Title No. 61-32

Influence of Ties on the Behavior of Reinforced Concrete Columns

By JAMES F. PFISTER

To aid development of the 1963 ACI Building Code, 11 rectangular tied columns were tested under concentric load to explore the influence of arrangement and spacing of lateral ties on the strength and behavior of tied columns. In three of the columns, full ties were provided as required by the 1956 Code, and in another three columns only exterior ties were used. Two columns had ties only at the ends and at midheight of the columns, and three columns were provided with ties only at their ends.

It was found that the primary function of the ties was to restrain the concrete laterally so that it could develop its full strength in a gradual type of compression failure. Exterior ties surrounding the longitudinal reinforcement were found to be as effective as combined interior and exterior ties conforming to the 1956 Code. It is concluded that the new tie requirements of the 1963 Code should be entirely adequate.

Key words: code requirements; column; high strength steel; rectangular column; reinforced concrete; tie; tied column; ultimate strength.

■ THE MINIMUM REQUIREMENTS of the 1956 ACI Building Code for lateral ties in tied columns are similar to the requirements embodied in the codes governing reinforced concrete construction in most other countries. In the 1963 revision of the ACI Code, these tie requirements were considerably relaxed.

SCOPE

To aid ACI Committee 318 in revising the 1956 ACI Code, this investigation was undertaken to explore the effect of arrangement and spacing of ties on the behavior and ultimate strength of concentrically loaded tied columns. The principal variables considered were form and spacing of ties, shape of column cross section, and the yield point of the longitudinal reinforcement.

No consideration was given in this investigation to the influence of arrangement and spacing of ties on the shear strength and ductility of tied columns, so important in the design of structures for earthquake motions.

BACKGROUND

In nonseismic areas, ties have traditionally been provided with a view toward reducing the possibility of local buckling of the longitudinal reinforcement under loads approaching the ultimate strength of the column. To this end, the 1956 ACI Code[1] requires that ties of at least ¼ in. diameter be provided at intervals not exceeding 16 bar diameters, 48 tie diameters, or the least lateral dimension of the column. The 1956 ACI Code further requires that the ties be so arranged that every longitudinal bar has lateral support provided by a bend or corner of a tie having an included angle of not more than 90 deg. These requirements lead to tie arrangements of the kind shown in Fig. 1, which is taken from the 1957 ACI Detailing Manual (ACI 318-57).[2]

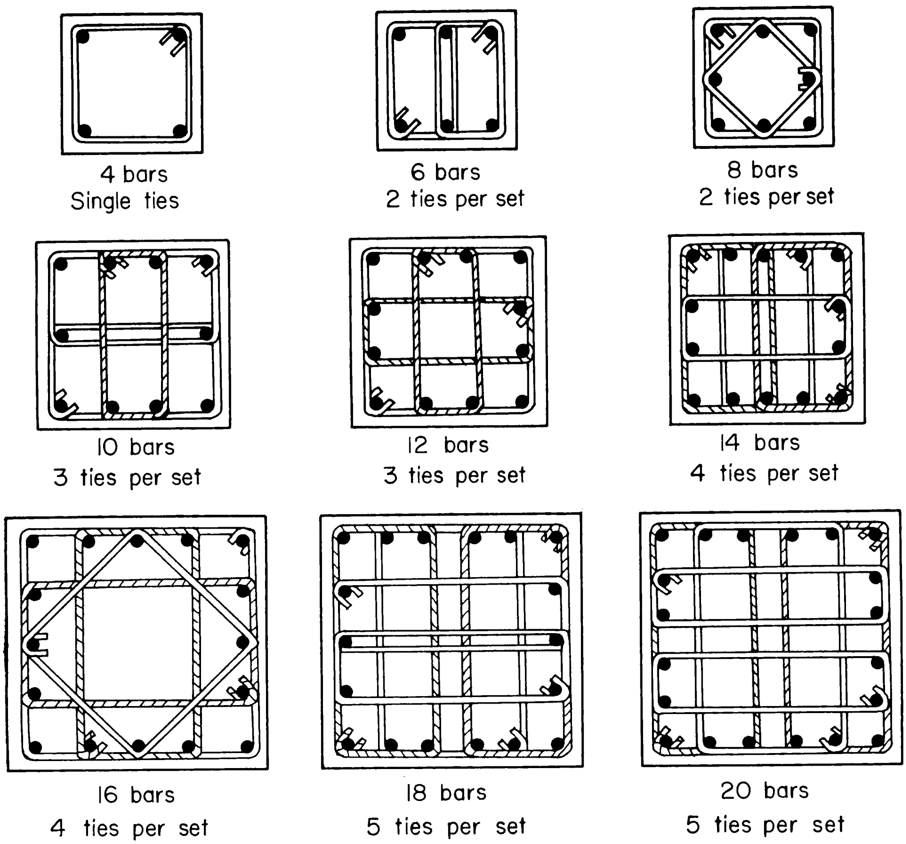

Fig. 1—Arrangement of ties in 1957 ACI Detailing Manual

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Bresler and Gilbert[3] made a theoretical analysis of the stability of the longitudinal bars in a tied column, and reported the results of tests on four tied columns embodying alternate designs of lateral ties. On the basis of their analytical and test results they tentatively concluded that "lighter ties than currently (1956 ACI Code) used would be adequate for reinforced concrete columns, but that in columns with high strength longitudinal reinforcing bars closer tie spacing than presently required should be used." They were particularly concerned with the elimination or reduction in size of the interior ties. In two of their test columns, the interior ties were reduced from ¼ in. diameter bar to 1/16 in. diameter wire, both rod and wire having approximately the same yield point. The columns incorporating the 1/16 in. wire interior ties performed as well as similar columns containing the ¼ in. diameter ties as required by the 1956 ACI Code.

In addition to the obvious economy resulting from a reduction in the amount of steel used, elimination or reduction in the amount of interior tie elements in a tied column would lead to several other benefits. The detailing and fixing of the column reinforcement would be simplified, and the placing and compaction of the concrete within the core of the column would be facilitated. This last benefit could result in an increase in the concrete core strength, since Larsson[4] has shown in tests of walls that the obstruction to compaction and settlement of concrete by reinforcement can result in a reduction in concrete strength. It is clear from a study of Fig. 1 that tie arrangements involving several interior ties will probably impede the compaction and settlement of the concrete in the column core to a considerable extent. Indeed, such interior ties probably decrease total column strength under practical construction conditions.

TEST SPECIMENS

Series A—Square section with intermediate grade reinforcement

All the test columns in this series were 12 x 12 in. in cross section with an over-all length of 72 in. Each column was reinforced longitudinally by 12 #6 intermediate grade reinforcing bars. The elevations and sections of the columns are shown in Fig. 2. Lateral reinforcement consisted of ties made from #2 intermediate grade deformed bar provided as follows:

Column 1A—Ties conforming to the requirements of the 1956 ACI Code were provided at 12-in. centers over the middle 60 in. of the column. At the two ends of the column, ties were grouped closely together to prevent end splitting of the column under load.

Column 2A—Ties were provided at the same spacing as in Column 1A but consisted only of exterior rectangular ties enclosing the longitudinal reinforcement and the column core.

Column 3A—A single exterior rectangular tie was provided at mid-height of the column, together with a group of three ties at each column end.

Column 4A—No ties were provided within the middle 66 in. of the column. Groups of three ties were provided at each column end.

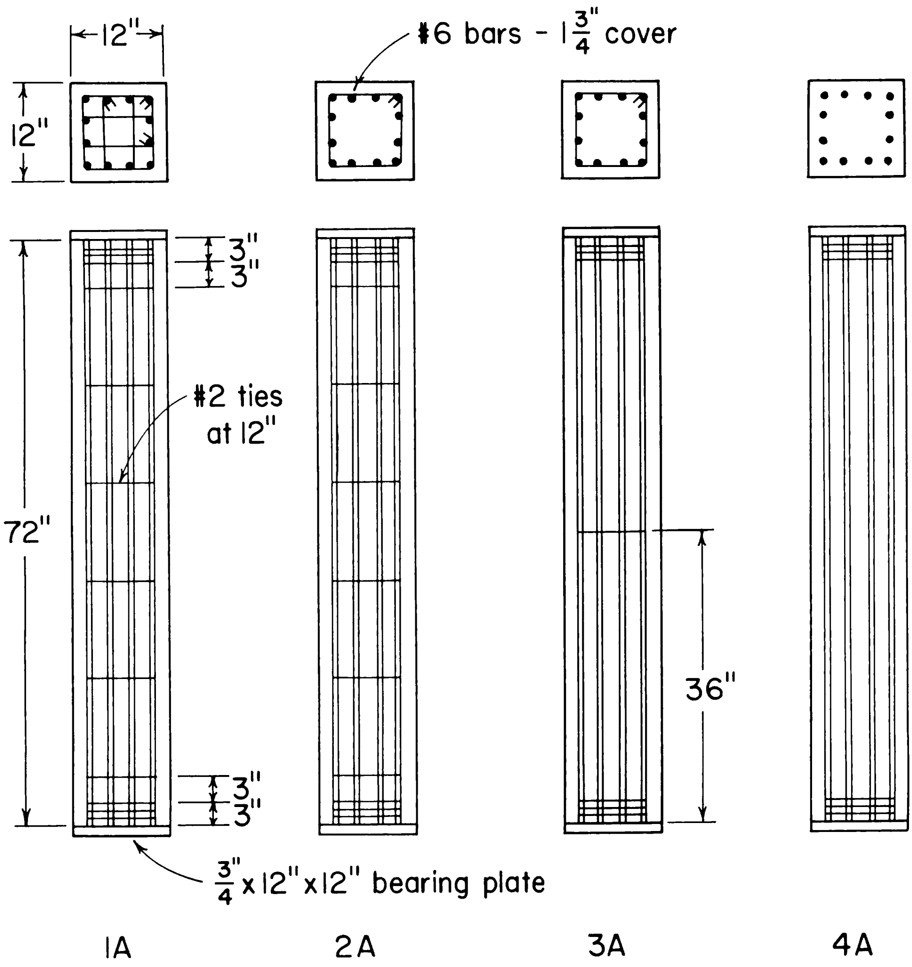

Fig. 2—Details of Series A columns

Series B—Rectangular section with intermediate grade reinforcement

The columns in this series were 8 x 18 in. in cross section and 72 in. long. The elevations and sections of the columns are shown in Fig. 3. The columns were reinforced longitudinally by 12 #6 intermediate grade reinforcing bars, and the lateral reinforcement consisted of ties made from #2 intermediate grade deformed bar. As in Series A, groups of three ties were provided at each end of each column to guard against end splitting. Other ties were provided as follows:

Column 1B—Ties conforming to the requirements of the 1956 ACI Code were provided at 8-in. centers over the middle 64 in. of the column.

Column 2B—Ties were provided at the same spacing as in Column 1B, but consisted only of exterior rectangular ties enclosing the longitudinal reinforcement.

Column 3B—No ties were provided within the middle 64 in. of the column.

Series C—Rectangular section with high strength reinforcement

The 10 x 12 in. section columns of this test series had an over-all length of 72 in. Elevations and sections of these columns are shown in Fig. 4. The longitudinal reinforcement of each column consisted of six #8 high strength steel bars conforming to ASTM A 431-59T for 75,000-psi yield. The lateral reinforcement consisted of ties made from #2 intermediate grade deformed bar. As in Series

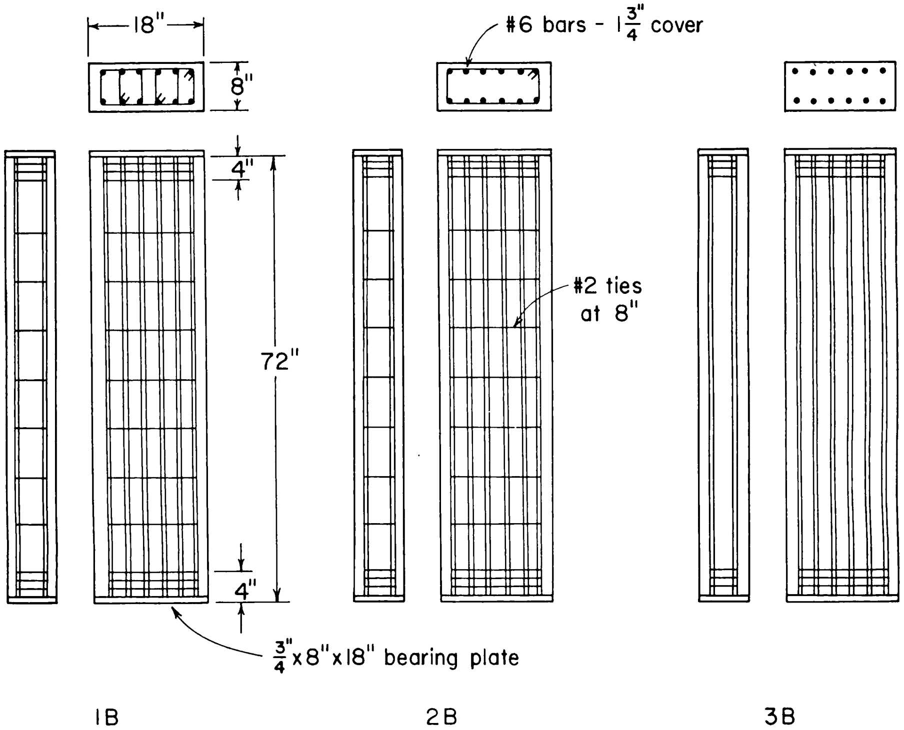

Fig. 3—Details of Series B columns

A and B, groups of three ties were provided at each end of each column. Other ties were provided as follows:

Column 1C—Ties conforming to the requirements of the 1956 ACI Code were provided at 10-in. centers over the middle 60 in. of the column.

Column 2C—Ties were provided at the same spacing as in Column 1C, but consisted only of exterior rectangular ties enclosing the longitudinal reinforcement.

Column 3C—A single exterior tie was provided at midheight of the column.

Column 4C—No ties were provided within the middle 66 in. of the column.

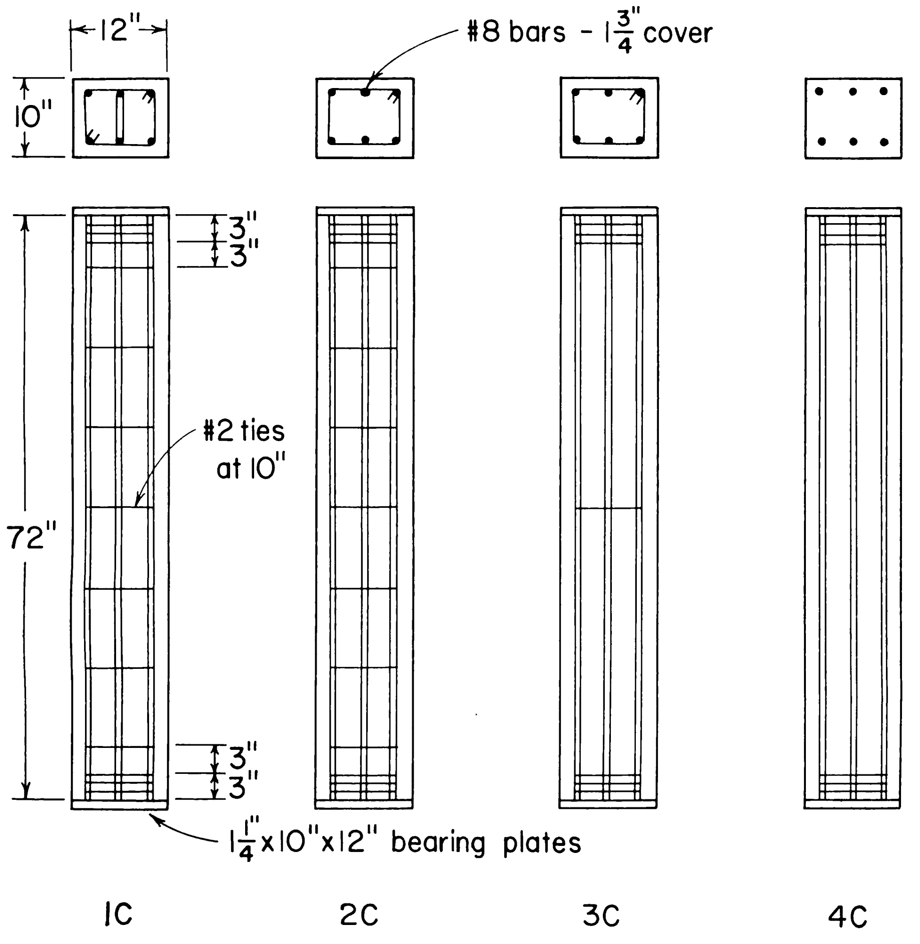

Fig. 4—Details of Series C columns

Materials

The longitudinal reinforcement in the Series A and B columns was of #6 intermediate grade deformed bars with a yield point of 48 kips per sq in. The longitudinal reinforcement used in the Series C columns was of #8 high strength deformed bars conforming to ASTM A 431-59T, and having a yield strength of 92 kips per sq in. as determined in accord with applicable ASTM standards at a strain of 0.006. The stress-strain curves for the longitudinal reinforcement are shown in Fig. 5. The lateral ties in all three series of columns were made from #2 intermediate grade deformed bar with a yield point of 58.5 kips per sq in.

All longitudinal reinforcement conformed to ASTM A 305 for deformations. Although #2 bars are not covered by ASTM A 305, the #2 bars used for ties were deformed in a similar manner to larger bars conforming to this designation.

The concrete used in the Series A and B columns contained 3.5 bags per cu yd of Type III portland cement, and that in the Series C columns 4.5 bags per cu yd of Type I portland cement.*

A ¾ in. maximum size aggregate was used, and 4 to 5 percent of air was entrained in the concrete for all the columns. The columns were cured at 70 F, sealed in the forms for 3 days, and were subsequently stored at 70 F and 50 percent relative humidity. The concrete strengths at the time of test are shown in Table 1. The strengths quoted in this table are the average of three 6 x 12-in.

*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.

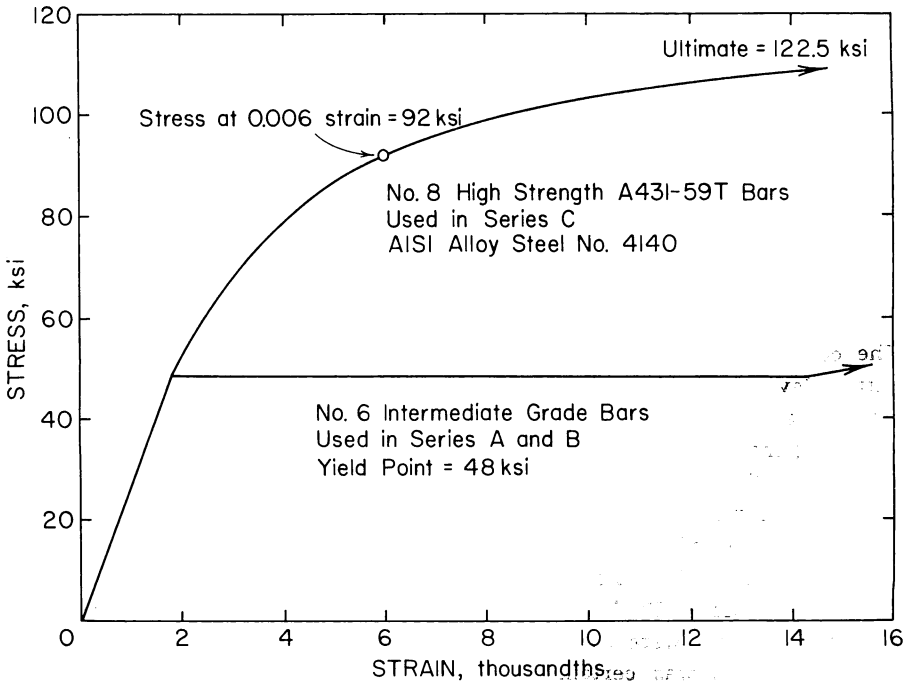

Fig. 5—Stress-strain curves for longitudinal reinforcement

TABLE 1—CONCRETE STRENGTH AT TIME OF TESTING (SERIES A AND B AT 14 DAYS, SERIES C AT 7 DAYS)

Column No.	Cylinder strength, psi	Column No.	Cylinder strength, psi	Column No.	Cylinder strength, psi
1A	3790	1B	4310	1C	3600
2A	3820	2B	4350	2C	3680
3A	3820	3B	4350	3C	3780
4A	3840	—	—	4C	3690

cylinders cast at the time of fabrication of the columns, and stored alongside the columns until the testing time. Series A and B columns were tested at 14 days, and the Series C columns at 7 days.

Fabrication

All columns were cast in a horizontal position, thus eliminating any effects the ties might have on concrete compaction. In the Series A and B columns, the longitudinal reinforcing bars were spot welded to ¾ in. steel plates at each end to assure positive bearing and alignment of the bars. In the Series C columns, the ends of the longitudinal bars were milled and a shallow ¼ in. diameter hole drilled and tapped in each end. The milled ends butted to 1¼ in. thick end plates and were held in place by screws, thus assuring positive bearing and alignment of the bars. The #2 bar ties were then tied in place and the reinforcement cage placed in the form. The concrete was compacted by internal vibration.

Instrumentation

All columns were instrumented with SR-4 electrical resistance strain gages, which were monitored continuously throughout the tests by a strip chart recorder. The SR-4 gages were all located at midheight of the column, and were attached to the longitudinal reinforcement, to a transverse tie, and to the surface of the concrete.

TEST PROCEDURE

The columns were tested in a 1,000,000 lb capacity testing machine. To facilitate leveling, the columns were placed on a thin bed of high strength plaster on a steel plate resting on the bottom platen of the testing machine. The testing machine head was lowered so as to bring the upper platen into contact with the steel plate on the top of the column. Wedges were then inserted to prevent rotation of the machine head under load. The columns were therefore tested with their ends effectively "fixed."

The load was applied at a uniform rate of 50 kips per min until failure of the column occurred. During the tests the strain gages were monitored continuously. The tests were continued until the columns had deformed to such an extent that it was certain that the maximum load capacity had been developed.

TABLE 2—TEST RESULTS

Column No.	Type	P_{calc}, kips	P_{test}, kips	$\frac{P_{test}}{P_{calc}}$
1A	Full ties	701	684	0.98
2A	Exterior ties	704	695	0.99
3A	Single tie	704	700	0.99
4A	No ties	707	650	0.92
1B	Full ties	762	762	1.00
2B	Exterior ties	767	774	1.01
3B	No ties	767	751	0.98
1C	Full ties	655	654	1.00
2C	Exterior ties	663	646	0.98
3C	Single tie	673	624	0.93
4C	No ties	664	622	0.94

TEST RESULTS

The ultimate loads carried by the columns are given in Table 2. These measured ultimate loads are also compared in Table 2 with the ultimate loads calculated by the addition law equation:

$$P_{calc} = 0.85 f_c'A_c + f_yA_s \quad \text{(1)}$$

where

f_c' = concrete compressive strength measured on a 6 x 12-in. cylinder

A_c = net concrete cross sectional area, sq in.

f_y = yield point of longitudinal reinforcement, psi

A_s = cross sectional area of longitudinal reinforcement, sq in.

The intermediate grade reinforcement used in Series A and B had a clearly defined yield point and the stress corresponding to this yield point was used in Eq. (1) to calculate the ultimate strength of the columns in these two series. The high strength reinforcement used in Series C had no clearly defined yield point and reached its specified yield point of 75 kips per sq in. at a strain of 0.0036. In accordance with Section 1505(a) of the 1963 ACI Code,[5] the yield stress used in Eq. (1) when calculating the ultimate strength of the Series C columns, was taken as 85 percent of the specified minimum yield strength, that is $0.85 \times 75 = 63.75$ kips per sq in.*

It can be seen in Table 2 that, for all columns except Columns 4A, 3C, and 4C, the actual ultimate strength measured in the tests was within 2 percent of the calculated ultimate strength. All these columns failed

*If the yield stress in Eq. (1) is taken as 85 percent of the actual 92 kips per sq in. yield, the P_{test}/P_{calc} ratios will range from 0.85 to 0.90.

gradually. The first visible sign of failure was a slow crushing of the concrete shell of the column. When loading was continued beyond the point at which maximum load was reached, the longitudinal reinforcing bars buckled.

Columns 4A, 3C and 4C failed very suddenly by violent disruption of the concrete, which took place after longitudinal cracking of the concrete shell. Buckling of the longitudinal reinforcing bars occurred *after* failure of the concrete. These columns developed ultimate strengths from 6 to 8 percent less than the calculated values.

In Table 3 are shown the contributions of the longitudinal reinforcement and the concrete to the ultimate strength developed by each of the columns. The values of steel stress at ultimate strength were obtained from the measured strains in the longitudinal reinforcement shown in Fig. 6, using the stress-strain relationships in Fig. 5. The load carried by the concrete was obtained by subtracting from the measured ultimate load, the measured steel stress multiplied by the reinforcement cross section. In Table 3 the computed concrete stress at ultimate strength of the columns has been tabulated as a proportion of the concrete cylinder strength. It can be seen that in all the columns except Columns 4A, 3C, and 4C, the concrete compressive stress at ultimate strength of the columns was very close to 0.85 f_c', the value determined in the ACI Column Investigation[6] to be the average concrete compressive stress

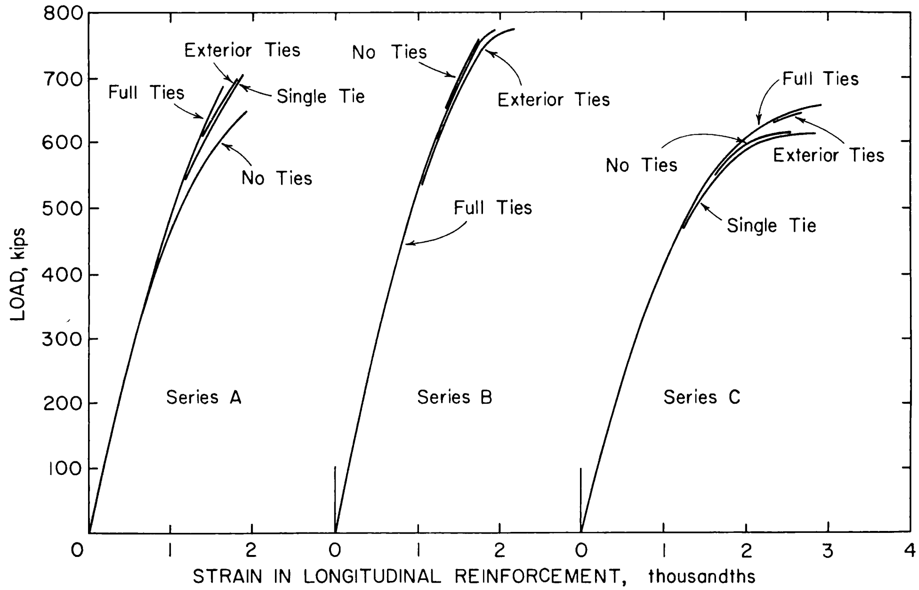

Fig. 6 — Variation with load of strain in longitudinal reinforcement

TABLE 3—COMPONENT LOADS

Column No.	Type	Steel stress at ultimate load, f_{su}, ksi	Load carried by steel $P_s = A_s f_{su}$, kips	Load carried by concrete $P_{test} - P_s$, kips	$\frac{P_{test} - P_s}{A_c f_c'}$
1A	Full ties	45.5	240	444	0.85
2A	Exterior ties	48.0	253	442	0.83
3A	Single tie	48.0	253	447	0.84
4A	No ties	48.0	253	397	0.74
1B	Full ties	48.0	253	509	0.85
2B	Exterior ties	48.0	253	521	0.84
3B	No ties	48.0	253	498	0.83
1C	Full ties	66.5	315	339	0.82
2C	Exterior ties	62.5	296	350	0.83
3C	Single tie	65.5	311	313	0.72
4C	No ties	64.0	304	318	0.75

when an adequately tied column reaches its ultimate strength. In these columns the failure mode of the concrete was similar to that observed in 6 x 12 in. concrete cylinders tested in the standard manner so that crushing of the concrete and formation of "shear cones" occurs, which is in accord with findings in earlier column tests.

In Columns 4A, 3C, and 4C the concrete compressive stress at ultimate column strength was 0.74, 0.72, and 0.75 f_c', respectively. The lower concrete compressive stresses at ultimate strength are compatible with the different failure mode of the concrete observed in these columns, as compared with that observed in all the other columns tested. This difference in failure mode can be seen in Fig. 7. Columns 1A and 1C which had full ties, failed in the normal manner found in numerous previous tests. In the columns without ties, however, longitudinal splitting was observed over an abnormal length of each column before failure occurred. The actual failure occurred so suddenly that it was difficult to observe the sequence of events as failure progressed. However, it appears that the final destruction of the concrete was a combination of crushing and outward buckling of the slender longitudinal strips of concrete into which the longitudinal cracks had divided the outer parts of the column. This is similar to the type of failure which occurs in a test cylinder when its ends are treated so as to greatly reduce the friction between its ends and the platens of the testing machine, thus reducing the lateral restraint at the ends of the cylinder. This treatment can result in a reduction in strength of up to 20 percent as compared with the strength of an identical cylinder tested in the standard manner. In Columns 4A, 3C, and 4C the concrete stress at ultimate strength of the columns was 13, 15, and 12 percent below the value of 0.85 f_c' for adequately tied columns.

This difference in behavior between columns with normally spaced ties and those with no ties at all or with a single tie was anticipated by Bresler and Gilbert[3] from a consideration of the effect of end restraint on the failure mode of concrete cylinders. They concluded that, "To obtain the maximum effectiveness of the concrete core . . . ties should have a spacing equal to or less than twice the core dimension of the column." Although Column 3B had no ties between its ends, it did not have a significantly lower ultimate strength than comparable Columns 1B and 2B, which had ties at the spacing called for in the 1956 ACI Code. Neither was the concrete stress at ultimate strength in Column 3B significantly less than that which occurred in Columns 1B and 2B. The reasons are not clear for the superior performance of Column 3B as compared with that of Columns 4A and 4C, both of which were also without ties between their ends. The result may be a random high value, but also may perhaps have been influenced by the elongated form of cross section of Column 3B. Longitudinal cracking in a column

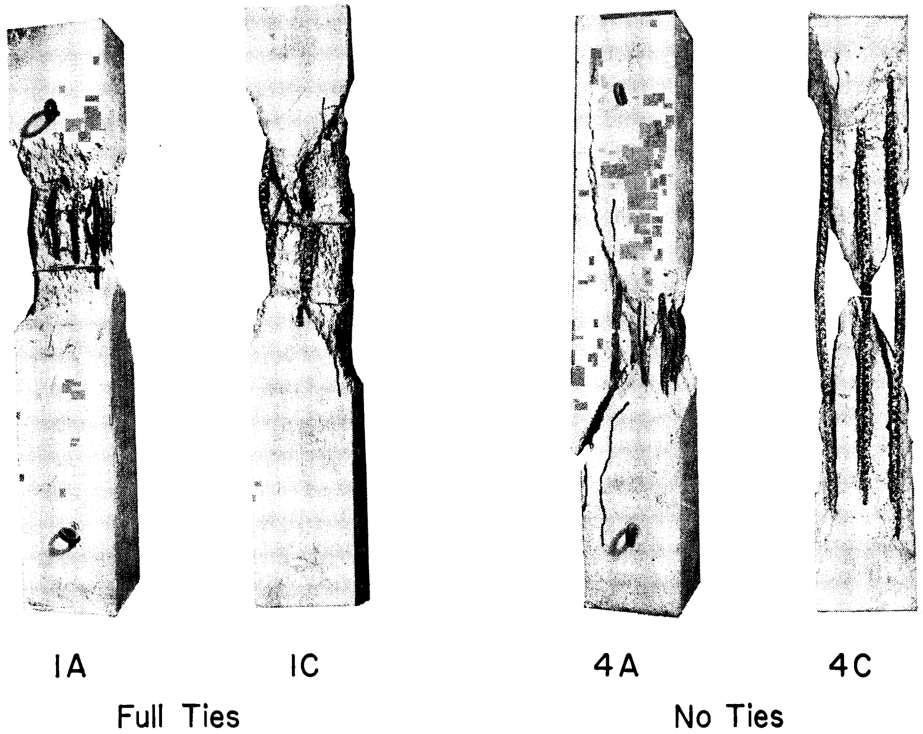

Fig. 7—Columns after failure

with an elongated rectangular cross section occurs primarily in the longer faces. The cracks tend to divide the concrete into strips which have a weak axis, relative to buckling, parallel to the short faces of the column. The strips near the center of the long face of the column will be restrained from buckling about their weak axis by adjacent strips. Hence, a true compression failure of the concrete may occur in this region of the column cross section, rather than a combined buckling-compression failure. This would result in a higher average concrete compressive stress at ultimate strength of the column than is the case in an untied "near-square" section column failing by combined buckling and crushing of the concrete.

The relationships between the strains measured in the ties and the strain in the longitudinal reinforcement are plotted in Fig. 8 for each of the tied columns tested. Also plotted in this figure is the relationship between lateral and longitudinal strain measured in the test of a 4000 psi concrete cylinder. It can be seen that, for longitudinal strains up to about 0.0018 the strains in the ties were close to the lateral strain measured in the 6 x 12 in. concrete cylinder. It appears, therefore, that the strains developed in the ties of a tied column are primarily due to lateral expansion of the concrete when subjected to longitudinal compression. The curves for tie strains in Fig. 8 are discontinued by arrows where yielding of the longitudinal reinforcement led to rapid increases in both longitudinal and lateral strains.

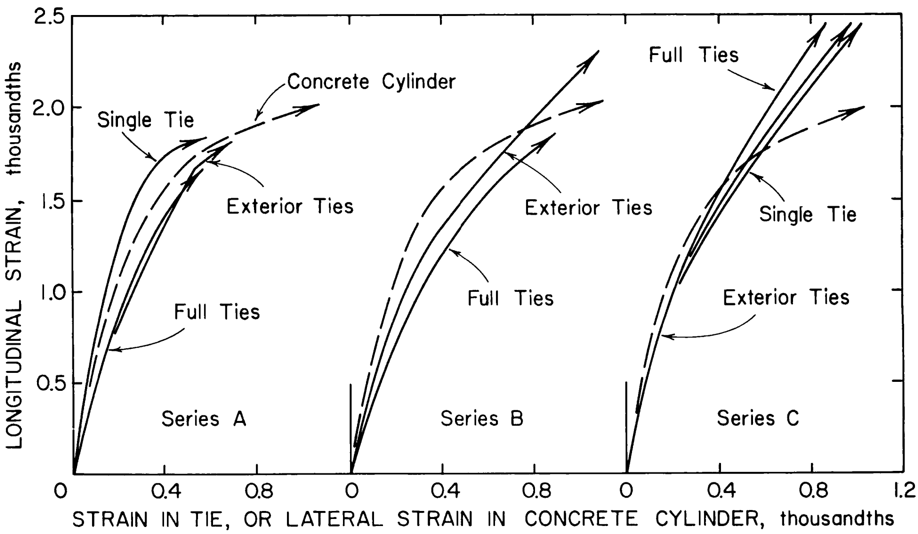

Fig. 8—Strain variation in lateral ties

1963 ACI BUILDING CODE

The requirements for interior ties were relaxed considerably in the 1963 ACI Code as compared to the 1956 Code. Both codes call for ties at least ¼ in. in diameter spaced apart not more than 16 bar diameters, 48 tie diameters, or the least dimension of the column. When there are more than four vertical bars, the 1956 Code calls for interior ties to hold each longitudinal bar in a manner equivalent to a 90-deg corner of a tie. The 1963 Code, however, calls for ties to hold every corner and alternate longitudinal bar by ties having an included angle of not more than 135 deg, provided that no bar be more than 6 in. from a supported bar.

This change leads to a reduction of interior ties for most of the columns shown in Fig. 1. For example, the new rules will permit a reduction in interior ties for columns with 16 and 20 longitudinal bars as shown in Fig. 9.

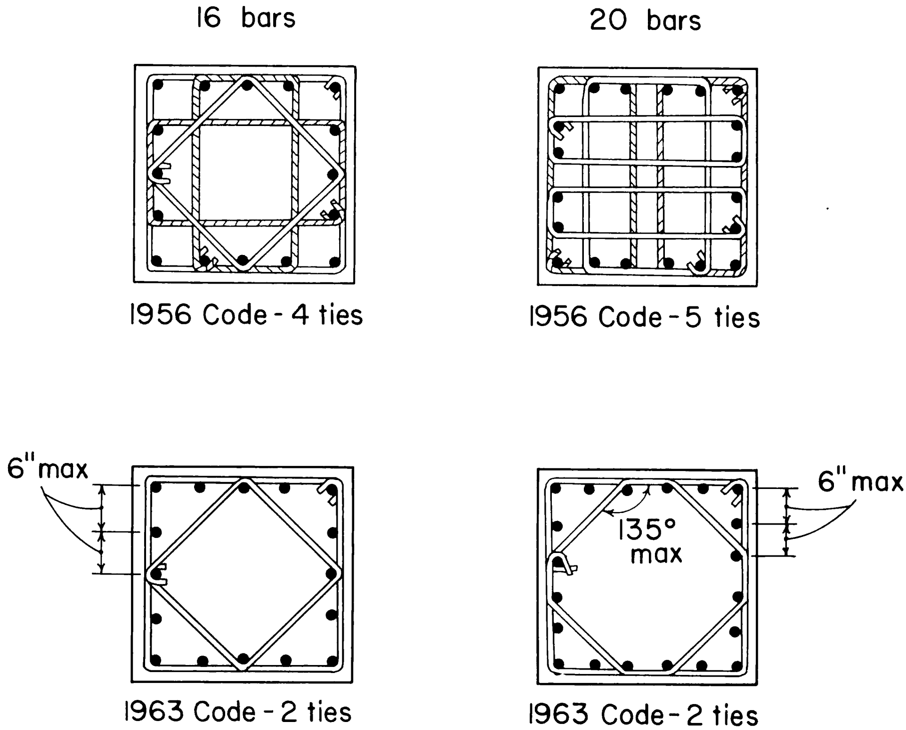

Fig. 9—Reduction in interior ties by 1963 ACI Code

CONCLUDING REMARKS

The results obtained in this exploratory study indicate that the primary function of ties in an axially loaded tied column is to provide lateral restraint for the concrete. The lateral restraint causes the column to fail in a more gradual manner than would be the case if ties were not provided. It also enables the concrete to develop a larger resistance to compression at ultimate strength of the column. It further appears that simple rectangular ties provide as effective lateral restraint as that provided by ties conforming to the requirements of the 1956 ACI Code. This finding is in agreement with the recommendations of Blume, Newmark, and Corning[7] as to the form of additional ties provided to confine the concrete in columns subject to earthquake motions.

In view of the above findings, of the possibility of achieving better quality concrete in the column core, and of the economic advantages which would follow a reduction in the amount and complexity of ties, it seems both reasonable and desirable to reduce the number of interior ties in tied columns, where shear is not a controlling design factor.

It is considered that similar conclusions would be arrived at from a study of the behavior of eccentrically loaded tied columns. Curvature of a column due to the action of an eccentric load would reduce the possibility of outward buckling of the longitudinal reinforcing bars subject to compression, compared with the case of bars in a concentrically loaded column in which no curvature results from the applied load. It could therefore be expected that lateral restraint of the concrete would be the main function of ties in an eccentrically loaded column, as in the case of a concentrically loaded column.

It is believed that the new tie requirements given in the 1963 ACI Code are adequate. Through improvement in concrete compaction, they will probably even lead to an improved total column strength under practical construction conditions.

ACKNOWLEDGMENTS

The investigation reported herein was carried out in the Structural Laboratory of the Portland Cement Association under the direction of Eivind Hognestad and Alan H. Mattock. Contributions were made by several members of the laboratory staff. Particular credit is due R. K. Richter and R. J. Morand, expert technicians, for their contributions to the testing and instrumentation work involved.

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Sinopsis — Résumés — Zusammenfassung

Influencia de los Zunchos en el Comportamiento de Columnas de Concreto Reforzadas

Para ayudar en el desarroyo del Reglamento ACI 1963, se ensayaron 11 columnas rectangulares zunchadas, sometidas a cargas concéntricas con el propósito de explorar la influencia de la distribución y espaciamiento de los zunchos laterales en la resistencia y comportamiento de las columnas zunchadas. En tres (3) de las columnas se proveyó zunchado completo como requerido por el Reglamento 1956, y en otras tres, (3) columnas sólo se usaron zunchos exteriores. Dos de las columnas se zuncharon sólo en los extremos y medio, y tres columnas fueron zunchadas sólo en sus extremos.

Se encontró que la función principal de los zunchos fué la de restringir lateralmente al concreto de manera que pudiera desarrollar su resistencia total en un tipo de fallo a compresión gradual. Los zunchos exteriores rodeando el refuerzo longitudinal se encontraron ser tan efectivos como combinando zunchos interiores y exteriores conforme al Reglamento de 1956. Se concluye que los nuevos requisitos de zunchos del Reglamento de 1963 debieran ser enteramente adecuados.

Influence des Tirants sur le Comportement des Colonnes en Béton Renforcées

Pour aider le développement du Code du Bâtiment ACI 1963, on a épreuvé 11 colonnes rectangulaires tirées sous charge concentrique pour explorer l'influence de l'arrangement et l'espacement des tirants laterales sur la résistance et le comportement des colonnes tirées. Dans trois d'entre elles, on a employé des tirants complets selon le Code 1956; dans trois autres, des tirants extérieurs seulement. Deux colonnes avaient des tirants aux extremités et au mi-hauteur; trois colonnes étaient tirées seulement à leurs bouts.

On a trouvé que la fonction primaire des tirants est de restreindre le béton lateralement de sorte qu'il puisse développer pleine résistance dans un type de rupture de compression graduelle. Les tirants extérieures qui entouraient l'armature longitudinale étaient aussi éfficaces qu'une combinaison de tirants extérieures et intérieures selon le Code 1956. On conclut que les nouvelles exigences du Code 1963 doivent être entièrement adéquates.

Einfluss von Bügeln auf das Verhalten von Stahlbetonpfeilern

Zum Zwecke der Weiterentwicklung der ACI Baurichtlinien von 1963 wurden 11 rechteckige bügelbewehrte Stützen unter konzentrischer Belastung geprüft, um den Einfluss von Anordnung und Abstand der seitlichen Bügel auf Festigkeit und Verhalten bügelbewehrter Stützen festzustellen. In drei von den Pfeilern wurden volle Bügel vorgesehen in Uebereinstimmung mit den Anforderungen der Richtlinien von 1956, und in drei anderen Pfeilern wurden nur die äusseren Bügel benutzt. Zwei Pfeiler hatten Bügel nur an den Enden und in der Mitte, und drei Pfeiler waren mit Bügeln nur an ihren Enden ausgestattet.

Es wurde festgestellt, dass die Hauptfunktion der Bügel war, den Beton seitlich in Schranken zu halten, so dass er seine volle Festigkeit in einer graduellen Art von Druckversagen entfalten konnte. Es zeigte sich, dass äussere Bügel, die die längsweise Bewehrung umgeben, ebenso wirksam sind wie die kombinierten inneren und äusseren Bügel gemäss den Richtlinien von 1956. Es wird daraus gefolgert, dass die neuen Bügelanforderungen der Richtlinien von 1963 völlig hinreichend sind.

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