, 270 and 540 kip (61, 123, and 245 tonnes) were imposed on the 12 x 15 in. (30 x 38 cm) columns. To retain the same stress levels the 540 kip load was reduced to 420 kip in the 12 x 12 in. (30 x 30 cm) columns.

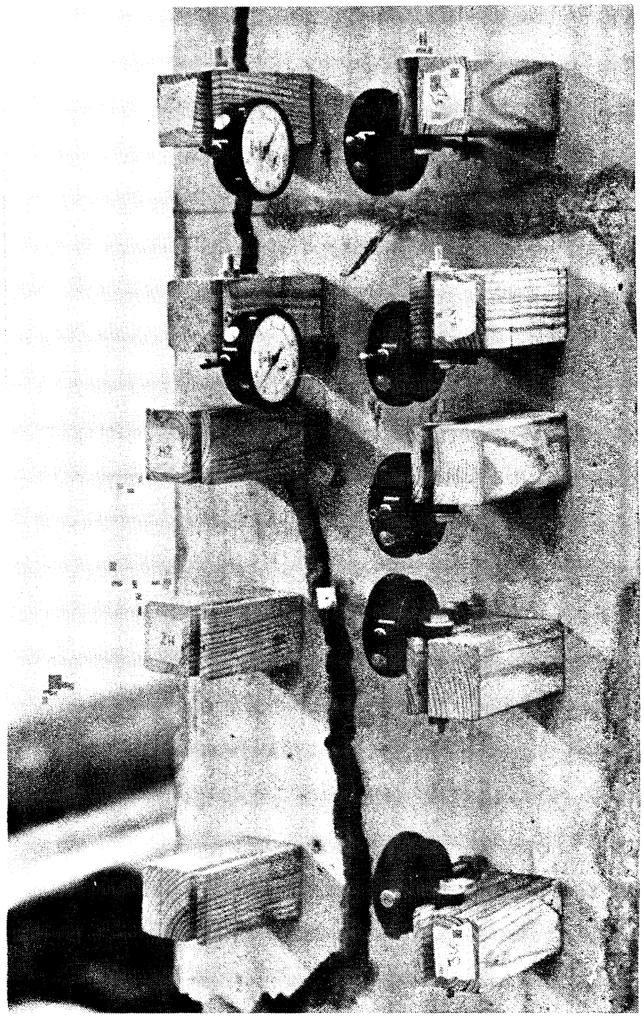
For recognition of the variables in each specimen the following notation was used. For example, specimen J7-90-15-1-L: *J* signifies a joint test, #7 longitudinal beam reinforcement, 90 deg the angle of bend, 15 in. column thickness, 1 indicates the type of confinement and *L* refers to the level of nominal column axial load. The lateral confinement designations are indicated in Table 1. The designation for nominal axial load refers to the three levels, High, Medium or Low.

Material properties—The bars used throughout the study had the same deformation pattern. All reinforcement was Grade 60 with yield strengths ranging from 64 to 68 ksi (4500 to 4750 kgf/cm²). One basic mix of concrete was used throughout the study. The mix proportions were selected to give an average compressive strength f_c' of 4500 psi (316 kgf/cm²) at 14 to 21 days. All the specimens and control cylinders were cured at room temperatures. Each specimen required casting two batches of concrete. The first batch completed the column to a level just below the horizontal portion of the anchored beam bars. The concrete strengths for both batches are listed in Table 1.

Test procedure

Loading—A loading frame shown in Fig. 2a was constructed to apply axial loads to the column, and tensile and compressive beam forces to the joint. To load the column four 100 ton (91 tonne) center-hole rams (A) were placed below the bottom support platform (C) and alloy steel rods (B) instrumented to serve as load cells were passed through the rams and bottom platform along the side faces of the column to the top head (C). The anchored beam bars protruded from the column about 12 in. (30 cm). Threaded rods were attached to the reinforcing bars using a proprietary thermically welded splice. Two center-hole hydraulic rams (D) were placed over the threaded rods and reacted against a column made up of channels. The reaction column transferred a compressive load to the specimen through a large box section (E) to simulate the compressive zone of the beam. A horizontal reaction (F) was provided at the top of the test column to balance the moment imposed by the simulated beam. The

Wires attached to bars



Dial gages on back surface

Fig. 2b—Instrumentation for slip

column axial load was applied and maintained constant throughout the test. The anchored bars were loaded in increments of roughly 2000 psi (140 kgf/cm^2). The load increments were applied continuously at 2 min intervals. The test was terminated when one of the anchored bars pulled out of the column. In general, failure was fairly sudden and resulted in the entire side face of the column spalling.

Instrumentation—To determine slip of the bar relative to the concrete the procedure developed by Minor[1] in the basic hook tests was used in the joint tests and is discussed in detail in Reference 2 and 3. In all tests slip was measured at five points along the length of the anchored bar (see Fig. 1). The back face served as the reference plane for slip measurement. Fig. 2b shows the slip measuring wires and neoprene tubes in place

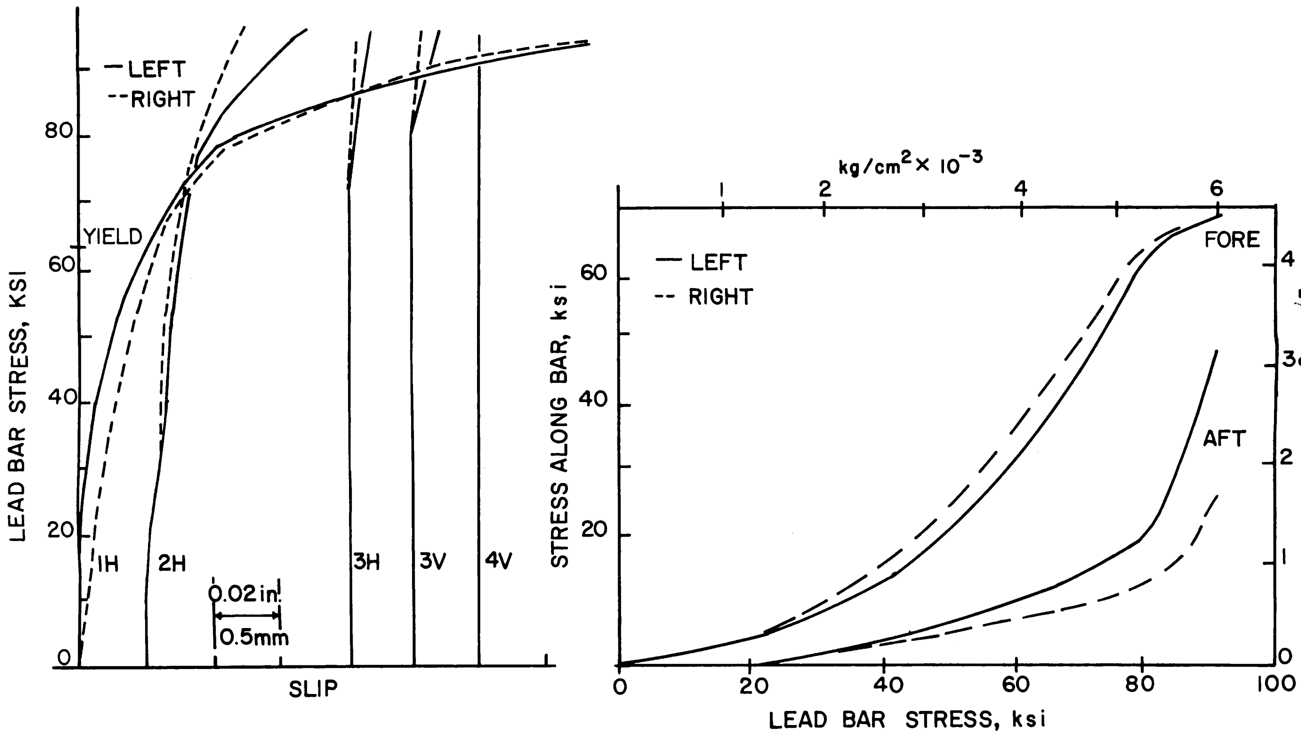


Fig. 3—Measured stress and slip relationships J7-90-15-1-H

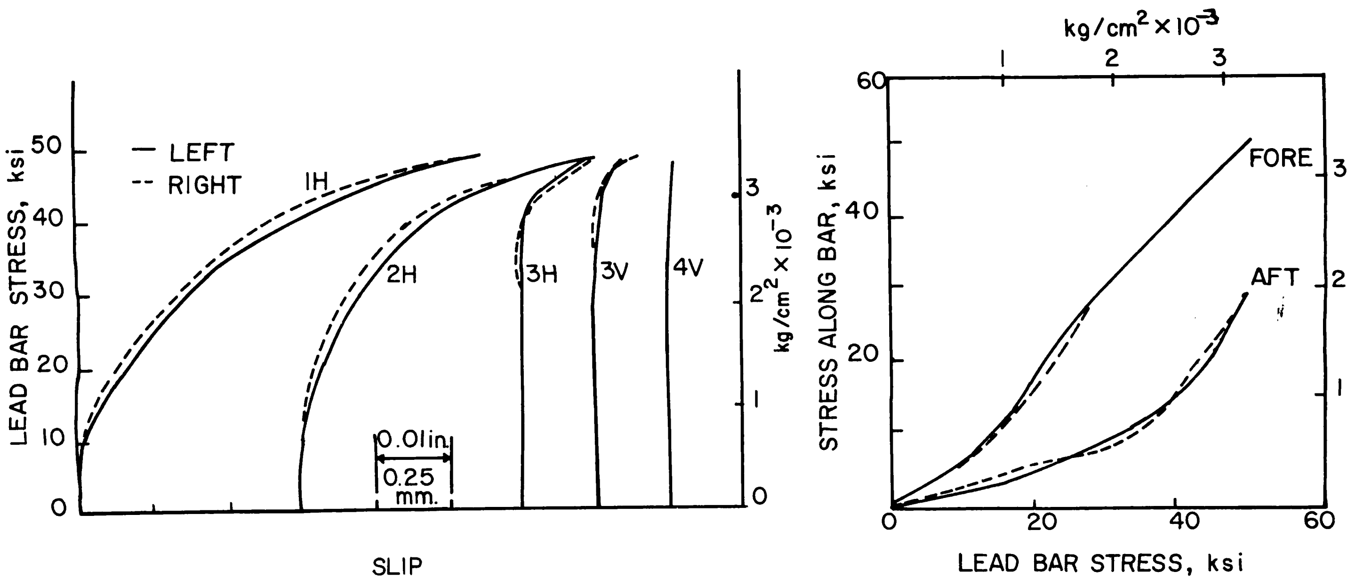


Fig. 4—Measured stress and slip relationships J11-90-15-1-H

on a specimen in the form prior to casting. The installation of dial gages is also shown in Fig. 2b.

To measure the stresses in the anchored reinforcing bars, three strain gages were mounted on each bar. Gages were located roughly at the point of horizontal and vertical tangency of the bent portion of the hook and on the bar protruding from the column (see Fig. 1).

EVALUATION OF TEST RESULTS

Typical stress-slip curves and lead stress versus bar stress are shown in Fig. 3 and 4. Curves are plotted for both bars in the specimens. Measured results for all tests are given in Reference 3. The test results indicate the following general trends:

(a) Most of the slip occurs over the lead straight embedment and the curved portion of the hooked bar. Very little slip was measured on the tail extensions of the hooks. This is consistent with trends noted in the basic hook tests.[1,2] The lead slip (Point 1H) is greatest in all cases. When the lead straight embedment was small, the slip at Point 2H was nearly as large as the lead slip. The slip at Points 3H, 3V, and 4H or 4V was very small. In most tests no slip was measured at the end of the tail (Points 4H or 4V) until failure was imminent. In many tests there was no measurable slip at the end of the tail.

(b) Slip measurements show that 180 deg hooks pulled toward the front face of the specimen rather than around the bend.

(c) The initial stiffness (stress over slip) up to levels of about 30 ksi (2100 kgf/cm^2) lead stress of #11 bars was about one-third that of the #7 bars. At failure the lead slip of the #7 bars was about two to three times that of the #11 bars.

(d) Stress transfer along the straight lead bar embedment varied from about 20 to 40 *ksi* for the #7 bars and was negligible for the #11 bars. The stress transferred to the tail extension was generally small, less than 20 ksi until failure was imminent. Near failure stresses on the tail increased rapidly.

A summary of measured slip behavior is listed in Table 2. Applied lead stress at lead slips of 0.005, 0.016, and 0.05 in. (0.12, 0.41, 1.27 mm) is listed. Stress at a slip of 0.016 in. was selected because it is in the range suggested as a permissible crack width in beams.[4,5] If it is assumed that the crack width at the beam column joint is about equal to the slip of the anchored bar, the observed stress at 0.016 in. slip provides a measure

TABLE 2—SUMMARY OF TEST RESULTS

Specimen	Measured results: Stress, ksi, at lead slip of 0.005 in.	0.016 in.	0.05 in.	Stress at failure, f_u, ksi	Approx. slip at failure, in.	Computed results: $(f_h + f_l)$,* ksi	$\frac{f_u}{(f_h + f_l)}$	$(f_h + f_{sh})$,† ksi	$\frac{f_u}{(f_h + f_{sh})}$	Slip at 0.6 $(f_h + f_{sh})$, in.
J7-90-15-1-H	33	55	77	91	0.15	63	1.44	60	1.52	0.006
J7-90-15-1-M	35	61	78	100	0.18	67	1.50	60	1.67	0.005
J7-90-15-1-L	30	52	78	97	0.21	65	1.49	60	1.61	0.006
J7-90-12-1-H	20	31	59	63	0.08	53	1.19	51	1.24	0.015
J7-180-15-1-H	24	45	72	87	0.15	59	1.47	60	1.45	0.012
J7-180-12-1-H	23	38	59	61	0.07	53	1.16	56	1.09	0.012
J7-90-15-2-H	33	52	75	99	0.25	64	1.55	60	1.65	0.007
J7-70-15-2-M	32	46	70	95	0.21	64	1.49	60	1.58	0.008
J7-90-15-3-H	37	65	80	104	0.21	64	1.68	60	1.73	0.005
J7-90-15-3*a*-H	37	63	80	98	0.22	57	1.72	60	1.65	0.005
J7-90-15-4-H	29	54	73	74	0.05	63	1.17	50	1.48	0.006
J11-90-15-1-H	19	33	48	49	0.06	36	1.36	41	1.20	0.009
J11-70-15-1-L	19	29	48	52	0.06	36	1.45	40	1.30	0.012
J11-90-12-1-H	18	30	—	42	0.04	32	1.32	28	1.50	0.005
J11-180-15-1-H	13	25	43	44	0.05	34	1.30	39	1.13	0.015
J11-180-15-1-H	26	37	—	50	0.05	34	1.47	38	1.31	0.009
J11-90-15-2-H	22	35	48	48	0.06	37	1.29	40	1.20	0.007
J11-90-15-2-L	18	28	50	53	0.06	36	1.52	38	1.39	0.011
J11-90-15-3-L	16	28	53	62	0.09	36	1.72	40	1.55	0.014
J11-90-15-3*a*-L	24	42	66	69	0.06	37	1.86	52	1.33	0.010
J11-90-15-4-L	26	39	—	44	0.03	33	1.34	28	1.57	0.003
J11-90-15-5-L	23	39	63	66	0.08	37	1.78	52	1.27	0.011

*ACI 318-71, "other" bar values.
†Using proposed recommendations.

Note: 1 ksi = 70.3 kgf/cm^2
0.01 in. = 0.25 mm

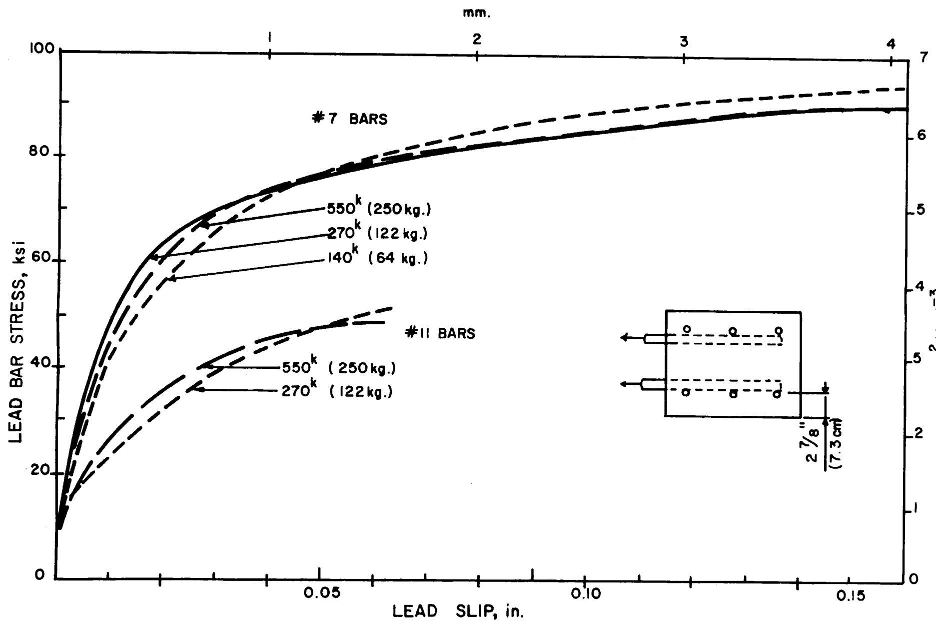


Fig. 5—Influence of column axial load on slip

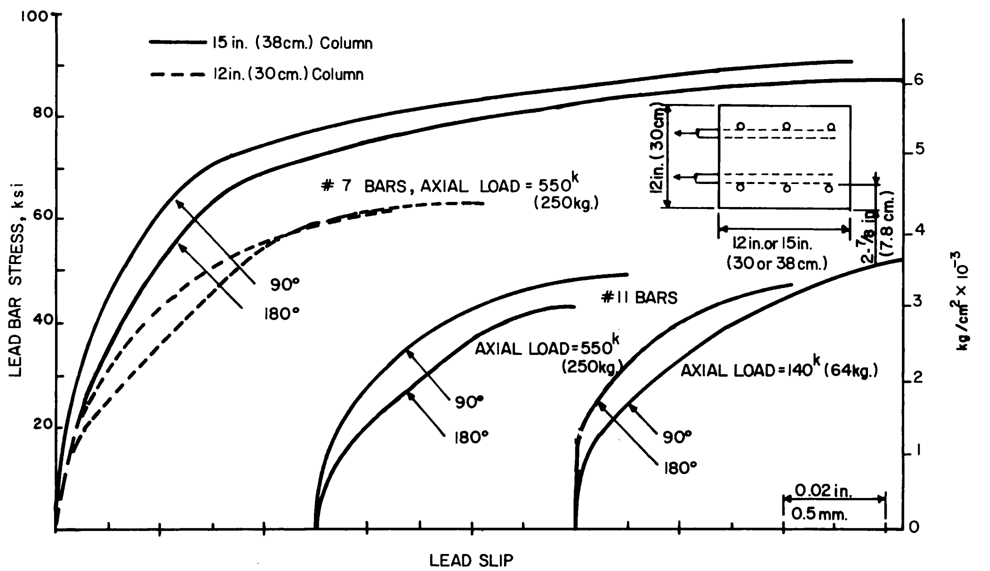


Fig. 6—Influence of bend angle on slip

of the serviceability of the hooked bar. In addition, slip and stress at failure are tabulated.

To evaluate the influence of the individual parameters considered in this study, the lead bar stress and lead slip for comparable tests will be discusssed in detail in the following section. In each case average values for the two bars of a specimen are compared.

Influence of column axial load

Fig. 5 shows stress-slip curves for five specimens with the same lateral confinement, column bars outside the beam bars and concrete cover constant. Only the level of column load was varied. Based on the stress and slip measurements for the tests in which axial loads were varied, the influence of column axial loads appears to be negligible. It should be noted that in all cases the tail of the hook was oriented in the direction of the column axial load. Other orientations of bent bars and different lateral confinement might produce different results.

Influence of bend angle

Stress-slip curves are shown in Fig. 6 for tests in which axial load and lateral confinement remain constant, but the bend geometry varied. Curves are shown for 90 deg and 180 deg hooks. In the study of basic hook strength reported previously,[1] 90 deg hooks tended to be stiffer (slip less at a given stress) than 180 deg hooks. For the hooks anchored in beam-column joints, there

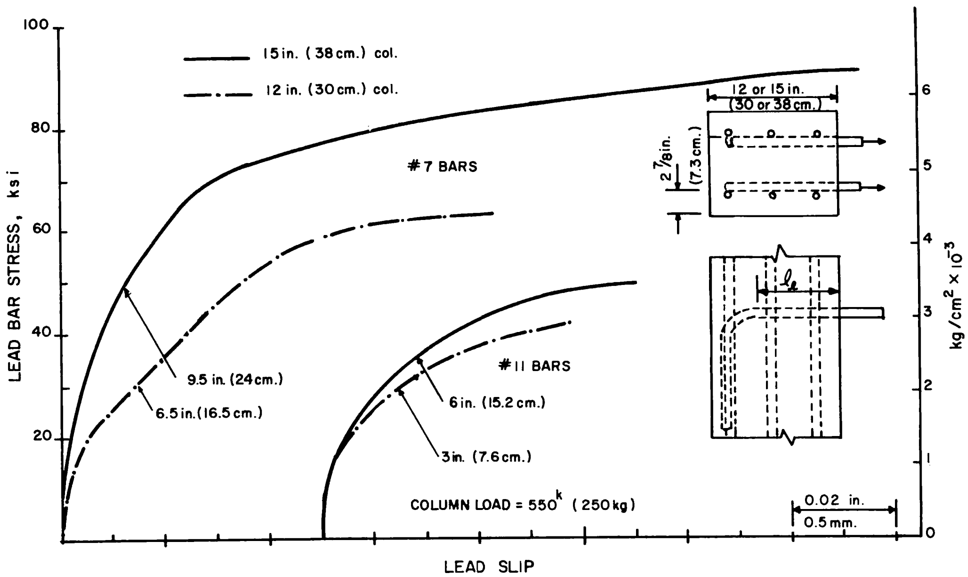


Fig. 7—Influence of lead embedment on slip

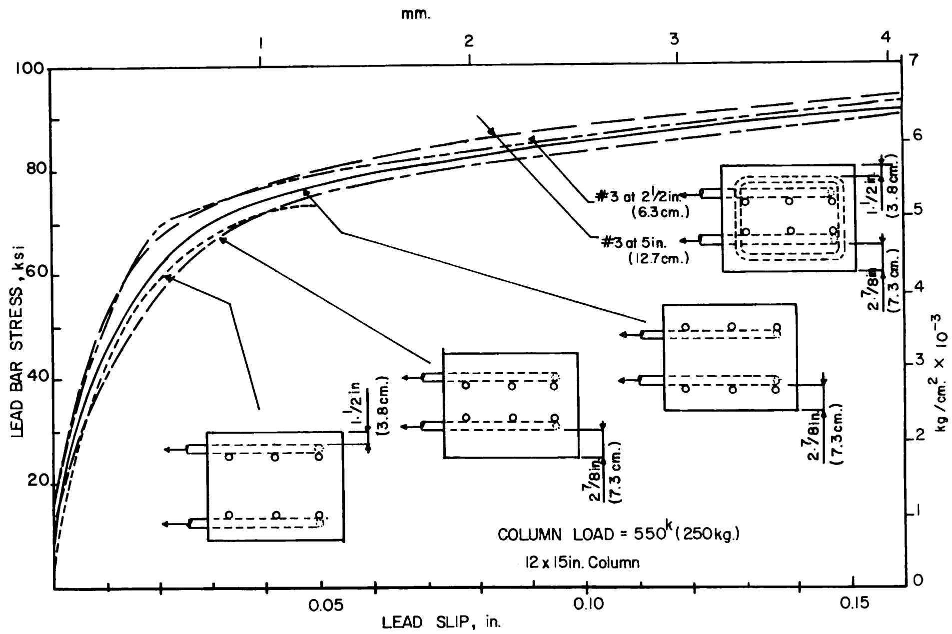


Fig. 8—Influence of confinement on slip, #7 bars, 90 deg hook

does not appear to be any significant difference in the behavior of 90 and 180 deg hooks.

Influence of lead embedment

The influence of the lead embedment, that is, the straight bar embedment between the hook and the column face, is shown in Fig. 7. The slip is greater at all stress levels for those specimens with shorter lead embedment. The lead embedment in these tests provided only limited length for stress transfer to the concrete before the hook, especially for large bars. However, with larger lead embedment, the lateral restraint against splitting is improved because a larger area of concrete must spall or split before the bar will fail.

Influence of lateral confinement

Fig. 8 shows stress-slip curves for five specimens with #7 bars but with different lateral restraint against side splitting. The curves lie in a very narrow band and indicate that there was little difference in slip behavior for four of the specimens. Reduction of concrete cover from 2⅞ in. to 1½ in. did not change the shape of the curve but did reduce drastically the stress and slip at failure.

Fig. 9 shows the effect of lateral confinement on the response of six specimens with #11 bars. It is apparent that the location of the column bars had very little influence. However, with ties through the joint the stress at failure reached

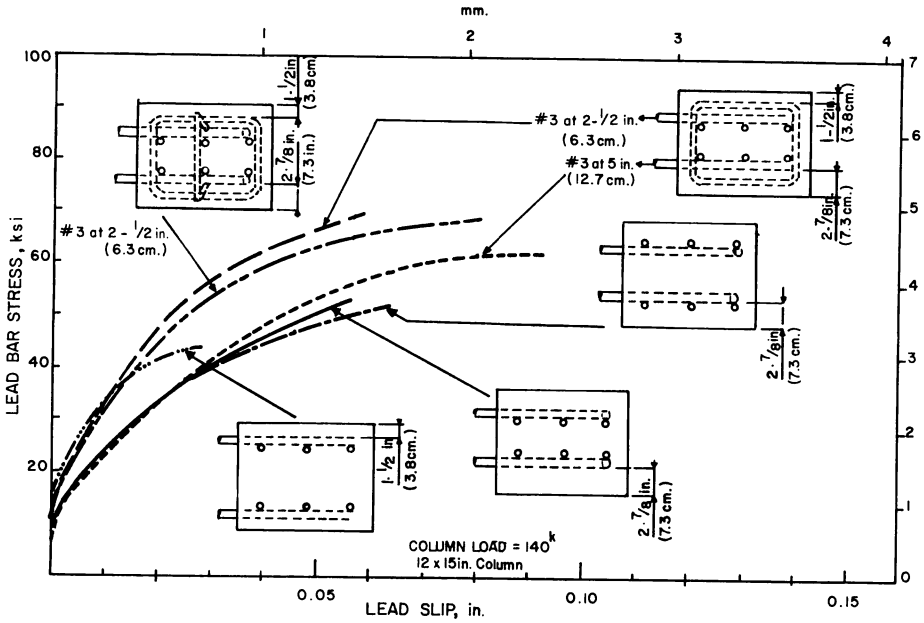


Fig. 9—Influence of confinement on slip, #11 bars, 90 deg hook

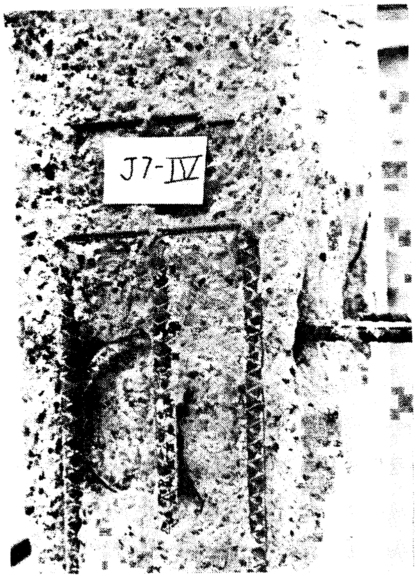
yield in all cases. The test results indicate that closely spaced ties are especially beneficial in the case of large anchored bars. A reduction in cover reduced both the strength and deformation capacity. A combination of ties through the joint and column bars outside the anchored bars might have improved the response further, however, no tests were run using this combination of variables.

Mode of failure

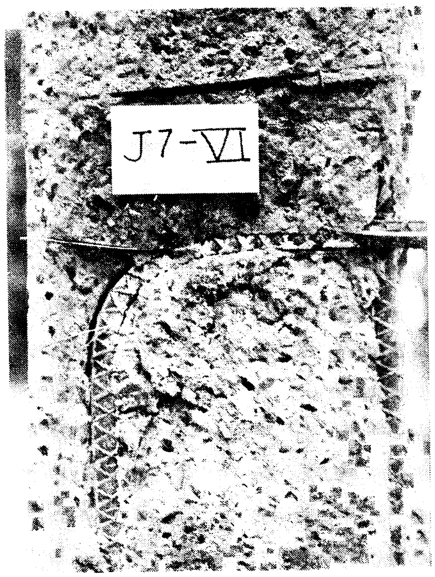
In nearly all the tests the sequence of cracking and subsequent failure followed a similar pattern. First cracking occurred on the front face of the column with cracks radiating outward from the bars. In most tests a vertical crack appeared on the side faces of the specimen at about the column longitudinal bars located nearest the front face. As stress increased cracks appeared on the side faces in the vicinity of the bent portion of the anchored bar.

Failure was always sudden and complete, with the entire side cover spalling away to the level of the anchored bars and the load dropping immediately to a fraction of its previous level. Fig. 10 shows the types of failure observed. There was very little difference between failure patterns in specimens with column bars located inside or outside the anchored bars. In those cases where the column bars were outside the beam bars, the column bars and ties through the joint were bent outward after failure which indicates that the compressive forces developed under the hook were very large and forced the concrete outward. It is also possible that the hook pulling forward formed a wedge which forced the concrete cover off and deformed the steel bars confining the joint core. In most tests a very large area of the side cover was destroyed, as can be seen in Fig. 10.

Once the sequence of events leading to failure is understood, the influence of various parameters on the strength and slip characteristics of the hooked bars can be more clearly established. While column axial loads would appear beneficial, in reality axial forces produce lateral strains which cause splitting on the same plane as that produced by the hooked bars. Therefore, the axial load does not offer any restraint to splitting of the side cover and may actually reduce strength by causing lateral strains in the same direction. Column bars unrestrained by ties through the joint likewise offer little lateral support. Such bars are too flexible and are not located near enough to the region of most intense distress, that is, the inside of the hook. As the column thickness was increased or ties carried through the joint, some improvement in stress and slip characteristics was noted. The improved performance of hooks in 12 x 15 in. columns can be attributed primarily to the larger side cover which must spall before the bar fails. Ties through the joint appear to be most beneficial if the spacing is about equal to or less than the radius of the hook so that confinement is provided in the region of most intense lateral pressure. The thickness of concrete cover does not seem to be too important as long as a local failure at the inside of the bend does not occur. In general it would appear that if anchored bars are placed inside beam bars and a 1½ in. (3.8 cm) clear cover is maintained on the reinforcement, side cover on the anchored bars will be sufficient to prevent a very localized failure of the side cover.

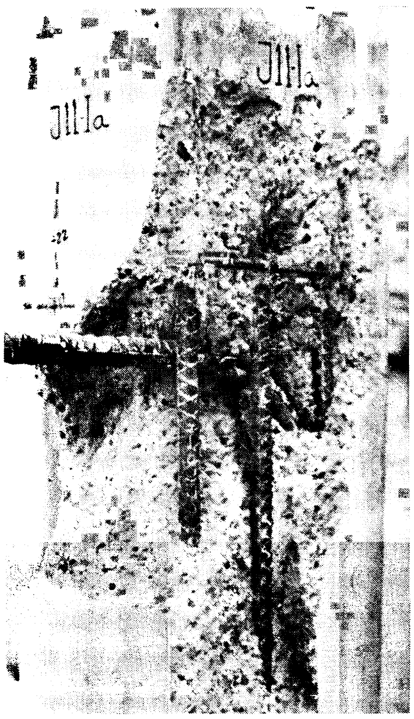


J7-180-15-1-H

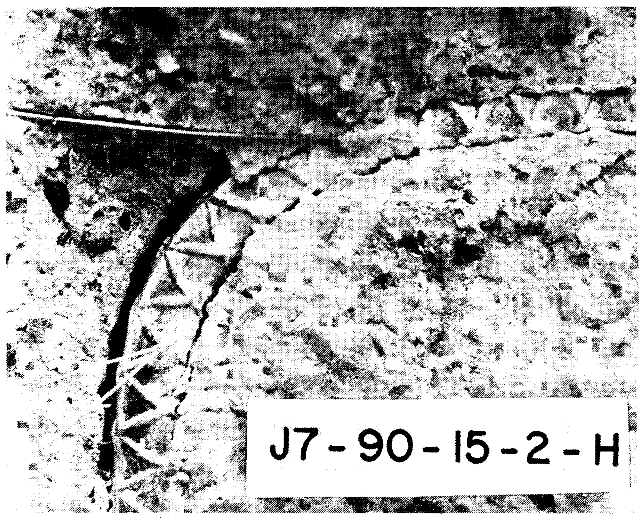


J7-90-15-2-H

J7-90-15-3a-H



J11-90-15-1-H



Slip and crushing at bend

Fig. 10—Appearance of specimens after failure

IMPLICATIONS OF TEST RESULTS ON DESIGN OF HOOKED BARS

The design provisions for hooked bar anchorages in ACI 318-71[4] are based primarily on provisions appearing in previous codes, the origin of which is not well-documented. Recent pullout tests of hooked bars embedded in massive concrete slabs[7] provide some additional information, but very limited data are available regarding hooked bar strength in typical structural applications. Therefore, the tests conducted in this study offer an opportunity to evaluate present design recommendations and to suggest changes.

Measured and computed strength (ACI 318-71)

Using the provisions of ACI 318-71, Section 12.8,[4] the strength of the bars in the test specimens was evaluated. The stress f_h developed by the standard hook was computed using values listed in Table 12.8.1 of the Code. In these computations, values for "other" bars were used. The stress developed over the straight lead embedment f_l was computed using the basic equation for development length (Section 12.5) and solving for f_l in terms of the straight lead embedment between the point of tangency of the hook and the face of the column. No modification was made in f_l for top cast or other bars. Values for the sum of f_h and f_l are listed in Table 2. Measured lead stresses at failure are listed and compared *with computed* values.

A ratio of measured to computed stress at failure varies from about 1.2 to 1.9. It should be noted that the equation for development length was not intended for bar lengths shorter than those required to develop yield stress, and it may be unrealistic to compute the anchorage strength on that basis. As indicated by the test results, very short straight embedment is not efficient in transferring stress from the bar to the concrete. Therefore, the importance of short straight lead embedment should probably be minimized in making design calculations.

Section 12.3 of ACI 318-71 requires that the embedment length of tension reinforcement from a critical section satisfy the requirements for the development length. Implications are that the total length of embedment from the critical section, including the hook and any tail extension, may be considered as development length. (Fig. 12.4 of the Commentary to ACI 318-71 and Section A.5.5 dealing with seismic design of ductile frames indicate that tail extensions may be considered as development lengths.) From the test results of this study, tail extensions beyond those of a standard hook cannot be considered as adequate for providing development length. In all cases, failure was produced by side splitting of the joint and not by pullout of the hooked bar. Therefore, tail extensions would not be effective in increasing anchorage strength where side splitting may occur. Although no specimens with lateral beams preventing side splitting were tested, the lateral restraint provided by beams might improve the effectiveness of tail extensions. Tail extensions beyond those required in a standard hook may also provide a margin against bond deterioration under cyclic loading, such as would be expected in earthquakes.

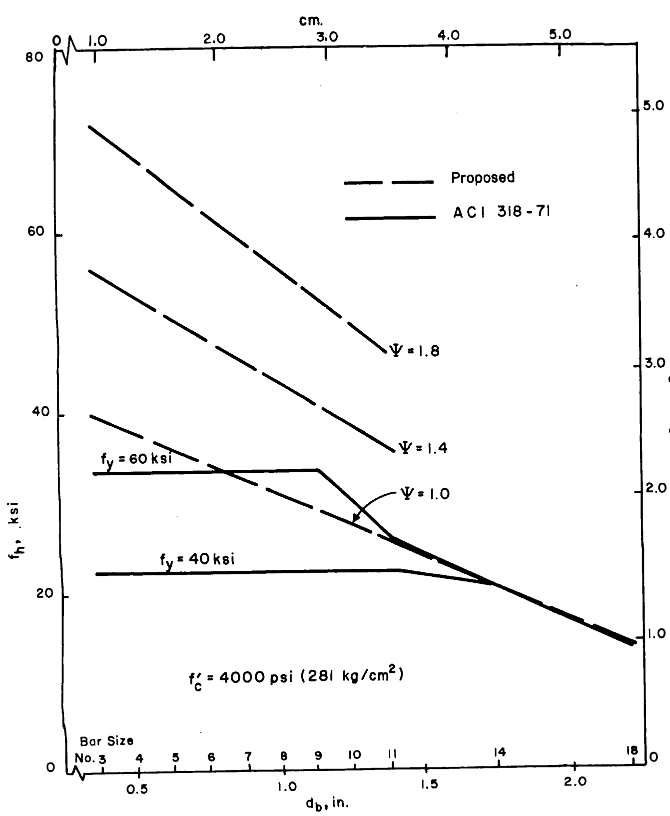


Fig. 11—Proposed and ACI 318-71 standard hook strength

PROPOSED DESIGN RECOMMENDATIONS

Standard hooks anchored in a joint shall be considered to develop tensile stresses in bar reinforcement of

$$f_h = 700\,(1 - 0.3d_b)\,\psi\sqrt{f_c'} \qquad (1)$$

but not greater than f_y.

The coefficient ψ shall be taken as unity unless the following conditions are satisfied.

The value of ψ may be taken as 1.4 if (a) the bar is #11 or smaller, (b) the lead straight embedment between the standard hook and the critical section is not less than four bar diameters or 4 in. whichever is greater, (c) side concrete cover normal to the plane of the hooked bar is not less than 2.5 in., and (d) cover on the tail extension is not less than 2 in.

The value of ψ may be taken as 1.8 if the joint is confined by closed ties at a spacing of $3d_b$ or less and meets the requirements for $\psi = 1.4$.

No increase in f_h shall be permitted for tail extensions or bend radii greater than required for a standard hook.

For better control of deflections and cracking, 90 deg hooks are preferable.

If additional development length is required the straight lead embedment length l_l between the critical section and the hook shall be computed by

$$l_l = [0.04A_b\,(f_y - f_h)\,/\sqrt{f_c'}] + l' \qquad (2)$$

where l' is $4d_b$ or 4 in., whichever is greater.

Comparison of proposed recommendations with ACI 318-71 and with test results

Fig. 11 shows the proposed and ACI 318-71 values for f_h for bar sizes from #3 to #18 and f_c'

$= 4000$ psi (280 kgf/cm^2). For hooks which have minimum lateral restraint, the values of f_h are quite close to those given in ACI 318-71. For bars which have greater cover provided, the values of f_h are increased roughly 40 percent. If in addition to sufficient cover, closely spaced ties are included through the joint, f_h is increased about 80 percent. No distinction is made for anchorage capacities of bars with different yield stresses.

Computed strengths using these recommendations for the bars tested in this study are listed in Table 2. Ratios of measured-to-computed strength vary from a minimum of about 1.1 to 1.7. Table 2 also lists the measured lead slip for the test specimens at 0.6 of the computed strength to give an indication of the possible crack width at the beam-column joint due to slip of the anchorage at service loads. For the #7 bars, measured slip values at "service loads" for most specimens were in the range of 0.005 to 0.008 in. (0.13-0.20 mm). In the case of the #11 bars, the largest slip at "service loads" was 0.015 in. (0.38 mm).

SUMMARY AND CONCLUSIONS

The design recommendations proposed for computing the anchorage capacities of hooked bars reflect the trends observed in the experimental program and are summarized below.

(a) Strength is increased as restraint against side splitting is increased. Standard hooks embedded in mass concrete exhibit strengths well in excess of yield.[6]

(b) Much higher strengths can be permitted than in current specifications. For example, there is no reason a standard hooked bar with 60 ksi (4200 kgf/cm^2) yield strength should be permitted to develop 30 ksi (2100 kgf/cm^2), while an identical hook of 40 ksi (2800 kgf/cm^2) yield strength should be permitted to develop only 20 ksi (1400 kgf/cm^2). For small bar sizes these values are unrealistically prohibitive. However, to allow higher stresses, minimum straight embedments before the hook are required, especially for larger bar sizes.

(c) No distinction is made between the strength of 90 deg and 180 deg standard hooks.

The results indicate that while the strength of the hook can be reasonably estimated, the reliability of computed straight lead embedment lengths for stress values less than f_y, say 10 or 20 ksi (700 or 1400 kgf/cm^2), cannot be verified from the available test results, especially for large bar sizes, and further study is needed. Additional work is also needed to evaluate the confinement provided to a hooked bar by lateral beams framing into the joint. However, it would appear that design recommendations can be adjusted to realistically reflect the actual strength of hooks and to satisfy required serviceability criteria.

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NOTATION

A_b = area of bar
d_b = diameter of bar
f_c' = concrete strength, 6 x 12 in. cylinders
f_h = stress developed by hook
f_l = stress developed over straight lead embedment
f_{sh} = stress developed over effective straight lead embedment
f_u = measured stress at failure of anchored bar
f_y = yield stress of bar
l_l = straight lead embedment
N = axial load on column
ψ = constant reflecting lateral confinement

REFERENCES

1. Minor, John, and Jirsa, James O., "Behavior of Bent-Bar Anchorages," ACI Journal, *Proceedings* V. 72, No. 4, Apr. 1975, pp. 141-149.

2. Minor, John, "A Study of Bent-Bar Anchorages in Concrete," PhD Thesis, Rice University, Houston, Jan. 1971.

3. Marques, Jose L. G., "A Study of Anchorage Capacities of Confined Bent-Bar Reinforcement," PhD Thesis, Rice University, Houston, May 1973.

4. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-71)," American Concrete Institute, Detroit, 1971, 78 pp.

5. ACI Committee 318, "Commentary on Building Code Requirements for Reinforced Concrete (ACI 318-71)," American Concrete Institute, Detroit, 1971, 96 pp.

6. Untrauer, Raymond E., and Henry, Robert L., "Influence of Normal Pressure on Bond Strength," ACI Journal, *Proceedings* V. 62, No. 5, May 1965, pp. 577-586.

7. Hribar, John A., and Vasko, Raymond C., "End Anchorage of High Strength Steel Reinforcing Bars," ACI Journal, *Proceedings* V. 66, No. 11, Nov. 1969, pp. 875-883.

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