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High Strength Bars as Concrete Reinforcement. Part 5.

Lapped Splices in Concentrically Loaded Columns

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SYNOPSIS

Tests were made of 16 concentrically loaded columns reinforced by high strength bars with and without splices. Circular columns were spirally reinforced and rectangular columns were tied. Both butt and lapped splices were incorporated, with lap lengths varying from zero to 30 diameters. The data showed that force in a lapped compression splice is transferred by a combination of bond and end bearing, and the contribution of each was evaluated. Test results were in reasonable agreement with the lap length requirements of the 1963 ACI Building Code, and also confirmed the acceptability of a butt splice confined by a sleeve.

HIGH STRENGTH REINFORCEMENT TEST PROGRAM

An experimental program at the PCA Laboratories concerned with the use of high strength bars as concrete reinforcement is being reported in a series of papers collectively entitled, "High Strength Bars as Concrete Reinforcement." The first part, "Introduction to a Series of Experimental Reports," PCA Development Department Bulletin D52(1)*, outlines the program and includes discussion of the nature of high strength steels as well as of some economic aspects and design considerations for concrete structures reinforced with these steels. In Part 2, "Control of Flexural Cracking," Bulletin D53(2), an exploratory laboratory study is reported, covering the design factors which can be used to control flexural cracking under high steel stress. Part 3, "Tests of a Full-Scale Roof Girder," Bulletin D54(3), deals with a sustained load test followed by a test to destruction of a full-scale 60-ft roof girder of unusual design reinforced with high

*Numbers in parentheses refer to references at end of paper.

strength steel. Part 4, "Control of Cracking," Bulletin D59[4], presents further investigations of cracking and deals particularly with girder and slab specimens having cross-sections of the type used in bridge construction when subjected to both dynamic and static loads.

This Part 5 reports an exploratory investigation of the performance of concentrically loaded columns which incorporate lapped splices in the high strength longitudinal reinforcement. The performance of the splices is compared with the requirements of Section 805(c)1 of the 1963 ACI Building Code[5].

BACKGROUND AND SCOPE

Development of Column Design

Current design of reinforced concrete columns for concentric loading utilizes the basic addition law developed 30 years ago from the results of the ACI column investigation[6]. Tests carried out since that investigation have repeatedly confirmed that for concentric loading the ultimate strength, P_u, of a tied column or the yield point strength of a spirally reinforced column is given by:

$$P_u = 0.85\, f'_c\, A_c + f_y\, A_s \quad (1)$$

where A_c = net concrete cross-section
A_s = cross-section of longitudinal reinforcement
f'_c = 6 x 12-in. cylinder strength of concrete
f_y = yield point of longitudinal reinforcement.

In the original ACI column investigation and since that time, the effect of many variables on column strength has been studied experimentally. However, almost without exception these studies have been made on columns in which the longitudinal reinforcement was continuous from one end of the column to the other; the effect on column strength of splices in the longitudinal reinforcement has received little attention. Four columns in Series 1 of the ACI column investigation were tested to determine the effect of various end conditions on column performance. These four columns had lap splices at each end between dowel rods and the main reinforcement, with splice lengths of 20 and 30 diameters. In all cases the 47.9 ksi yield point of the longitudinal reinforcement was developed at ultimate strength of the columns.

In practice, the longitudinal reinforcement of multistory columns usually consists of a number of lengths of spliced reinforcing bars. Splicing may be by lapping, welding, or mechanical couplers. These three types of splices are suitable for the transfer of both compression and tension. Where it is certain that under all conditions of loading a reinforcing bar will remain in compression, a fourth type of splice, the butt joint may be used. In this joint the square cut ends of the reinforcing bars are placed in contact and held in alignment with a light metal sleeve.

The most commonly used splice is probably the lapped splice. The effectiveness of a 20-diameter splice in developing the yield point of intermediate grade reinforcement was demonstrated in the tests of the ACI column investigation referred to above, and such splices have performed satisfactorily in structures the world over. However, with the growing use of higher strength reinforcement it was considered desirable to examine the behavior of lapped compression splices in more detail, and in particular to check the extent to which yield point stresses of 60 and 75 ksi can be developed in column reinforcement containing lapped splices.

Behavior of Lapped Compression Splices

The lapped splice transfers force from one bar to another through the concrete which surrounds both bars. At any point in the length of the splice, force is transferred by bond from one bar to the surrounding concrete. Simultaneously, and also by bond, force is being transferred from the concrete to the other bar of the pair forming the splice. Within the concrete the force is apparently transferred by shear. The integrity of a lapped splice therefore depends upon the development of adequate bond between the surface of the reinforcing bars and the surrounding concrete. Should the bond between the bars and the concrete break down, the splice will fail.

Force will also be transferred from reinforcing bar to concrete by direct end bearing at the square-cut end of the bar. This end bearing effect has generally been neglected in formulating rules governing the length of lapped splices, probably because experimental data have been lacking.

These rules usually imply that the force that can be transferred by a lapped splice is directly proportional to the lap length. However, the results of the tests described in this paper have indicated that a considerable proportion of the total force transferred by a lapped compression splice is transferred by end bearing between the bars and the concrete.

Considering the way in which force is transferred by a lapped splice, it is possible that the performance of such a splice may be affected by the following variables; length of splice, concrete strength, bar diameter, reinforcement percentage, amount of cover and lateral ties, and the shape of the reinforcement stress-strain curve.

The load capacity of a lapped splice will probably increase with increase in length of the lap, with increase in concrete strength and with increase in lateral restraint of the concrete round the splice caused by increase in cover or the amount of lateral ties. Ferguson[7] has shown that in a tension lap splice the average bond stress at failure reduced as the bar diameter increased, a finding which may or may not be applicable to compression splices. Since lapped splices appear to fail at longitudinal strains of 0.0025 in./in. or less in the surrounding concrete, a stress-strain curve with a clearly defined yield point should result in a higher steel stress being developed at the failure strain than would be the case with a rounded stress-strain curve. The effects of duration of test and of eccentricity of load are uncertain but are probably not significant.

Scope

This investigation was restricted to an exploration of lapped splices in concentrically loaded columns having either a circular section with spiral reinforcement, or a rectangular section with ties. In both types of column, the amount of lateral reinforcement provided was close to the minimum amount allowed by the ACI Building Code (ACI 318-63). The principal variable investigated was the effect of splice length on the stress which could be developed in the reinforcement away from the splice. Tests were also conducted

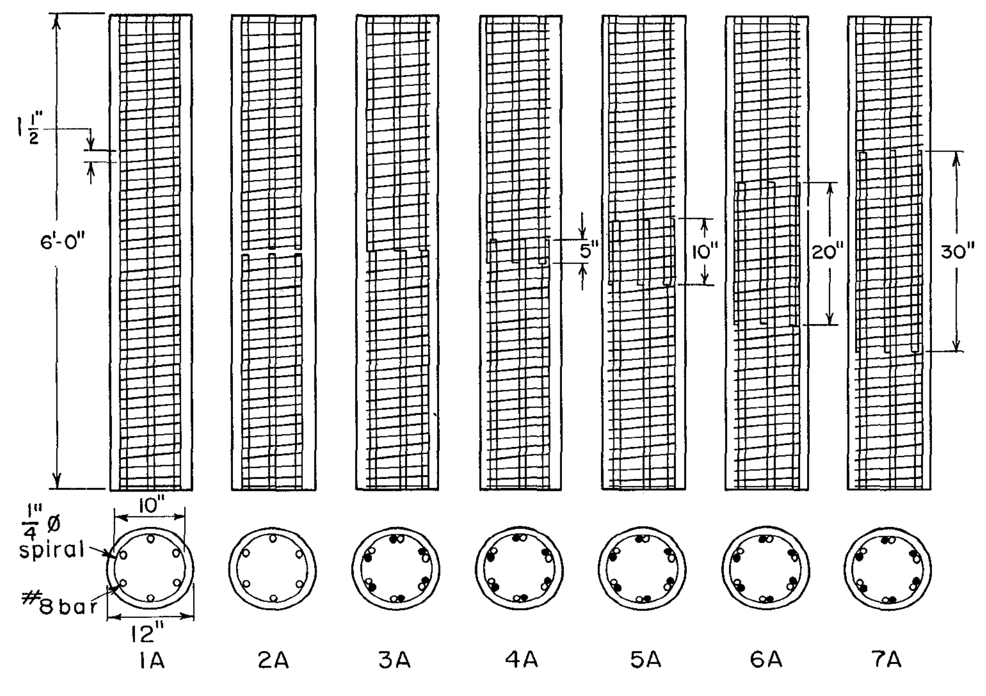

Fig. 1 — Details of Series A Columns.

on a column of each type in which butt joints were made in the longitudinal reinforcement. The column cross-sections were the largest which could be tested in a 1,000,000-lb testing machine.

TEST SPECIMENS

Series A — Spirally Reinforced Circular Section Columns

The elevations and cross-sections of the seven columns tested in this series are shown in Fig. 1. All columns of this series were of 12-in. overall diameter and were 72 in. long. Other details of these columns are as follows:

Column 1A — Longitudinal reinforcement was six No. 8 deformed high strength steel bars continuous from end to end of the column. Spiral reinforcement was ¼-in. diameter rod bent to give a 10-in. spiral with a 1½-in. pitch. The ratio of the spiral reinforcement provided is 0.0133, as compared with the minimum ratio of 0.0116 permitted by Section 913(b) of the 1963 ACI Building Code, for $f'_c = 3500$ psi and $f_y = 60{,}000$ psi.

Column 2A — As Column 1A, but with the longitudinal reinforcement cut at mid-height of the column. The two halves of each reinforcing bar were placed so that the cut faces abutted one another. No further attempt was made to join the reinforcing bars together. The reinforcing bars were cut with a power hack saw and edge burrs were removed. No other treatment was given the abutting end faces.

Column 3A — As Column 1A, but with the longitudinal reinforcing bars cut at mid-height of the column. The two halves of each reinforcing bar were aligned so that their ends were just offset from one another, in effect forming a lapped splice of zero length.

Columns 4A, 5A, 6A, and 7A — As Column 1A, but with each longitudinal reinforcing bar made up of two pieces joined together by a lapped splice. The lengths of the lap were 5, 10, 20, and 30 in. respectively.

Series B — Rectangular Section Tied Columns

The elevations and cross-sections of the nine columns tested in this series are shown

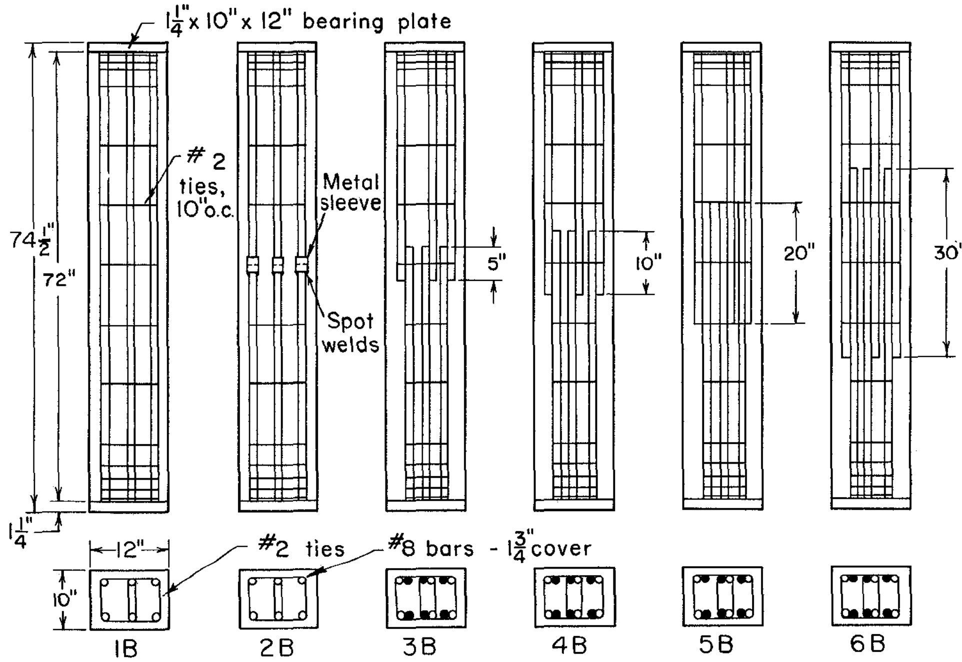

Fig. 2 — Details of Series B Columns.

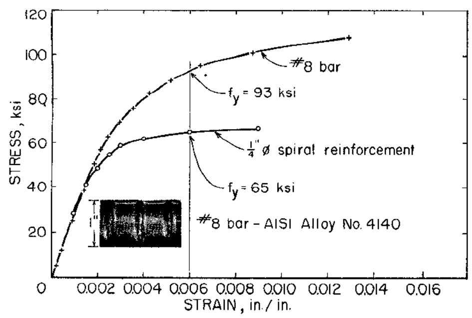

Fig. 3 — Stress-Strain Curves for Column Reinforcement — Series A.

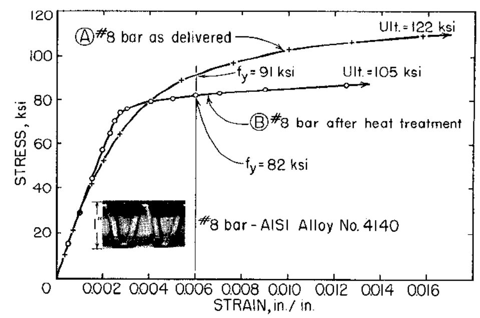

Fig. 4 — Stress-Strain Curves for Column Reinforcement — Series B.

in Fig. 2. All the columns in this series were 10 by 12 in. in cross-section and were 72 in. long. Other details of these columns are as follows:

Column 1B — Longitudinal reinforcement was six No. 8 deformed high strength steel bars continuous from end to end of the column. The lateral reinforcement consisted of ties made from No. 2 deformed bars, spaced 10 in. apart. These lateral ties constitute the minimum amount permitted by Section 806(b) of the 1963 ACI Building Code.

Column 2B — As Column 1B, but with the longitudinal reinforcement cut at midheight of the column. The two halves of each reinforcing bar were placed so that the cut faces abutted one another. A tubular steel sleeve $\frac{1}{16}$-in. thick and 2 in. long was used to hold the cut ends of each bar in alignment. The sleeves were spot welded to the lower half of each cut bar, as indicated in Fig. 2. The reinforcing bars were cut with a power hack saw and edge burrs were removed. No other treatment was given the abutting end faces.

Columns 3B, 4B, 5B, and 6B — As Column 1B, but with each longitudinal reinforcing bar made up of two pieces joined together by a lapped splice. The lengths of the lap were 5, 10, 20, and 30 in., respectively.

Columns 1B1, 5B1, 6B1 — As Columns 1B, 5B, and 6B respectively, but reinforced longitudinally with high strength steel that had been heat treated to give a sharp yield point.

Materials

The longitudinal reinforcement in the Series A columns was of No. 8 deformed bars having a yield strength of 93,000 psi at a strain of 0.006 in./in., and conforming to ASTM Designation: A431-59T. The stress-strain curve for this bar is as shown in Fig. 3.

The longitudinal reinforcement in the series B columns, 1B through 6B, was of No. 8 deformed bars having a yield strength of 91,000 psi at a strain of 0.006 in./in., and conforming to ASTM Designation: A431-59T. The stress-strain curve for this reinforcement is shown by curve A in Fig. 4. In Columns 1B1, 5B1, and 6B1 the same reinforcement was used as in the other Series B columns but after it had been heat treated. The heat treatment gave the steel a more clearly defined yield point, and a more linear stress-strain curve up to yield, than had the untreated steel, as shown by curve B in Fig. 4. The heat treatment consisted of heating the bars to 1200 F for 8 hours, followed by slow cooling over a further period of 8 hours. This treatment reduced the yield stress at 0.006 strain from 91,000 psi to 82,000 psi, but increased the limit of proportionality from about 35,000 psi to about 70,000 psi.

The spiral reinforcement in the Series A columns was formed from $\frac{1}{4}$-in. smooth rod with a yield strength of 65,000 psi at 0.006 strain. The stress-strain curve for this rod is shown in Fig. 3. The ties in the Series B columns were formed from No. 2 deformed bars having a clearly defined yield point of 58,500 psi.

All longitudinal reinforcement conformed to ASTM Designation: A305 for deformations. Although No. 2 bars are not covered by ASTM Designation: A305, the No. 2 bars used for ties in the Series B columns were deformed in a similar manner to that of larger bars conforming to this designation.

The concrete used in the columns contained 4.5* bags per cubic yard of a blend of Type I portland cement, and ¾-in. maximum size aggregate. Four to five per cent of air was entrained in the concrete using an air-entraining agent. The concrete was sealed in the forms and moist cured at 70 F for the first three days. Subsequently the columns were stored at 70 F and 50 per cent relative humidity. The concrete strengths at the time of test are set out in Table I. The strengths quoted in this table are the average of three 6 x 12-in. cylinders cast at the time of fabrication of the columns, and stored alongside the columns until the time of testing at an age of 10 to 14 days.

Fabrication

Series A — The circular section columns were cast in a vertical position. The reinforcement cage was fabricated by tying the longitudinal reinforcing bars to the preformed spiral. The cage was then placed in an impregnated cardboard form, and the spacers used to maintain the pitch of the spiral reinforcement were withdrawn. The longitudinal reinforcement rested on a plywood base, and was held in position by wire ties which were subsequently cast in the column.

To help maintain the correct concrete cover over the reinforcement during the concreting operation, three cardboard tubes were inserted between the reinforcement cage and the form at 120° intervals; these tubes were withdrawn as concreting progressed. An immersion vibrator was used to compact the concrete. The top of the column was trowelled flush with the ends of the longitudinal reinforcing bars.

Series B — The rectangular tied columns were cast horizontally. The ends of the longitudinal reinforcing bars were milled, then drilled and tapped with a shallow ¼-in. diameter hole. The milled ends butted to 1¼-in. thick end plates, and were held in place by screws. This was done to provide positive end bearing for the bars against the end plates in order to avoid premature end splitting of the columns due to uneven bearing. The No. 2 bar ties were then tied in place and the reinforcement cage was placed in the form. As with the Series A columns, the concrete was compacted with an immersion vibrator.

TABLE 1—CONCRETE STRENGTHS AT COLUMN TESTING

Column No.	Cylinder Strength, psi	Column No.	Cylinder Strength, psi
		1B	3830
1A	3305	1B1	3800
2A	3685	2B	3605
3A	3575	3B	3515
4A	3530	4B	3715
5A	3530	5B	4140
6A	3510	5B1	4190
7A	3510	6B	3950
		6B1	3640

*It should be noted that laboratory concretes are made, compacted, and cured under controlled conditions. Hence, for a given cement content, higher strengths are usually obtained than those that may reasonably be expected in the field.

Instrumentation

Series A — The spirally reinforced columns were instrumented to record the shortening of the middle 40 in. of each column, using linear differential transformers with suitable clamps and extension rods. The output from these transformer gages was monitored continuously by a Sanborn 67A strip chart recorder. One of these gages may be seen in Fig. 5. Strain gages were not used on the steel in Series A.

Series B — The rectangular tied columns were instrumented to record the shortening of the middle 40 in. of each column, using linear differential transformers as in Series A. In addition, SR-4 strain gages were mounted on the longitudinal reinforcement midway between the upper end of the splice and the upper end of the column, on one of the ties at the level of the splice, and on the face of the column at the level of the splice. The number of gages at each location was varied from test to test. Both the SR-4 gages and the differential transformer gages were monitored continuously by strip chart recorders.

TEST PROCEDURE

The columns were tested in a one-million pound capacity testing machine. To facilitate leveling, the columns were placed on a thin bed of Hydrocal high strength plaster on a steel plate resting on the bottom platten of the testing machine. In the Series A tests a second steel plate was seated horizontally on the top of the column, again using a thin layer of Hydrocal. In the Series B columns the upper steel plate was attached to the longitudinal reinforcement and cast integrally with the column as described earlier. The testing machine head was lowered so as to bring the upper platten into contact with the steel plate on the column top. Wedges were then inserted to prevent rotation of the upper machine head platten under load. The columns were therefore tested with their ends effectively "fixed." A typical test in progress is seen in Fig. 5.

The load applied to the columns was increased uniformly at a rate of 50 kips per minute until failure occurred. During the tests the strain gages and transformer gages were monitored continuously, and the development of cracks in the concrete was noted. The tests were continued until the columns had deformed to such an extent that it was certain that the maximum load capacity of the columns had been developed.

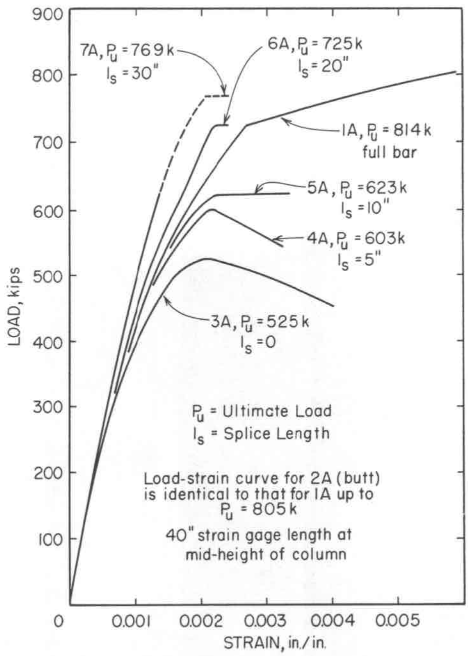

Fig. 6 — Load-Strain Curves for Series A Columns.

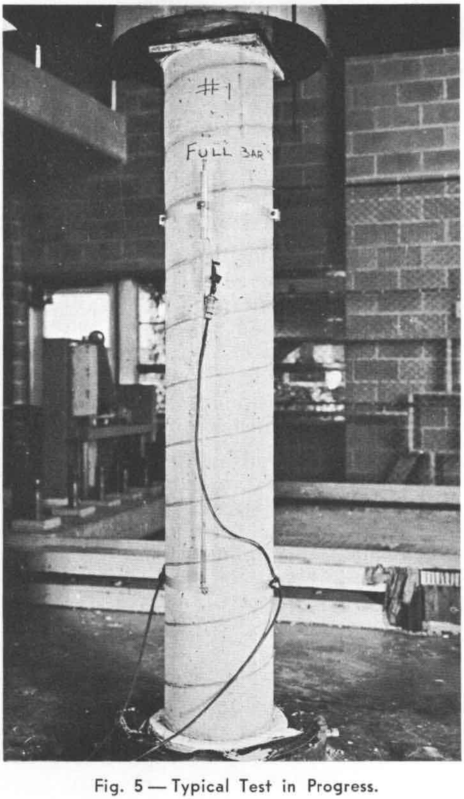

Fig. 5 — Typical Test in Progress.

TEST RESULTS

Series A — Spirally Reinforced Circular Section Columns

The behavior of Column 1A, with continuous bars, followed the classical pattern for an axially loaded, spirally reinforced column. Fine vertical cracks were observed on the surface of the column at a longitudinal strain of about 0.002 in./in. At a strain of 0.0026 the shell of the column commenced to spall off, and the strain in the column began to increase much more rapidly with increase in load, as may be seen in Fig. 6. Failure occurred near the top of the column after a large part of the shell had fallen away. The spiral reinforcement broke and this was followed by disintegration of the concrete core and by buckling of the longitudinal bars.

Column 2A, with a butt joint in the bars, behaved in a manner almost identical to that of Column 1A; the load-strain curve was practically the same, but failure occurred when the spiral reinforcement broke at mid-height of the column and the abutting ends of the longitudinal reinforcement slipped off one another.

The progressive change in behavior of Columns 3A through 7A due to the gradu-

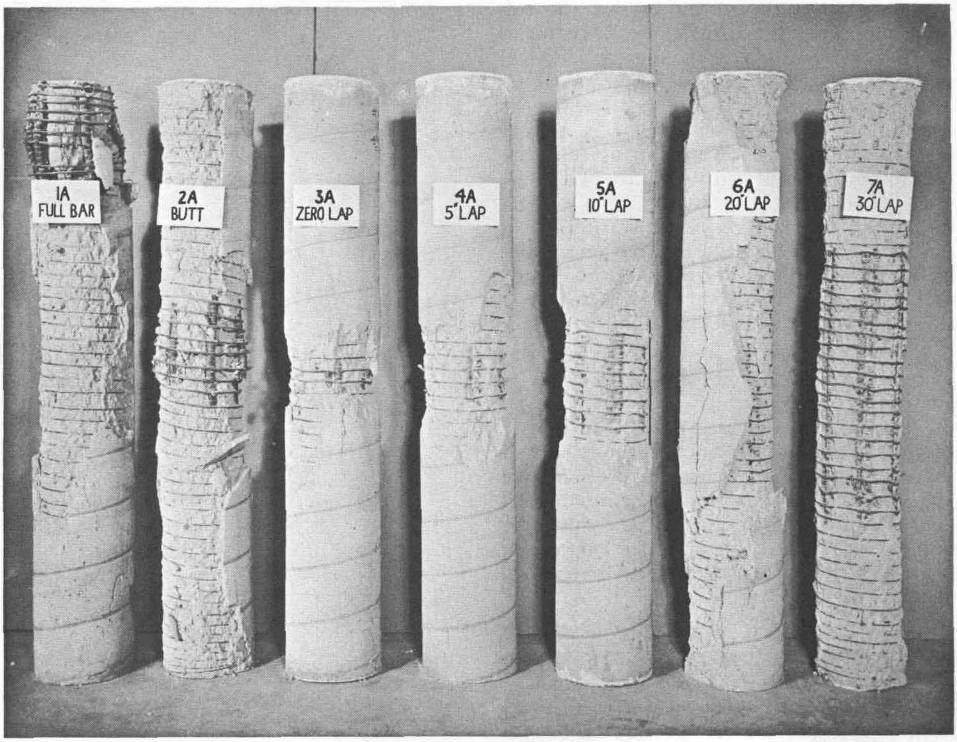

Fig. 7 — Series A Columns After Test.

ally increasing length of splice from zero to 30 in. is readily seen in Fig. 6. In all cases failure occurred in the region of the splice, and for splice lengths of up to 10 in. the ultimate load was reached shortly after vertical cracks were observed in the shell of the column at strains of about 0.002 in./in. For splice lengths of 20 and 30 in., a greater increase in load was observed after vertical cracks first appeared than in the case of the columns with shorter splice lengths. The failures appeared to be due to crushing of the concrete and breaking of the spiral after prior slip of the longitudinal reinforcement at the splice. The appearance of the Series A columns after test may be seen in Fig. 7.

TABLE 2—SERIES A TEST RESULTS

Column No.	Type	Ultimate Load Carried by Column, kips	Load Carried by Steel, kips	Steel Stress at Ultimate Load, ksi
1A	Full Bar	814	471	99.4
2A	Butt	805	441	93.0
3A	Zero Lap	525	183	38.6
4A	5 in. Lap	603	237	50.0
5A	10 in. Lap	623	251	52.9
6A	20 in. Lap	725	318	67.0
7A	30 in. Lap	769	392	82.6

It should be noted that no further increase in load occurred after the strain commenced to increase rapidly in the columns incorporating lapped splices. This is contrary to the behavior usually observed in concentrically loaded, spirally reinforced columns with continuous longitudinal reinforcement, of which column 1A is typical.

The ultimate loads carried by the Series A columns are listed in Table 2. Also given in Table 2 are the stresses in the longitudinal reinforcement, away from the splice, at the time the columns carried their maximum load. Since no strain gages were provided away from the splices, these reinforcement stresses were calculated by the method described below.

The cross-sections of all columns of this series were identical except at the splices, so it was considered that at all stages of loading, the load in the unspliced regions would be divided between the steel and

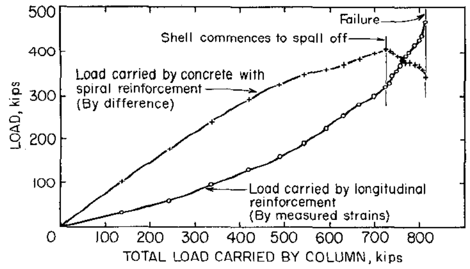

Fig. 8 — Division of Load Between Longitudinal Reinforcement and Concrete, Column No. 1A.

the concrete in the same proportions in all columns. The ultimate load carried by the longitudinal reinforcement at sections away from the splice was therefore taken as the load measured in the reinforcement of the unspliced Column 1A when that column was subjected to a total load equal to the total load causing failure of the spliced column.

The load carried by the longitudinal reinforcement in Column 1A was determined from the measured strains in the column and the stress-strain curve for the steel. This was done for increments of strain of 0.00025 in./in., and in Fig. 8 the load carried by the longitudinal reinforcement obtained in this manner is plotted against the total load acting on the column.

Uncertainties in this method of dividing the load were recognized, so additional instrumentation was used in Series B to provide a more accurate method and also to provide a check on the accuracy of the above procedure.

Series B — Rectangular Section Tied Columns

Column 1B, without splices, failed by a gradual crushing of the concrete followed by buckling of the longitudinal bars and disintegration of the concrete core. Buckling of the bars did not occur until after the maximum load had been passed. Vertical cracks were first observed shortly before failure at a longitudinal concrete strain of about 0.002 in./in. Crushing of the concrete commenced at a strain of about 0.0026. The load-strain curve for a 40-in. gage length at mid-height of the column is shown in Fig. 9, and for SR-4 strain gages on the longitudinal reinforcement away from the splice in Fig. 10.

Column 2B, having the butt joint in the reinforcement, failed by a sudden crushing of the concrete above the splice fol-

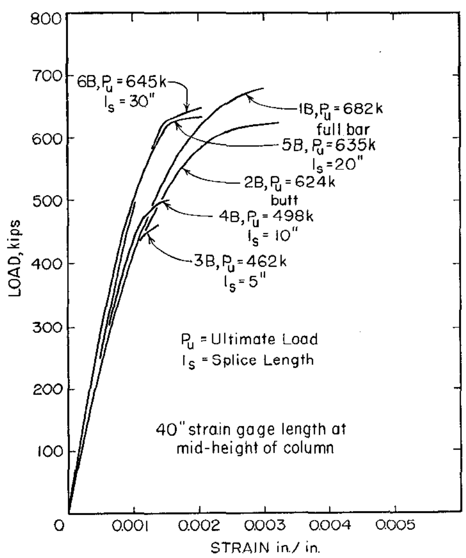

Fig. 9 — Load-Strain Curves for Series B Columns with Reinforcement "As Rolled."

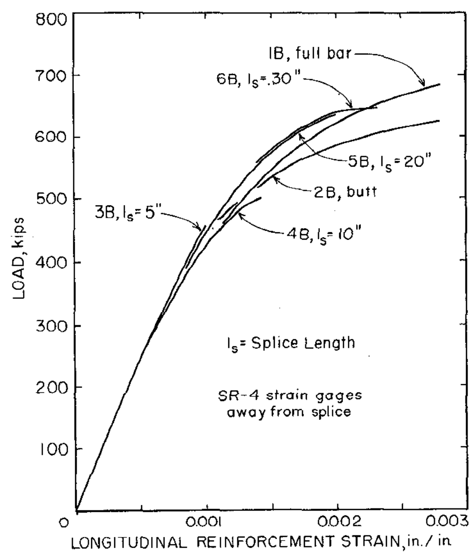

Fig. 10 — Load-Longitudinal Reinforcement Strain Curves for Series B Columns with Reinforcement "As Rolled."

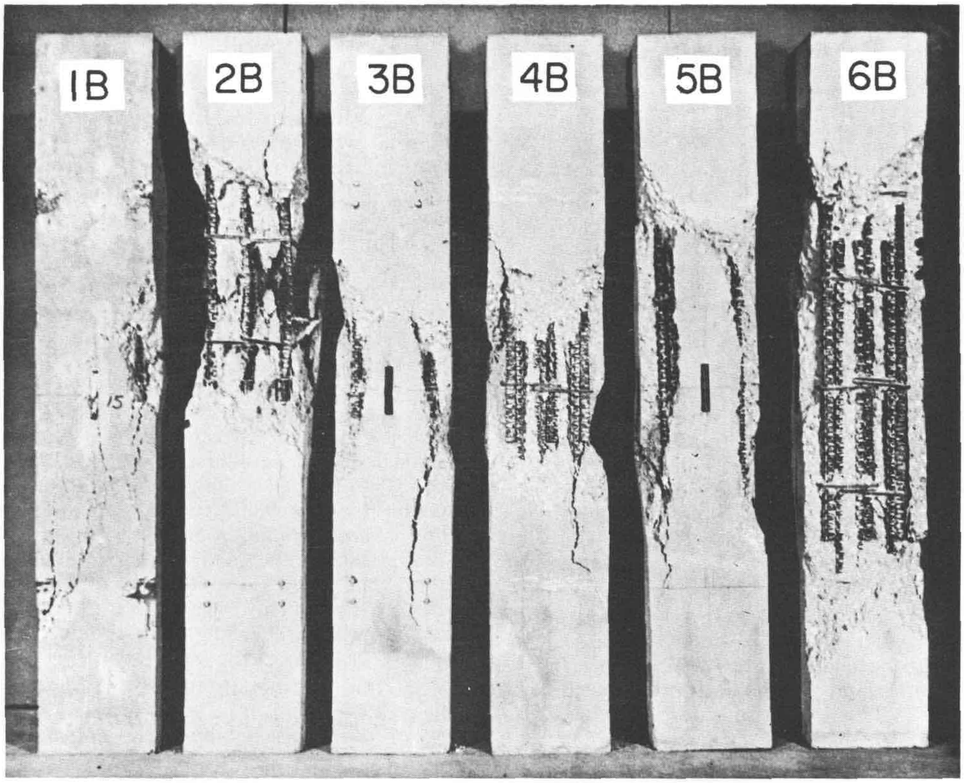

Fig. 11 — Condition of Columns 1B through 6B After Test.

lowed by buckling of the reinforcement. The concrete strains measured at the level of the joint and the strain in the column measured over the 40-in. gage length both exceeded the strain measured in the longitudinal reinforcement away from the splice. This indicated a discontinuity of the reinforcement probably due to initially poor bearing between the bar ends at the butt joints. However, local deformation at high loads must have improved this bearing since the same longitudinal reinforcement stress was finally developed away from the splice at ultimate strength in this column as in Column 1A in which the bars were continuous from end to end of the column.

Columns 3B, 4B, 5B, and 6B failed very suddenly by slipping of the reinforcement at the splice together with splitting and crushing of the concrete. First slipping of the reinforcement in the splice could be detected by comparing the shortening of the column measured over the 40-in. gage length with the strain in the reinforcement measured by SR-4 gages away from the splice. Using this method, first slipping was detected at 0.75, 0.80, 0.95, and 0.97 of ultimate load, respectively, for these columns. The condition of Columns 1B through 6B after test can be seen in Fig. 11.

Columns 1B1, 5B1, and 6B1 were identical with Columns 1B, 5B, and 6B, except that the longitudinal reinforcement had been heat treated to produce a more linear stress-strain curve up to the yield point. It was expected that higher steel stresses would be developed in these columns at ultimate strength if the column strains at ultimate strength were the same for corresponding columns. This was indeed found to be the case. The modes of behavior and failure for Columns 1B1, 5B1, and 6B1, were similar to those of Columns 1B, 5B, and 6B. The load-reinforcement strain curves for these columns are shown in Fig. 12.

The ultimate loads carried by the Series B columns are listed in Table 3. Also

TABLE 3—SERIES B TEST RESULTS

Column No.	Type	Ultimate Load Carried by Column, kips	Steel Strain at Ultimate Load, in./in.	Steel Stress at Ultimate Load, ksi
1B	Full Bar	682	0.0028	65.5
2B	Butt	624	0.0028	65.5
3B	5 in. Lap	462	0.0010	29.0
4B	10 in. Lap	498	0.0014	40.0
5B	20 in. Lap	635	0.0020	53.0
6B	30 in. Lap	645	0.0023	58.0
1B1	Full Bar	728	0.0026	71.5
5B1	20 in. Lap	659	0.0023	64.5
6B1	30 in. Lap	688	0.0024	68.0

given in this table are the stresses in the longitudinal reinforcement, as measured by SR-4 gages away from the splice, at the time the columns carried their maximum load. The strains are converted into stresses by the two curves of Fig. 4.

As a check on the method used to determine the longitudinal reinforcement stresses in the Series A columns, the same method was used to calculate the reinforcement stresses at ultimate strength for Columns 3B through 6B. The stresses calculated in this manner were found to be in close agreement with the stresses obtained from measured reinforcement strains and listed in Table 3.

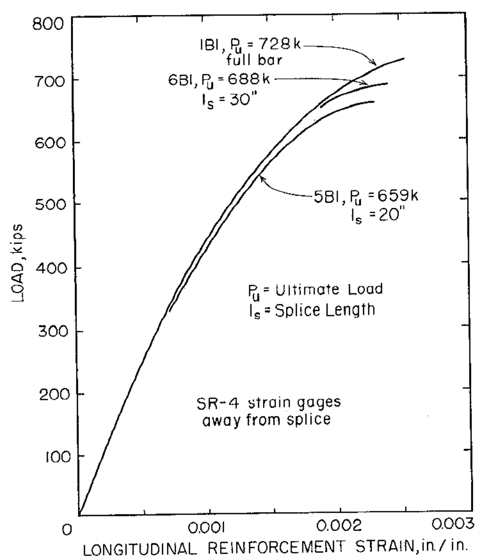

Fig. 12 — Load-Longitudinal Reinforcement Strain Curves for Series B Columns with Heat Treated Reinforcement.

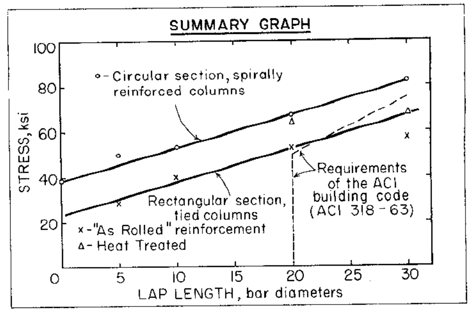

Fig. 13 — Variation with Lap Length of Stress in Longitudinal Reinforcement at Ultimate Strength of Column.

DISCUSSION OF TEST RESULTS

The stresses in the longitudinal reinforcement away from the splices, at the time of failure of the splices, are plotted against the lap length in Fig. 13. Also plotted in Fig. 13 are lines representing the requirements for lap splices in compression reinforcement set out in Section 805(c)1 of the 1963 ACI Building Code[5]. These requirements imply that for laps of 20, 24, and 30 bar diameters, respectively, yield point stresses of 50, 60, and 75 ksi can be developed in the reinforcement away from the splice. In Fig. 13 it can be seen that even higher stresses than these were developed in the longitudinal reinforcement of the spirally reinforced columns for the specified lap lengths. In the rectangular tied columns the test results were less favorable. While a little over 50 ksi was developed for a lap length of 20 diameters, the 30-diameter splices developed only 58 to 68 ksi, depending on the nature of the steel. However, it should be noted that the 1963 Code requires a lower capacity reduction factor, ϕ, for tied columns than for spirally reinforced columns.

Behavior of Lapped Splices

It can be seen from the trend of results given in Fig. 13 for both types of columns that the effectiveness of a lapped splice is not directly proportional to its length. It appears that the force in the longitudinal bars is transferred by a combination of bond on the surface of the bars and end bearing of the bars on the concrete. The linear variation of maximum reinforcement stress with length of lap seen in Fig. 13 is consistent with a constant end bearing stress and a constant limiting average bond stress. The end bearing stress is different in the two column types, but the limiting bond stress appears to be very closely the same for both types since the straight lines drawn through the two sets of data are very nearly parallel.

Assuming that the straight lines drawn in Fig. 13 represent the two sets of data reasonably closely, it is found that the limiting average bond stress for both types of column is 370 psi, and that the end bearing stresses are $38/2=19$ and $23/2=11.5$ ksi for the spirally reinforced and rectangular tied columns respectively*. The average cylinder strength for the columns represented in Fig. 13 was 3700 psi.

The relatively low average bond stress of 370 psi observed at failure is probably due to the fact that, at this stage of loading, the concrete in the column is on the verge of failing in axial compression. Richart, Brandtzaeg and Brown(8) showed that at a concrete compressive stress equal to about 85 per cent of its failure stress, internal longitudinal cracks commence to form in the concrete. Cracking of this kind could be expected to reduce the stress at which a bond failure would occur.

The attainment of end bearing stresses of up to five times the cylinder strength of the concrete is due to the triaxial nature of the stresses at the end of the bar. Since the high bearing stresses exist over a relatively small part of the cross-section of the column, the concrete of the column surrounding the region of high bearing stress will provide lateral restraint to that highly stressed region. Additional restraint will also be provided by the lateral reinforcement. Many triaxial tests(8) have shown that lateral restraint can result in the normal stress at failure being several times the stress causing failure in a uniaxial compression test, such as the standard cylinder test. The higher end bearing stress in a spirally reinforced column is consistent with the greater degree of lateral restraint afforded by the closely spaced spiral than by the more widely spaced lateral ties.

*The total area subject to end bearing in a splice will be equal to the sum of the reinforcement cross-sectional areas above and below the splice. In this case the areas are equal, hence the bearing stress is half the stress in the reinforcement away from the splice, developed by a splice of zero length.

Addition Law Equation

An alternative way in which the performance of the various splices may be compared is by the use of a modified form of the addition law. Whereas for a column with continuous reinforcement the addition law for ultimate strength may be expressed as:

$$P_u = 0.85\, f'_c\, A_c + f_y\, A_s \quad \ldots\ldots (1)$$

the following modified form of this equation may be written for the strength of a column with spliced reinforcement:

$$P_u = 0.85\, f'_c\, A_c + f_s\, A_s \quad \ldots\ldots (2)$$

where f_s may be called the effective steel stress away from the splice at ultimate strength of the column.

Equation (2) may be transposed to yield f_s if the load at failure and f'_c are known:

$$f_s = \frac{P_u - 0.85\, f'_c\, A_c}{A_s} \quad \ldots\ldots (3)$$

This stress is fictitious since it is calculated assuming that away from the splice the concrete is crushing at a stress equal to $0.85\, f'_c$. In actual fact failure occurs within the splice and the concrete away from the splice does not crush. However, the value of f_s obtained in this manner does give a measure of the contribution of the spliced reinforcement to the strength of a column, assuming the contribution of the concrete to be the same as in a column with continuous reinforcement.

The values of f_s calculated in this manner from the test results reported in this paper are given in Table 4, as are the reinforcement stresses obtained from measured strains. It can be seen that the stresses

TABLE 4—EFFECTIVE REINFORCEMENT STRESSES

Column No.	Type	$(P_u-0.85f'_cA_c)$, kips	Computed f_s, ksi	Measured Steel Stress, from Strains, ksi
1A	Full Bar	510	107.5	99.4
2A	Butt	466	98.3	93.0
3A	Zero Lap	196	41.4	38.6
4A	5 in. Lap	278	58.6	50.0
5A	10 in. Lap	298	62.9	52.9
6A	20 in. Lap	402	84.8	67.0
7A	30 in. Lap	446	94.1	82.6
1B	Full Bar	320	67.5	65.5
2B	Butt	283	59.7	65.5
3B	5 in. Lap	130	27.4	29.0
4B	10 in. Lap	147	31.0	40.0
5B	20 in. Lap	243	51.3	53.0
6B	30 in. Lap	250	52.7	58.0
1B1	Full Bar	369	77.8	71.5
5B1	20 in. Lap	263	55.5	64.5
6B1	30 in. Lap.	344	72.6	68.0

calculated in this manner are in reasonable agreement with the stresses obtained from the measured strains, but tend to be more erratic.

This in some measure is due to the fact that the coefficient 0.85 in Equation (1) was derived as an average value for a large number of tests, but the actual value in individual tests may vary between about 0.75 and 1.00. Due to the form of Equation (3) variations in this coefficient are reflected directly in the calculated value of f_s

Design Considerations

Though the tests reported here are exploratory in nature, so that full ranges of applicable variables are not covered, the results suggest three design considerations:

Lapped Splices — The test results indicate that the splice lengths of 20, 24, and 30 bar diameters are adequate to develop 50, 60 and 75 ksi, respectively, in the longitudinal reinforcement of spirally reinforced columns. For tied columns, these splices are less effective, particularly for 75-ksi reinforcement. Together with other aspects of column strength, this confirms the desirability of the relatively low capacity reduction factor, ϕ, for tied columns given in Section 1504(b) of the 1963 ACI Code.

Other Splices — The tests confirmed that "stress may be transmitted by end bearing of square-cut ends held in contact by a suitably welded sleeve" as indicated by Section 805(c)2 of the 1963 ACI Code.

It is felt that, when 75-ksi reinforcement is used in tied columns, splices should preferably be made by bearing of square-cut ends, by welding, or by other positive connections. Lapped splices of 75-ksi reinforcement should preferably be spirally reinforced.

In future usage of 90-ksi column reinforcement, lapped splices will probably be impractical even with spiral reinforcement.

Reinforcement Yield Strain — Comparison of the ultimate loads carried by Columns 1B and 1B1 confirms the desirability of the longitudinal reinforcement having as near a linear stress-strain curve as possible up to the specified yield stress. It is apparent from these and other tests that development of reinforcement stresses corresponding to strains in excess of about 0.003 in./in. will generally not be possible at ultimate strength in an axially loaded tied column. The introduction of Section 1505(a) into the 1963 ACI Code is therefore warranted. This section states that, "When reinforcement is used that has a yield point, f_y, in excess of 60,000 psi, the yield point to be used in design shall be reduced to 0.85 f_y or 60,000 psi, whichever is greater, unless it is shown by tension tests that at a proof stress equal to the specified yield point, f_y, the strain does not exceed 0.003 in./in." The reinforcement used in Column 1B reached its specified yield point of 75,000 psi at a strain of 0.0036. According to Section 1505(a) the yield point to be used in calculation of the ultimate strength of the column would therefore have to be taken as 0.85 $\times$ 75,000 = 63,750 psi. This compares well with the steel stress measured at ultimate strength of Column 1B, which was 65,500 psi.

CONCLUSIONS

The test results reported in this paper suggest the following conclusions:

1. A lapped compression splice transfers force by a combination of bond on the surface of the reinforcing bars and end bearing of the bars on the concrete.

2. The requirements of Section 805(c)1 of the 1963 ACI Building Code governing lap splices in compression reinforcement are reasonably conservative for spirally reinforced columns having longitudinal reinforcement with a yield point of up to 75 ksi. These same requirements are adequate for tied columns reinforced with bars having a yield point of up to 60 ksi, but may be unconservative for bars having a yield point of 75 ksi.

3. Butt joints in reinforcing bars in compression, made in accordance with the final sentence of Section 805(c)2 of the 1963 ACI Code are able to develop the same stress at ultimate strength of a column as would a continuous bar without any joint.

4. If the specified yield point of longitudinal reinforcement in tied columns is to be developed at ultimate strength of the columns, then it is necessary that the yield point be reached at or before a strain of 0.003 in./in. This condition will normally be more readily complied with by bars having a clearly defined yield point and a nearly linear stress-strain curve up to yield than by bars having a gradually curving stress-strain curve with no clearly defined yield point.

FURTHER INVESTIGATION OF SPLICES

This exploratory study suggests that further investigation of splices of high strength longitudinal reinforcement in columns is desirable. Planning of such future studies will be carried out in collaboration with the "Task Committee on Reinforcing Details" of the Reinforced Concrete Research Council. Future experimentation may be undertaken at the PCA Laboratories and elsewhere.

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NOTATION

A_c = net cross-sectional area of concrete

A_s = cross-sectional area of longitudinal reinforcement

f'_c = compressive strength of concrete measured on a 6 x 12-in. cylinder

f_s = effective steel stress away from the splice at ultimate strength of the column

f_y = yield point of longitudinal reinforcement

l_s = length of a lapped splice

P_u = ultimate load carried by column

ϕ = capacity reduction factor used in 1963 ACI Building Code.

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