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Reinforced Concrete Columns Confined by Circular Spirals and Hoops

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Twenty-seven short concrete columns reinforced with longitudinal steel and circular spirals or hoops were tested to failure under monotonic axial compression. Effects of different variables, such as amount and type of lateral steel, lateral steel spacing, and specimen size, on the behavior of columns were investigated. The relation between lateral pressure on concrete and concrete strength enhancement, and the variation of spiral steel stress and confinement effectiveness coefficient k *with respect to the amount of spiral steel were also investigated. Requirements of the ACI 318-89 Building Code related to the minimum volumetric ratio of spiral reinforcement and the maximum spiral pitch of 80 mm (3 in.) were critically examined. An increase in the volumetric ratio of spiral steel was found to significantly improve strength and ductility of confined concrete, the effect on ductility being more pronounced. The maximum effect of spiral steel spacing was observed for the amount of spiral steel, which was approximately equal to that required by the ACI code. The specimen size appeared to have no significant effect on the behavior of similarly confined columns of different sizes. In well-confined specimens, the confinement effectiveness coefficient* k *corresponding to the maximum concrete force was between 2.1 and 4.0.*

Keywords: columns (supports); confined concrete; ductility; **hoops;** lateral pressure; **reinforced concrete;** strength.

Previous work on confinement dates back to the beginning of this century, when Considere[1] first pointed out the beneficial effects of confining axially loaded concrete using spiral reinforcement. In the 1920s and 1930s, others[2,3] studied confinement effects under lateral fluid pressure and then in spirally reinforced concrete columns, and proposed an equation to express the axial strength of spirally reinforced columns as

$$f'_{cc} = f'_c + 4.1 f_2 \qquad (1)$$

A satisfactory response of reinforced concrete structures is based on the ability of the sections to carry the imposed load accompanied by a certain ductility. Use of confining reinforcement in reinforced concrete columns in ACI 318-89,[4] especially in earthquake-resistant structures, is based on two conditions: 1) increase in compressive strength of concrete due to confinement should offset the strength loss due to cover spalling; and 2) columns should be able to sustain large deformations without a dramatic loss in strength. The current code provisions for spiral reinforcement, based on the ACI Committee 105 recommendations of 1933,[5] require that the volumetric ratio of spiral reinforcement ρ_s for nonseismic design of columns should not be less than the value given by

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y} \qquad (2)$$

where the specified yield strength of spiral steel f_y is not greater than 60,000 psi (400 MPa). The seismic provisions[4] require that in addition to satisfying Eq. (2), the spiral reinforcement ratio should also not be less than that given by the following equation

$$\rho_s = 0.12 f'_c / f_y \qquad (3)$$

RESEARCH SIGNIFICANCE

Use of spiral steel in a column results in enhancement of strength and ductility of concrete. Whereas the replacement of cover concrete contribution toward the load-carrying capacity of a column by the enhanced strength of confined concrete is a convenient and plausible criterion for the design of spiral reinforcement,[4] the enhancement in ductility is a more important outcome of confinement, considering extensive redistribution of forces at large deformations. The ACI Building Code[4] specifies the maximum allowable spiral spacing of 3 in. (80 mm), irrespective of column size, which for large columns may prove unnecessarily conservative and difficult to construct. A minimum of six longitudinal bars required in small columns may also be unnecessary.

The main objective of this paper is to present the results of an experimental program conducted to investigate the effects

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of different variables, such as amount and spacing of spiral steel and specimen size, on the behavior of confined concrete, and to evaluate the requirements of ACI 318-89 related to the minimum volumetric ratio and maximum spacing of spiral steel. Enhancement of concrete strength due to spiral steel is also examined critically, particularly with respect to the use of factor 4.1 in Eq. (1), which is also used in the development of Eq. (2).

EXPERIMENTAL PROGRAM

Test specimens

A total of 27 specimens consisting of nine each of 14-in. (356-mm), 10-in. (254-mm), and 8-in. (203-mm) diameter columns reinforced with spirals or hoops and longitudinal steel were tested under monotonic concentric compression. The height of each specimen was four times its diameter.

Concrete cover was provided in all the specimens. The minimum cover requirement of 1.5 in. (40 mm) was satisfied in the specimens, considering a prototype column to be of 24-in. (610-mm) diameter. This provided a clear cover thickness of 0.875, 0.675, and 0.5 in. (22, 17, and 13 mm) in 14-, 10-, and 8-in (356-, 254-, and 203-mm) diameter specimens, respectively. In all the specimens, the ratio of gross area of the section A_g to the core area measured to the outside of the lateral reinforcement A_c was 1.306.

In view of the constraints provided by the sizes of the available steel bars, longitudinal reinforcement ratio, capacities of the testing machines, and the requirement that specimens of different sizes be comparable, five longitudinal bars were used. Use of five bars instead of the minimum six required by the ACI Building Code[4] was believed to have minimal effect on the behavior of spirally confined concrete columns. A concrete cover of 1 in. (25 mm) was provided between the ends of the longitudinal bars and the top and bottom surfaces of the specimens to prevent direct loading of the bars.

Failure of the specimens was forced in the test region, which was equal approximately to the diameter of the specimen plus 2 in. (50 mm) in the middle of the specimen height, by reducing the spacing of lateral steel outside the test region to two-thirds of the specified spacing in the test region. The spiral spacing in the end regions, approximately equal to the column diameter, was further reduced to one-third of the specified spacing. The specimens were also externally confined by steel collars in the end regions, to further prevent premature failure there.

One specimen of each size was laterally reinforced with circular hoops, while all the others were spirally reinforced. Table 1 gives the details of the specimens. The alphanumeric characters in the labels of the specimens (e.g., D14-S10M-P3.0) have the following meaning. The first group consisted of letter "D," followed by a number representing the diameter of the column in inches. The second group, starting with letter "S" or "H," represents the spiral or hoop steel size. The third group represents the pitch of the spiral (or hoop reinforcement) in inches. In calculating lateral steel required by Eq. (2) and (3), actual yield strengths of lateral steel that were greater than 400 MPa were used. For steel with roundhouse stress-strain curves, yield stress was based on a 0.2 percent offset method. The ACI 318-89 requirement for the minimum spiral steel ratio for $A_g/A_c = 1.306, f'_c = 35$ MPa, and $f_y = 400$ MPa is 1.15 percent.

Material properties

Concrete—Ready-mix, normal weight, normal strength concrete with 10-mm (0.4-in.) maximum-size round aggregate with a 75-mm (3-in.) slump was used in all the specimens. Specified 28-day compressive strength was 35 MPa. Thirty-six 6 x 12-in. (150 x 300-mm nominal) concrete cylinders were also cast with the specimens and tested at various ages to develop a strength-versus-age relationship to determine the strength of concrete on the day of column testing.

Reinforcing steel—25M, 20M, and 15M deformed bars were used as longitudinal reinforcement in 14-, 10-, and 8-in. (356-, 254-, and 203-mm) diameter specimens, respectively. Stress-versus-strain curves for these bars are provided in Fig. 1. The stress-strain behaviors of lateral steel, which consisted of 10M, 8-mm, D5 (6.4-mm), D4 (5.7-mm) deformed bars and 3⁄16-in. (4.8-mm) undeformed cold-rolled wire, are provided in Fig. 2. Nominal cross-sectional areas were used for stress calculations in the case of 10M, 15M, 20M, and 25M bars. For other bars, areas based on actual diameters were used.

Instrumentation and data acquisition—Four linear variable differential transducers (LVDTs) were mounted vertically on the north, south, east, and west sides of the test region of each specimen using the threaded rods, which were previously installed before the concrete was placed. The LVDTs were utilized to obtain continuous plots of load-versus-longitudinal deformation of the test region. The gage lengths for these LVDTs were equal approximately to specimen diameter plus 50 mm (2 in.). A fifth LVDT was mounted horizontally in the test region to measure the diametral deformation of the concrete core. Fig. 3 shows a typical column with LVDTs and steel collars in the end regions.

All the specimens had seven electrical strain gages installed on the longitudinal bars in the test region. Two of the five longitudinal bars had two strain gages each, whereas the remaining three bars had one each. In addition, each specimen had four strain gages installed on the spiral (or hoop) reinforcement within the test region. These strain gages were 90 deg apart along the spiral perimeter. Among the test specimens, one 8-in. (203-mm) diameter specimen did not have any strain gage installed on the 3⁄16-in. (4.8-mm) lateral reinforcement due to the smallness of the wire.

Loading procedure—The specimens were tested under monotonically increasing axial compression. The load was applied from zero to failure, which was determined primarily by either the rupture of the lateral reinforcement or excessive crushing of core concrete, together with buckling of the longitudinal bars.

Table 1 — Specimen details

Specimen no.	Specimen label	Lateral steel required, ρ_s [Eq. (2)]	Lateral steel provided	ρ_s. provided	f_c', MPa
1	D14-S10M-P2.2	0.0110	10M @ 56 mm	0.0230	35.9
2	D14-S10M-P3.0	0.0110	10M @ 76 mm	0.0169	35.9
3	D14-S10M-P4.4	0.0110	10M @ 112 mm	0.0115	35.9
4	D14-S10M-P6.0	0.0110	10M @ 152 mm	0.0085	35.9
5	D14-S8M-P2.2	0.0083	8M @ 56 mm	0.0115	35.9
6	D14-S8M-P3.0	0.0083	8M @ 76 mm	0.0085	35.9
7	D14-S8M-P4.4	0.0083	8M @ 112 mm	0.0058	35.9
8	D14-SD4-P2.2	0.0085	D4 @ 56 mm	0.0059	35.9
9	D14-H10M-P3.0	0.0110	10M @ 76 mm	0.0169	35.9
10	D10-S10M-P3.1	0.0108	10M @ 79 mm	0.0230	35.5
11	D10-S10M-P4.3	0.0108	10M @ 109 mm	0.0167	35.5
12	D10-S8M-P1.6	0.0082	8M @ 41 mm	0.0223	35.5
13	D10-S8M-P2.1	0.0082	8M @ 53 mm	0.0170	35.5
14	D10-S8M-P3.1	0.0082	8M @ 79 mm	0.0115	35.5
15	D10-S8M-P4.3	0.0082	8M @ 109 mm	0.0084	35.5
17	D10-SD4-P1.6	0.0084	D4 @ 41 mm	0.0114	35.5
18	D10-SD4-P2.1	0.0084	D4 @ 53 mm	0.0087	35.5
19	D10-H8M-P2.1	0.0082	8M @ 53 mm	0.0170	35.5
20	D8-8M-P2.5	0.0080	8M @ 64 mm	0.0179	34.9
21	D8-SD5-P2.5	0.0078	D5 @ 64 mm	0.0115	34.9
22	D8-SD5-P2.5	0.0078	D5 @ 64 mm	0.0115	34.9
23	D8-SD5-P3.4	0.0078	D5 @ 86 mm	0.0086	34.9
24	D8-SD5-P1.7	0.0078	D5 @ 43 mm	0.0168	34.9
25	D8-SD5-P1.7	0.0078	D5 @ 43 mm	0.0168	34.9
26	D8-SD5-P1.7	0.0078	D5 @ 43 mm	0.0168	34.9
27	D8-S3/16-P1.7	0.0099	3/16 @ 43 MM	0.0093	34.9
28	D8-HD5-P2.5	0.0078	D5 @ 64 mm	0.0115	34.9

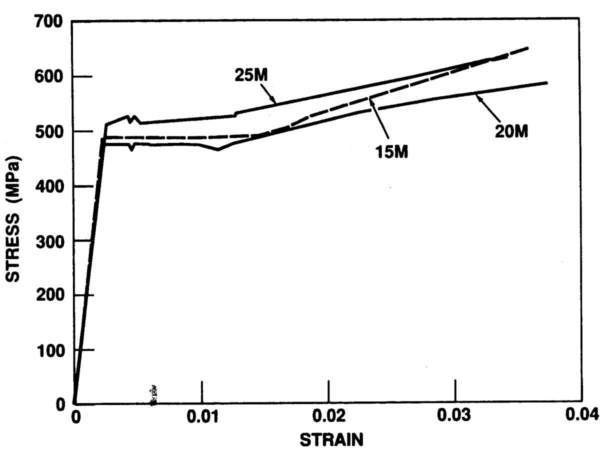

Fig. 1—Stress-strain relationships of longitudinal steel

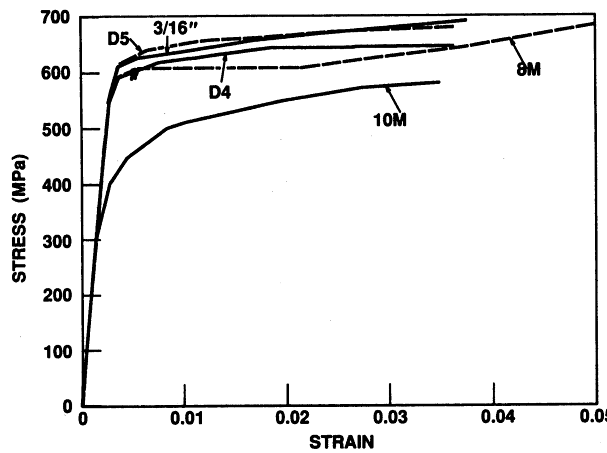

Fig. 2—Stress-strain relationships of lateral steel

To insure concentric loading, preparations before each test included monitoring of the readings from the strain gages and LVDTs for the range of loading between zero and 25 percent of the estimated ultimate load when the specimen was still in the elastic range. If any eccentricity in loading was detected, the specimen was unloaded and adjusted accordingly. To eliminate any nonuniform loading due to uneven top or bottom surfaces, plaster of Paris was applied at both ends of each specimen before testing.

To minimize the effect of eccentricity in loading due to uneven spalling of concrete cover and change in the location of the centroidal axis of the specimen, the spherical head was locked against rotation when 50 percent of the estimated ultimate load was reached. Since it was not possible practically to lock the spherical head when it was connected to the machine at the top of the specimen, it was located under the specimen. Overall view of the test setup is shown in Fig. 3. Fig. 4 shows a select group of specimens at the end of the tests.

The 14-in. (356-mm) diameter specimens were tested in a 1200-kip (5300-kN)-capacity universal testing machine, whereas 10- and 8-in. (254- and 203-mm) diameter specimens were tested in a 620-kip (2750-kN)-capacity MTS machine. In the interpretation of results, note that the MTS testing machine provided a servo-controlled loading mechanism, which was capable of applying load in a displacement-control mode, whereas loading in the universal testing machine was manually controlled.

TEST RESULTS

The columns were analyzed to obtain the stress-strain curves of confined concrete, as suggested by Sheikh and Uzumeri.[6] The concrete contribution at a certain deformation is determined by subtracting the contribution of longitudinal steel from the applied load (Fig. 5). The effect of buckling of longitudinal steel was also included, for which the stress-strain curve of steel in tension was assumed to represent its behavior in compression until buckling was observed. Most of the tests were continued until core concrete was completely destroyed and the load was carried only by the longitudinal bars that had extensively buckled. Between this point and the start of buckling, a straight line was assumed to represent longitudinal steel behavior. The concrete contribution curves were nondimensionalized with respect to gross concrete area force P_{oc} and core concrete area force P_{occ} (Fig. 6), where

$$P_{oc} = 0.85 f_c' A_{c0} \quad (4)$$

$$P_{occ} = 0.85 f_c' A_{cc} \quad (5)$$

While the gross concrete area represented column behavior before the cover started spalling, only the core concrete area resisted the applied load after concrete cover was completely spalled off. Therefore, a transition took place from the lower curve to the upper curve, which is assumed[6] to start at a strain value ε_{co}, which corresponds to the maximum plain concrete stress and ends at a strain value ε_{50u}, which corresponds to 50 percent of the peak stress of plain concrete on the descending branch of its stress-strain curve. Based on the standard cylinder tests, the two strain values ε_{co} and ε_{50u} were taken as 0.002 and 0.0035, respectively.

EFFECTS OF DIFFERENT VARIABLES ON CONFINED CONCRETE BEHAVIOR

Based on the analysis procedure described previously, the final stress-strain curves of confined concrete were established for all the specimens and used to evaluate the effects of different variables on confined concrete behavior.

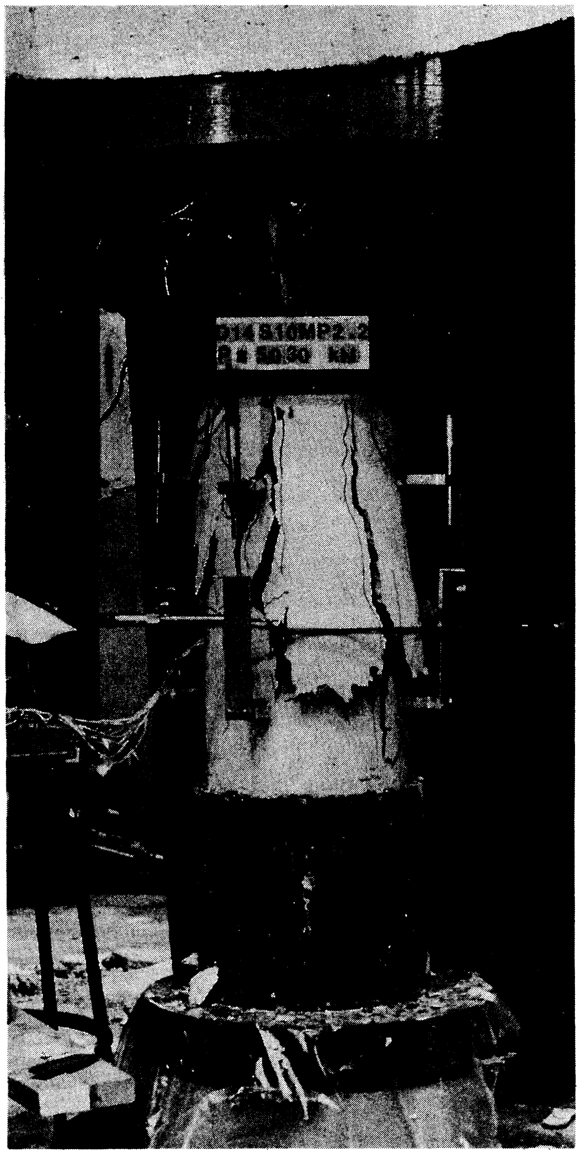

Fig. 3—Specimen during testing

The effects of volumetric ratio of lateral reinforcement ρ_s, spiral spacing s, specimen size, and hoop reinforcement on confined core behavior are presented graphically in Fig. 7 through 10. Table 2 also contains the test results. In addition to some physical properties, these figures also contain $\rho_s\sigma$ at peak concrete stress and $\rho_s f_y$ for each specimen. All the properties of the comparable specimens were kept reasonably the same except the parameter that was investigated for its effect. To achieve this, different sizes of spiral steel bars with some differences in strength had to be used in various specimens.

Effect of volumetric ratio of lateral steel

Fig. 7(a) through (c) show the effect of the volumetric ratio of lateral steel on the confined core behavior for 14-, 10-, and 8-in. (356-, 254-, 203-mm) diameter columns. In almost all cases, an increase in the volumetric ratio of spiral reinforcement resulted in an increase in σ/f_y, ductility, and strength of the confined concrete. The effect of the amount of spiral reinforcement on ductility was much more pronounced than on strength. In some cases, doubling the amount of spiral steel did not much affect strength. For most columns with s/D_c ratios equal to or greater than 0.36 and ρ_s less than 1.0 percent,

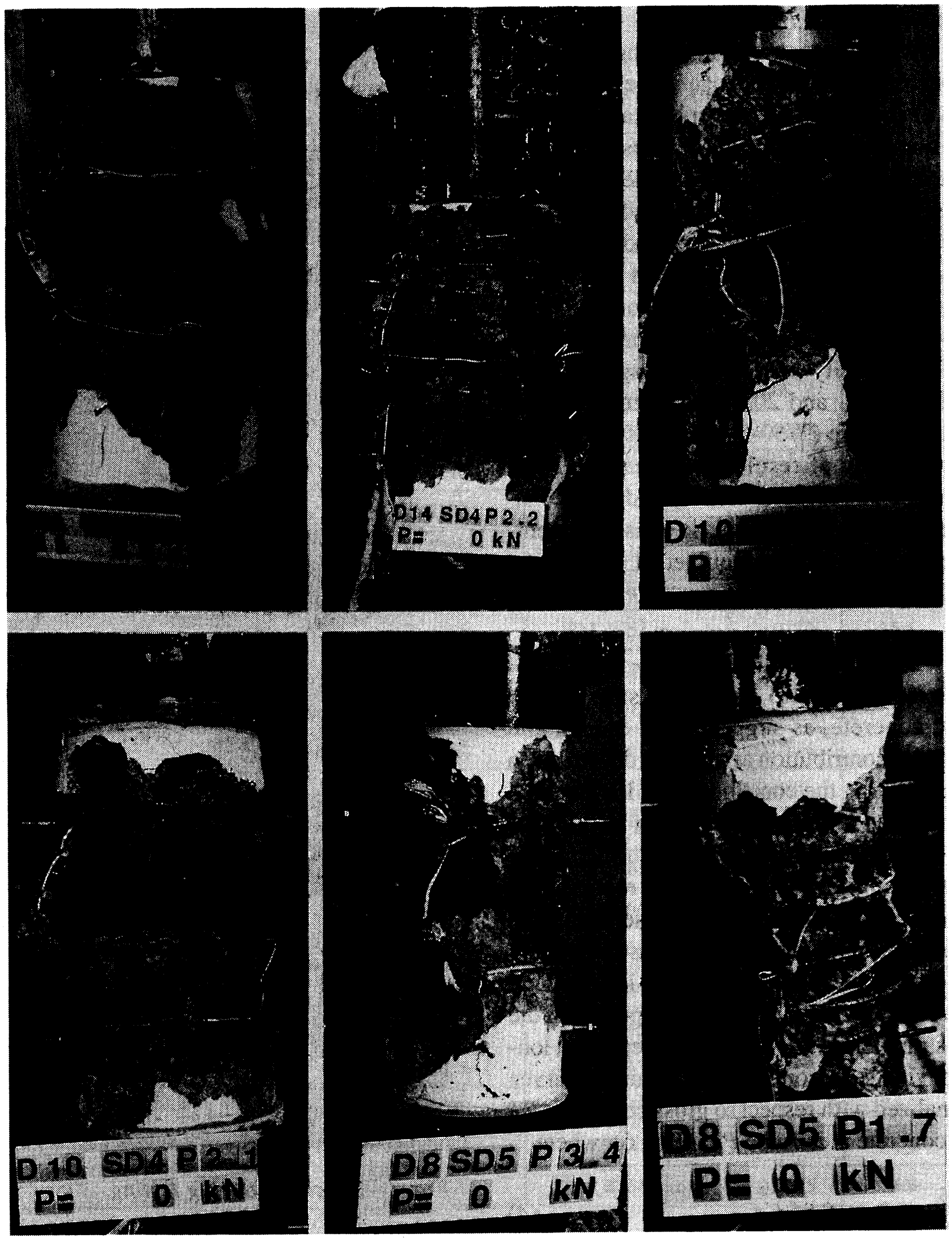

Fig. 4—Appearance of test zones of selected specimens at the end of tests

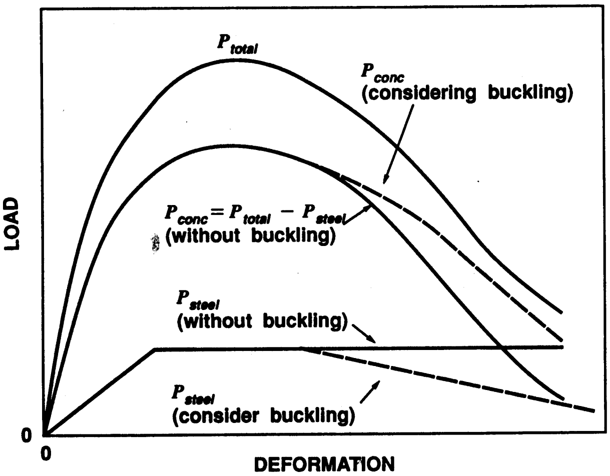

Fig. 5—Calculation of concrete contribution

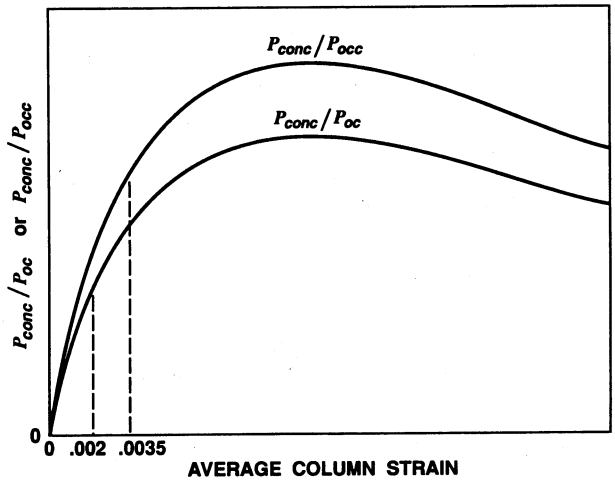

Fig. 6—Nondimensional concrete contribution curves

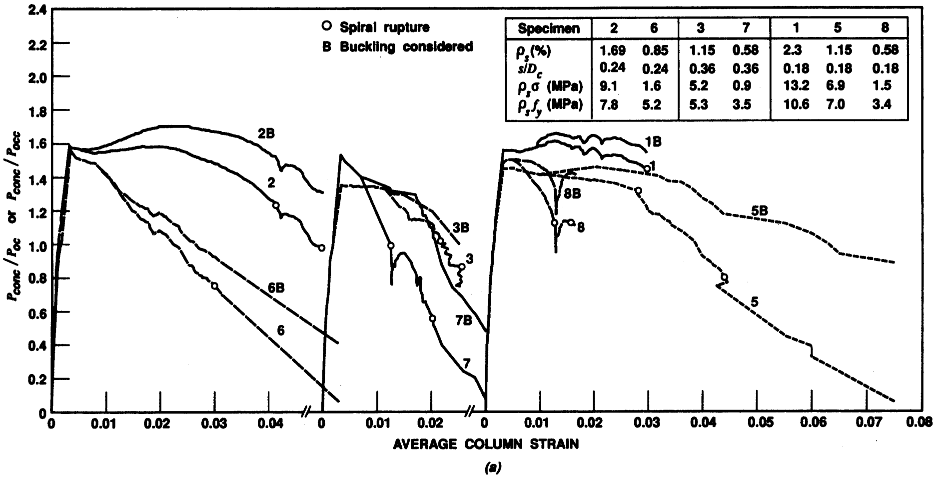

Fig. 7(a)—Effect of amount of lateral steel

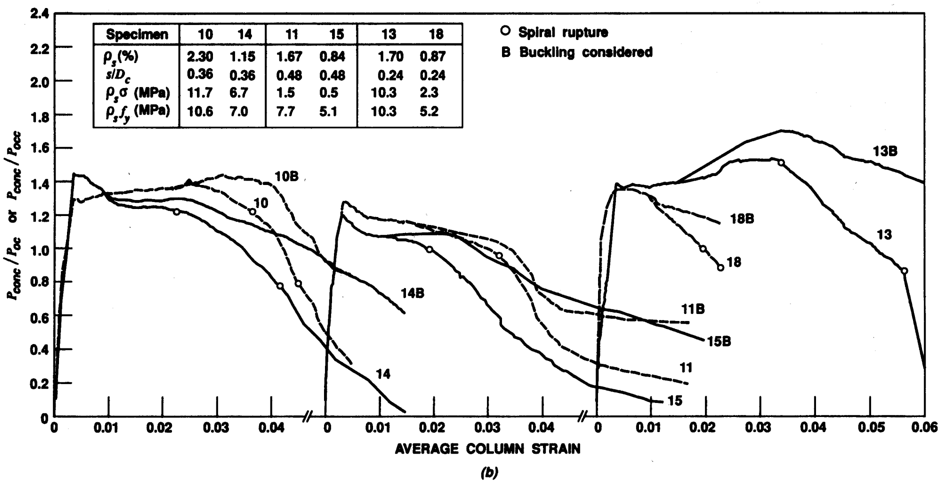

Fig. 7(b)—Effect of amount of lateral steel

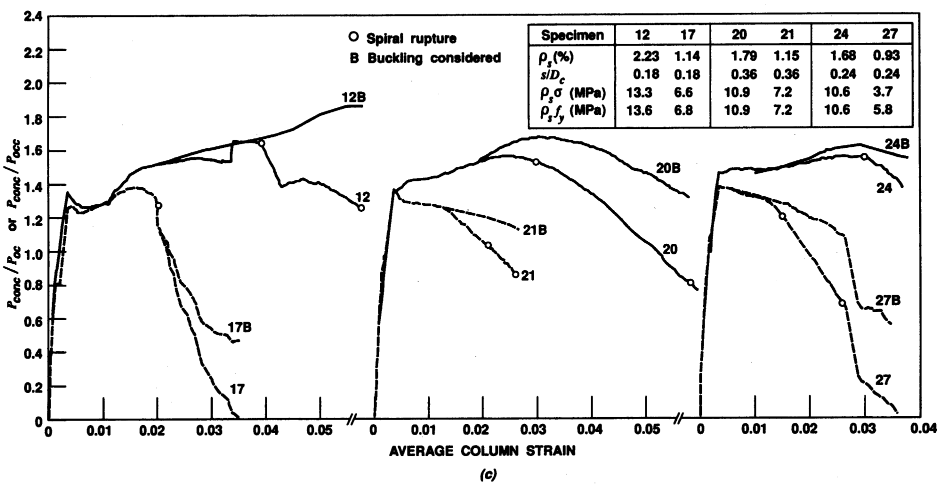

Fig. 7(c)—Effect of amount of lateral steel

the behavior included rapid to moderate strength degradation beyond the peak. For most of these specimens, $\rho_s\sigma$ values were significantly lower than $\rho_s f_y$. Poor behavior was also observed for columns with $s/D_c \geq 0.24$ and low values of ρ_s ($\approx$ 0.58 percent). For these specimens, spiral stress was much lower than yield strength, causing a very small lateral pressure. The columns with high ρ_s ($\approx$ 1.6 percent and larger) and low s/D_c ($\approx$ 0.24 or lower) displayed large strength enhancement and ductility. The lateral steel was stressed beyond yield, and core concrete and longitudinal steel carried high stresses. At this stage, while under high stresses, the longitudinal steel bars buckled in a rapid fashion, causing the rupture of spiral

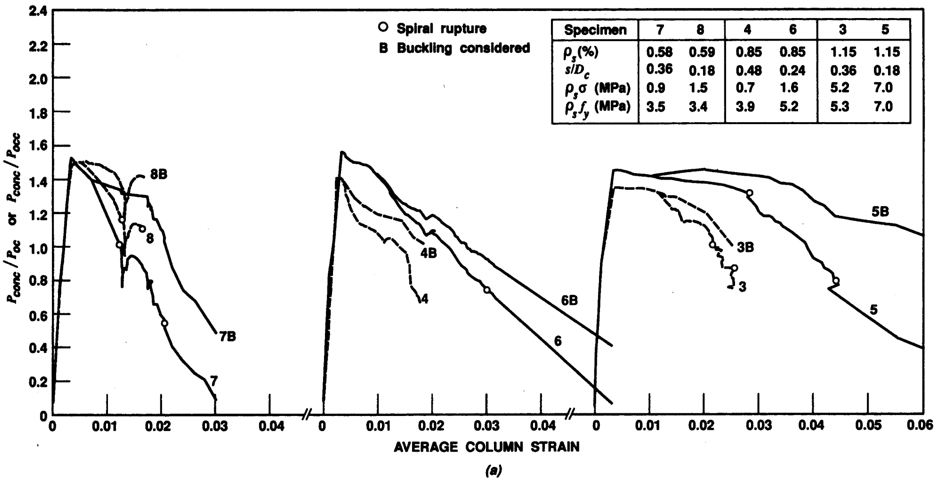

Fig. 8(a)—Effect of spiral spacing

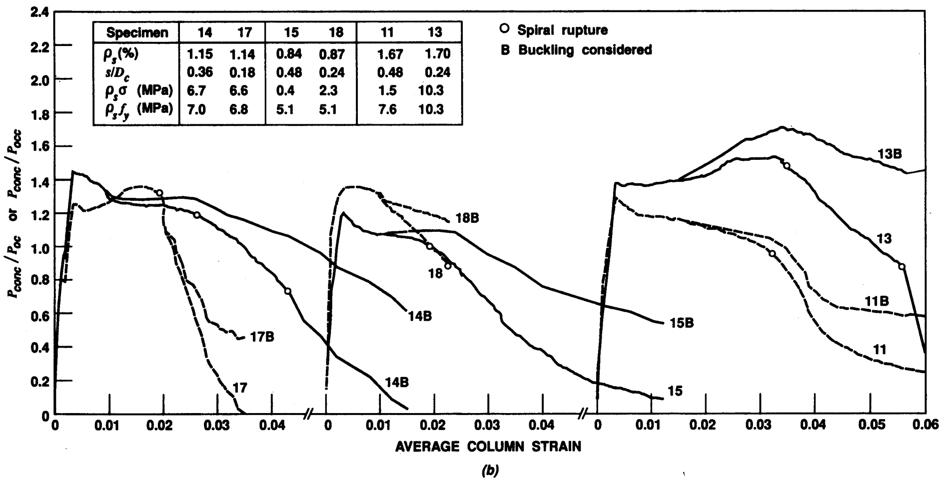

Fig. 8(b)—Effect of spiral spacing

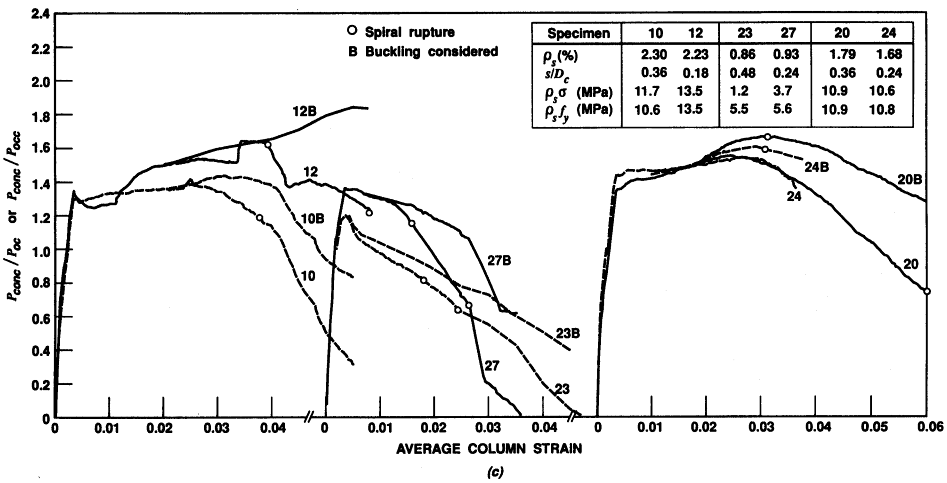

Fig. 8(c)—Effect of spiral spacing

steel and sudden failure of the column. The longitudinal strain at which this phenomenon occurred was fairly large, but the failure lacks a stable descending part of the behavior curve. The optimum values of s/D_c and ρ_s appeared to be around 0.24 and 1.2 percent, respectively, for satisfactory behavior of the specimens with stable unloading behavior. For these specimens, spiral stress was equal to or slightly smaller than yield strength.

Effect of lateral steel spacing

The effect of lateral steel spacing is shown in Fig. 8(a) through 8(c). In most cases, the effect of spiral spacing was

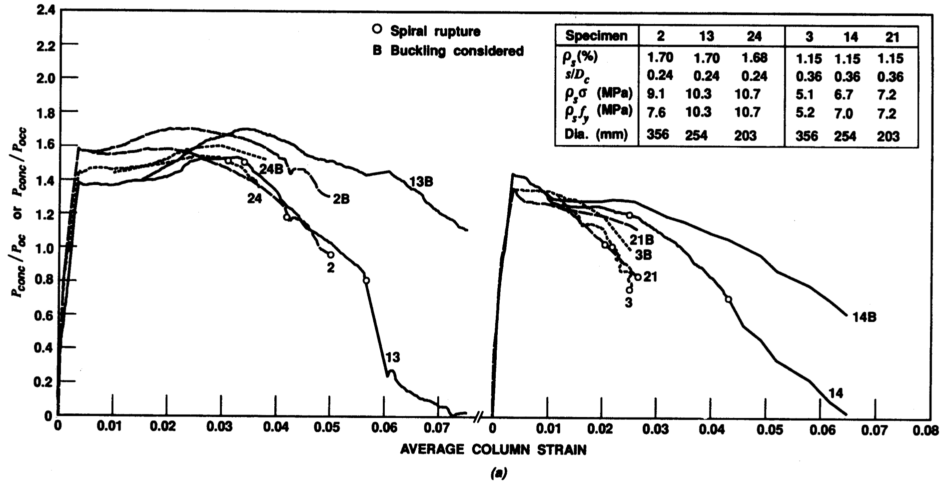

Fig. 9(a)—Effect of specimen size

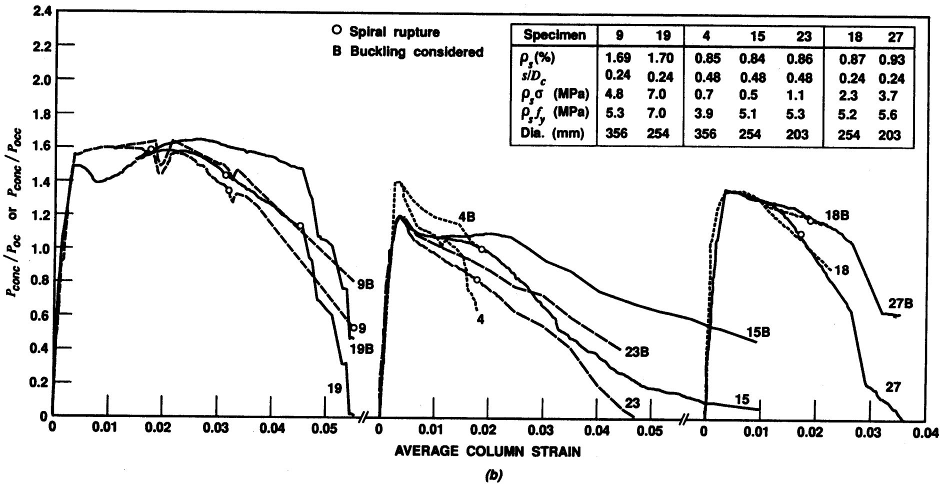

Fig. 9(b)—Effect of specimen size

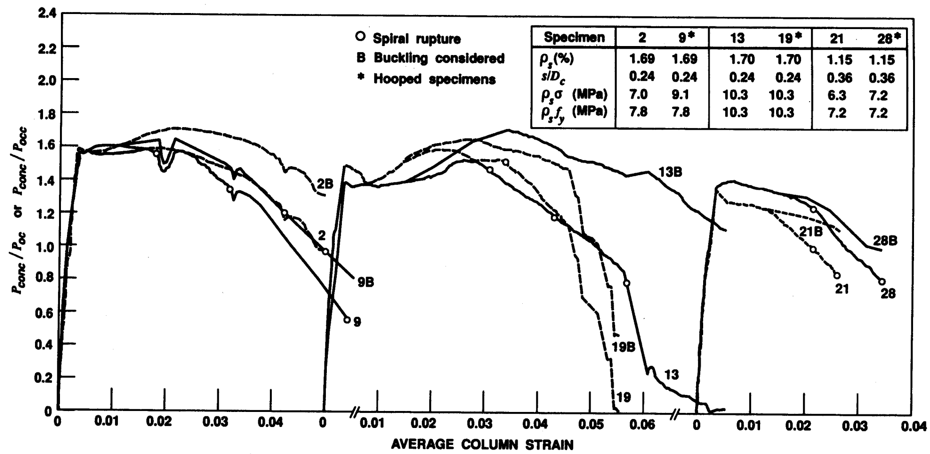

Fig. 10—Comparison of spirally reinforced and hooped specimens

most pronounced for spiral steel ratios between 1.1 and 1.7 percent when the behavior of confined concrete improved with respect to strength and ductility with a reduction of spiral spacing. Reduction in spiral spacing also resulted in an increase in spiral stress at peak concrete force in most cases. The extent of this increase depended on the reduction in spiral spacing. Where the spiral steel stress was already close to yield stress, a reduction in spiral spacing did not significantly affect the $\rho_s\sigma$ value and behavior of confined concrete. For satisfactory behavior of confined concrete, an s/D_c ratio equal to or less than 0.24 appears appropriate when ρ_s is between 1.1 and 1.7 percent.

For high spiral steel ratios (≈ 1.7 percent or higher), the effect of reducing the s/D_c ratio was further improvement of the satisfactory behavior that already existed due to the large amount of spiral steel. For low spiral steel ratios ($\rho_s < 1.0$ per-

Table 2 — Test results

Specimen no.	Spiral provided, in.	Spiral steel ratio, ρ_s	Strength enhancement, $f'_{cc}/0.85f'_c$	Spiral stress σ, MPa	Yield strength of spiral f_{sy}, MPa	Lateral pressure σ_2, MPa	k at maximum concentric force	k_1 at maximum concentric force	k_{max}	Long strain at k_{max}, ε_{kmax}	ε_{cu}	ε_c	ε_b	μ_1	μ_2	μ_3
1	10M @ 2.2	0.0230	1.70	574	452	6.89	3.1	2.3	52.7	0.0025	0.0300	0.0133	0.0095	16.2	*	*
2	10M @ 3.0	0.0169	1.59	536	452	4.72	3.8	2.7	40.3	0.0025	0.0420	0.0179	0.0052	24.3	27.8	21.6
3	10M @ 4.4	0.0115	1.36	452	452	2.71	4.0	2.0	7.3	0.0035	0.0220	0.0036	0.0047	15.0	15.6	12.2
4	10M @ 6.0	0.0085	1.41	77	452	0.31	40.5	22.9	40.5	0.0035	†	0.0030	0.0036	†	9.1	3.7
5	8M @ 2.2	0.0115	1.46	607	607	3.64	3.9	2.4	11.5	0.0030	0.0280	0.0042	0.0100	17.6	24.2	18.7
6	8M @ 3.0	0.0085	1.57	185	607	0.81	21.2	14.6	21.9	0.0030	0.0300	0.0035	0.0085	17.6	14.5	7.2
7	8M @ 4.4	0.0058	1.53	157	607	0.47	39.3	27.8	39.3	0.0025	0.0130	0.0035	0.0075	7.9	7.9	5.1
8	D4 @ 2.2	0.0059	1.51	257	593	0.77	20.1	13.1	20.4	0.0035	0.0140	0.0048	0.0054	8.5	*	6.8
9	10M @ 3.0	0.0169	1.61	415	452	3.65	3.4	2.0	42.9	0.0030	0.0190	0.0085	0.0101	10.9	23.4	18.1
10	10M @ 3.1	0.0230	1.42	509	452	6.05	2.1	1.2	7.4	0.0035	0.0380	0.0251	0.0180	24.5	27.1	23.9
11	10M @ 4.3	0.0167	1.29	90	452	0.79	11.2	4.5	11.2	0.0025	0.0330	0.0035	0.0110	23.4	21.2	16.1
12	8M @ 1.6	0.0223	1.65	607	607	7.16	2.7	2.0	8.6	0.0025	0.0400	0.0343	0.0210	24.5	*	22.5
13	8M @ 2.1	0.0170	1.54	607	607	5.49	3.0	2.0	8.2	0.0035	0.0350	0.0328	0.0150	21.0	30.5	24.2

*Sprial did not rupture.
†Stress did not drop to this level on descending branch of curve.

Table 2 (continued) — Test results

Specimen no.	Spiral provided, in.	Spiral steel ratio, ρ_s	Strength enhancement, $f'_{cc}/0.85f'_c$	Spiral stress σ, MPa	Yield strength of spiral f_{sy}, MPa	Lateral pressure σ_2, MPa	k at maximum concentric force	k_1 at maximum concentric force	k_{max}	Long strain at k_{max}, ε_{kmax}	ε_{cu}	ε_c	ε_b	μ_1	μ_2	μ_3
14	8M @ 3.1	0.0115	1.45	580	607	3.54	3.9	2.4	3.9	0.0035	0.0240	0.0035	0.0087	15.2	22.0	14.2
15	8M @ 4.3	0.0084	1.21	53	607	0.23	26.8	3.7	26.8	0.0035	0.0180	0.0035	0.0083	13.9	14.9	14.3
17	D4 @ 1.6	0.0114	1.37	575	593	3.32	3.3	1.7	16.1	0.0035	0.0220	0.0156	0.0140	14.8	14.6	13.6
18	D4 @ 2.1	0.0087	1.36	262	593	1.15	9.3	4.8	9.3	0.0035	0.0180	0.0055	0.0120	12.2	13.1	9.8
19	8M @ 2.1	0.0170	1.59	607	607	5.50	3.3	2.3	16.1	0.0030	0.0320	0.0210	0.0140	18.5	26.0	20.8
20	8M @ 2.5	0.0179	1.55	607	607	5.77	2.8	1.9	6.3	0.0035	0.0300	0.0235	0.0135	17.8	30.5	21.6
21	D5 @ 2.5	0.0115	1.36	625	629	3.65	3.0	1.5	3.0	0.0035	0.0200	0.0036	0.0090	13.5	14.5	10.9
22	D5 @ 2.5	0.0115	1.31	630	629	3.63	2.5	0.8	2.5	0.0035	0.0190	0.0036	0.0075	13.5	10.6	9.1
23	D5 @ 3.4	0.0084	1.21	477	629	2.05	3.0	0.4	3.0	0.0030	0.0180	0.0037	0.0055	13.7	6.6	5.9
24	D5 @ 1.7	0.0168	1.55	660	629	5.61	2.9	2.0	8.4	0.0035	0.0300	0.0254	0.0110	17.9	*	*
25	D5 @ 1.7	0.0168	1.51	650	629	5.53	2.8	1.8	4.3	0.0035	0.0300	0.0188	0.0090	18.3	24.1	18.9
26	D5 @ 1.7	0.0168	1.55	660	629	5.61	2.9	2.0	5.4	0.0035	0.0410	0.0284	0.0105	24.4	*	*
27	3/16 @ 1.7	0.0093	1.37	400	605	2.52	4.3	2.2	4.6	0.0035	0.0160	0.0034	0.0088	10.7	12.8	10.5
28	D5 @ 2.5	0.0115	1.42	629	629	3.65	3.4	2.0	3.4	0.0067	0.0210	0.0067	0.0130	13.6	18.1	14.9

*Stress did not drop to this level on descending branch of curve.

cent), the resulting poor behavior could not be improved beyond a certain point by reducing s/D_c in the range of 0.48 – 0.24, and spiral stress remained below yield. For very small spiral steel ratios (ρ_s < 0.8 percent), spiral stress was much lower than yield strength, and the increase in $\rho_s\sigma$ value due to reduced spacing was not sufficient to significantly affect the concrete behavior. Although no specimens with spiral steel ratios higher than 2.3 percent were tested, it is postulated that the effect of changing the spiral spacing within practical ranges would diminish with an increase in ρ_s value. Behavior of Specimen 17 compared with that of Specimen 14 in Fig. 8(b) requires some explanation. Reduction of spiral spacing resulted in an improvement of confinement in Specimen 17 and higher concrete stress after an initial drop due to cover spalling. The instability of the longitudinal bars under high stresses after spalling of the cover concrete, as explained earlier, caused a more rapid failure in the specimen with smaller spiral spacing.

Effect of specimen size

In this experimental program, 10- and 8-in. (254- and 203-mm) columns were designed as scale models of 14-in. (356-mm) columns to evaluate the effect of specimen size on their behavior [Fig. 9(a) and 9(b)]. From the observations during testing and from the fact that, in several cases, higher loads were resisted by larger columns at small strains, it appeared that the participation of cover concrete in larger specimens might have been better than that in smaller specimens. The difference is more pronounced between 14- and 10-in. (356- and 254-mm) columns than that between 10- and 8-in. (254- and 203-mm) columns. Another explanation of higher strength of larger columns at small strain may be the unavoidable eccentricity of load that will have a more severe effect on smaller columns.

From the limited test data, it can be stated that similar columns of different sizes behave in a reasonably similar manner if all the parameters, such as spiral spacing, spiral volume, etc., are scaled appropriately. The difference between the behavior of columns of various sizes can be attributed to the experimental scatter and the differences in material properties, particularly the stress-strain curves of spiral steel. The difference between testing machines used for different sized specimens may also have an effect on the descending parts of the specimen behavior.

Performance of hoops as confinement reinforcement

One important finding of the experimental program was the satisfactory performance of circular hoops as confinement reinforcement (Fig. 10). The hoops behaved as well as the spiral reinforcement. Spiral and hoop stresses at maximum concrete stress were reasonably close in almost all the comparable specimens.

In one comparison, the behavior of hooped concrete was somewhat superior to that of spirally reinforced concrete. Note that use of hoops might be preferable due to the fact that each hoop behaves independently and rupture of one single hoop would not affect the confinement provided by the remaining hoops, although other hoops may also be close to rupture. On the other hand, rupture of spiral steel at one location would cause relaxation of lateral confining stress on the concrete core wherever cover concrete has been spalled off.

Comments on ACI Building Code requirements

It was observed that columns that did not satisfy ACI code[4] requirements for the amount of spiral steel ($\rho_s < 1.15$ percent), including the columns in which small spiral pitch was used, did not show much strength enhancement after the first peak and displayed rapid or moderate strength degradation (Specimens 4, 6, 7, 8, 15, 18, 23, and 27). On the other hand, significant strength and ductility enhancements were observed in columns containing ρ_s exceeding the ACI requirement, even when the spiral pitch requirement was violated by about 50 percent (Specimens 3, 10, 11, 14, and 20), considering a 14-in. (356-mm) column to be the prototype with 80-mm (3-in.) spacing limit. The limit of 80 mm (3 in.) on spiral spacing appears to be unnecessarily restrictive, particularly for large columns. A more rational limit on spiral spacing should be in terms of the s/D_c ratio for confinement and s/d_b ratio for longitudinal bar buckling. Most columns containing the code-recommended confining steel and having s/D_c ratios less than or equal to 0.24 behaved in a very ductile manner. The ratio s/d_b in these columns was in the range of 2.70 to 3.0.

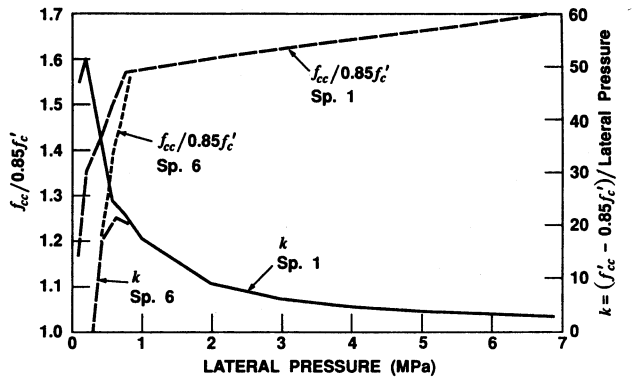

Fig. 11—Strength enhancement and confinement effectiveness coefficient

The maximum limit on f_y in Eq. (2) is 400 MPa (60,000 psi nominal). Several studies on confinement showed diminishing benefit of higher lateral pressure.[2,3] Mander, Priestley, and Park[7] stated that the allowance of a reduction in the required amount of spiral steel for higher yield strength would result in a premature fracture of the spiral and subsequent loss of confinement. A similar phenomenon was observed in the present research for Specimens 6, 15, 18, 23, and 27, which contained the amount of spiral steel calculated according to Eq. (2) and (3), using the actual yield strength of the spiral steel that was well above 400 MPa. The small amount of spiral steel in these specimens based on the actual spiral yield strength resulted in an ineffective confinement, and the spiral stresses corresponding to the maximum concrete force were much less than the yield stress (Table 2). The spiral stresses corresponding to the maximum column forces in Specimens 6, 15, 18, 23, and 27 were 185, 53, 184, 136, and 143 MPa, respectively. Hence, the concept of a limit on the specified yield strength of spiral steel[4] appears justified. However, a more rational approach is needed to assure development of high spiral stress.

Confinement effectiveness

The spiral steel stresses were calculated corresponding to the maximum confined concrete stress and an array of selected longitudinal strain values, including the range between the start and the completion of cover spalling (0.002 and 0.0035, respectively). Lateral stress σ_2 on concrete was calculated at each of these points from the thin-tube analogy using the measured spiral strain values.

The confinement effectiveness coefficient k corresponding to the considered strain values and the peak concrete stress were calculated as follows

$$k = \frac{\textit{confined concrete stress} - 0.85 f_c'}{\sigma_2} \quad (6)$$

The results are summarized in Table 2 where the coefficient k_1, calculated by omitting the factor 0.85 in Eq. (6), is also given. The graphs for the k-versus-lateral pressure σ_2 relation for all specimens are available elsewhere.[8] Two typical sets of curves are shown in Fig. 11.

It was found that the confinement effectiveness coefficient k reached a maximum value before the cover was completely spalled off. For specimens (1, 2, 5, 9, 10, 12, 13, 17, 19, 20, 24, 25, and 26) that showed strength enhancement or strength conservation accompanied by large ductility, the relation between the confinement effectiveness coefficient k and lateral pressure σ_2 had an ascending and strongly pronounced descending branch. The value of k corresponding to the maximum concrete stress for these specimens varied between 2.1 and 4.0, and spiral steel strains at this point were around or above the yield strain. Hence, the peak stress of the confined concrete for these specimens corresponded to high lateral pressure (3.54 to 6.89 MPa). In the case of certain specimens (3, 4, 6, 7, 8, 11, 14, 15, 18, 21, 22, 23, 27, and 28) displaying low ductility, the relation between the confinement effectiveness coefficient k and lateral pressure may or may not have a descending branch, depending on the relative amount of spiral steel ratio provided. If the descending branch exists, it is not very pronounced, and the plot terminates soon after the maximum k value. For these specimens, k values corresponding to the peak concrete stress were between 2.5 and 40.5, and the lateral steel strains in almost all the specimens were less than the yield strain. The higher k values occurred when the lateral stress was very low and the cover concrete still resisted some load. In addition, the strength of unconfined concrete in the column may be somewhat higher than 0.85 f_c' assumed in the calculations. The only dependable k values are those that occur when the cover concrete is completely spalled off. These were observed only in well-confined columns.

Concrete deformability

Ductility ratio μ_1 was found by dividing the axial strain ε_{cu}, at which the first spiral rupture occurred, by axial strain ε_{c1}, which was the strain corresponding to the maximum confined concrete stress on the initial tangent to the concrete stress-strain curve obtained from a standard cylinder. ε_{c1} values ranged between 0.00130 and 0.00185. ε_{cu} values for specimens having lateral steel ratios around or above 1.1 percent were between 0.019 and 0.041, and for other specimens ε_{cu} values ranged between 0.013 and 0.019. ε_{cu} and μ_1 values for all the specimens are given in Table 2, along with the ductility factors μ_2 and μ_3 corresponding to strains ε_{c2} and ε_{c3}, respectively. Strains ε_{c2} and ε_{c3} correspond to 0.85 f_c' and 0.85 f_{cc}', respectively, on the descending parts of the stress-strain curves, and are believed to be more meaningful than ε_{cu}, since they represent the deformability of concrete for a certain strength maintenance. In Table 2, strain ε_c corresponding to the maximum concrete stress and strain ε_b at which buckling was first observed are also given. For well-confined columns, buckling of bars was first observed at very large strains. ε_b values of about 0.01 and larger were quite common. Note that in all the specimens, the most damaged region was within the gage length, which was equal to the specimen diameter plus 50 mm (2 in.) (see Fig. 4).

SUMMARY AND CONCLUSIONS

Research based on 27 circular columns reinforced with spiral or hoop steel and longitudinal bars, and tested under concentric compression is summarized in the following.

Strength and ductility of confined concrete increased with an increase in the amount of lateral steel, the strength enhancement being much less sensitive than ductility.

For specimens containing a code-required amount of spiral steel ($\rho_s \simeq 1.1$ percent), a reduction in s/D_c ratio results in a significant improvement of concrete behavior, particularly ductility. For lower ρ_s values, a change in s/D_c ratio did not change concrete behavior radically, since the improvements in concrete properties due to confinement were minimal. For large ρ_s values ($\rho_s \geq 1.7$ percent), the confinement provided by the spiral steel is very effective and a change in the s/D_c ratio in the range tested here did not affect concrete behavior significantly. In some cases, a large amount of closely spaced spiral steel may result in a lack of stable descending branch of the column behavior curve due to the longitudinal bars' instability. Columns containing code-required ρ_s values and S/D_c ratios less than or equal to 0.24 behaved in a very ductile manner. With 50 percent more spiral steel and s/D_c equal to 0.36, column behavior remained comparably ductile. Instead of an absolute limit, the maximum spiral spacing should be determined based on s/D_c ratio for confinement and s/d_b ratio for bar buckling considerations. The 80-mm (3-in.) limit on spacing appears unnecessarily restrictive for large columns. Columns with five longitudinal bars with appropriate confining steel behaved in a ductile manner. Requirement of a minimum of six bars appears unnecessary and difficult to meet in small columns.

Columns with similar ρ_s and s/D_c ratios behaved similarly, irrespective of their sizes. Circular hoops were found to be as efficient in confining concrete as spirals in three different sizes of columns.

For well-confined columns, the spiral steel yielded when concrete carried the maximum stress. The increase in concrete strength due to confinement was observed to be between 2.1 and 4.0 times the lateral pressure. In poorly confined columns, the spiral steel did not yield at maximum concrete stress. The ρ_s values in most of these columns were less than 1.0 percent or s/D_c ratios were large (≥ 0.36). Corresponding to the first spiral rupture and 0.85 f_{cc}' beyond peak, the concrete compressive strains ranged between 0.013 and 0.041, and between 0.0057 and 0.040, respectively. Ductility factors as high as 24 were observed in well-confined columns for a 15 percent drop in capacity.

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NOTATION

A_g = gross concrete area of cross section in column
A_c = core concrete area in column measured from outside to outside of lateral steel, mm^2
A_{cc} = area of core concrete measured from centerline of spiral or hoop
A_{co} = gross area of concrete in column
D_c = core diameter
d_b = diameter of longitudinal bar
f_c' = compressive strength of concrete obtained from standard cylinder
f_{cc}' = confined concrete strength
f_y = specified yield strength of spiral or hoop steel

f_2 = lateral pressure applied passively by spiral or hoop steel
k = confinement effectiveness coefficient [Eq. (7)]
k_{max} = maximum value of confinement effectiveness coefficient
k_1 = confinement effectiveness coefficient calculated by using f_c' instead of $0.85 f_c'$ in Eq. (7)
P_{oc} = gross concrete area force [Eq. (4)]
P_{occ} = core concrete area force where core area is measured between centerlines of spiral steel [Eq. (5)]
s = spiral or hoop spacing
ε_b = strain at which longitudinal steel buckling was first observed
ε_c = confined concrete strain at maximum concrete stress
ε_{co} = confined concrete strain at which cover concrete starts to spall off
ε_{cu} = confined concrete strain at which first spiral or hoop steel rupture occurs
ε_{c1} = confined concrete strain corresponding to maximum concrete stress on the initial tangent to stress-strain curve of standard concrete cylinder
ε_{c2} = confined concrete strain corresponding to $0.85 f_c'$
ε_{c3} = confined concrete strain corresponding to $0.85 f_{cc}'$
ρ_s = volumetric ratio of spiral or hoop steel to core measured from out-to-out of lateral steel
σ = spiral or hoop steel stress
σ_2 = lateral pressure on confined concrete
μ_1 = ductility ratio, $\varepsilon_{cu}/\varepsilon_{c1}$
μ_2 = ductility ratio, $\varepsilon_{c2}/\varepsilon_{c1}$
μ_3 = ductility ratio, $\varepsilon_{c3}/\varepsilon_{c1}$

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