Tied Concrete Columns under Axial Load and Flexure

By Shamim A. Sheikh,[1] Member, ASCE, and Ching-Chung Yeh[2]

Abstract: Fifteen 12-in. (305 mm) square and 9-ft (2.74 m) long reinforced concrete columns were tested under flexure to large inelastic deformations while simultaneously subjected to constant axial load. The main purpose of this research was to investigate the behavior of column sections confined by rectilinear ties. Major variables considered in this program included: (1) Distribution of longitudinal and lateral steel, including unsupported longitudinal bars and supplementary cross-ties with 90° hooks; (2) level of axial load (0.46 f'_cA_g to 0.78 f'_cA_g); (3) amount of lateral steel (0.8% to 1.6% of core volume), and (4) spacing of ties (2-1/8 in. to 6-13/16 in. [54–173 mm]). Test results indicate that a larger number of laterally supported longitudinal bars results in better performance of columns. Unsupported longitudinal bars and cross-ties with 90° hooks confine concrete effectively only at small deformations and result in rapid deterioration of column behavior at a later stage, particularly under high axial load levels. The amount of lateral steel and the level of axial load have significant effects on the column behavior.

Introduction

Seismic design of most framed structures is based on the ductility approach, in which code-recommended lateral loads that are significantly less than the elastic response inertia loads are used to design the members for strength ("*Building*" 1983; "Code" 1984; "Code of practice" 1982; "Recommended" 1980; *Standard* 1986). The safety of the structure during a major earthquake then depends on its ability to deform plastically while maintaining near maximum load-carrying capacity. To dissipate seismic energy in the inelastic domain of the structural behavior, plastic hinging of beams is preferable to hinging of columns for safety, practical, and economic considerations. Several building codes attempt to achieve this by limiting the ratio of sum of flexural strength of the columns to that of the beams at a beam-column joint or by amplifying the column bending moments determined from elastic analysis. For example, the ACI code ("Building" 1983) requires that the aforementioned ratio be equal to 1.2 to avoid column hinging. Recent research (Paulay 1986), however, indicates that this ratio may have to be in the range of 2 to 2.5 to prevent the plastic hinges from forming in columns if all uncertain features are taken into account. Realizing that the recommended limitations may not be sufficient to avoid plastic hinging in the columns, most codes ("Building" 1983; "Code" 1984; "Code of practice" 1982; "Recommended" 1980) specify lateral reinforcement in the critical regions of columns to confine concrete.

Confinement requirements of various codes ("Building" 1983; "Code" 1984;

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Note. Discussion open until March 1, 1991. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on May 8, 1989. This paper is part of the ***Journal of Structural Engineering***, Vol. 116, No. 10, October, 1990. ©ASCE, ISSN 0733-9445/90/0010-2780/$1.00 + $.15 per page. Paper No. 25170.

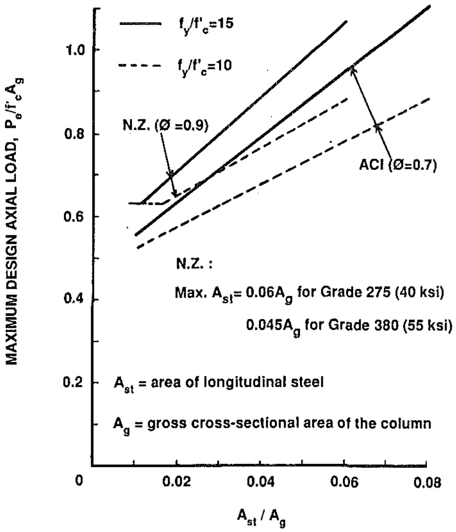


FIG. 1. Maximum Design Axial Loads According to Different Building Codes

"Recommended" 1980) are based on maintaining the axial strength of a column after cover concrete is spalled off. In a study (Sheikh 1982; Sheikh and Uzumeri 1980, 1982) aimed at understanding the mechanism of confinement under concentric compression, an experimental program involving large-size columns was carried out, followed by the development of an analytical model. The model was later extended to include the effect of strain gradient caused by flexure (Sheikh and Yeh 1982). Ideally, the design of confining steel should be such that in the presence of axial force and shear, the column should exhibit ductile flexural behavior. The ductility of a section under flexure is strongly influenced by the level of axial load. Fig. 1 shows the maximum loads on the columns allowed by various codes, which indicates that large axial loads are permitted for columns designed for seismic resistance. Most of the test data available in the literature (Hanson and Rabbat 1984; Park et al. 1982, 1984; Saatcioglu and Ozceb 1989; Soesianawati 1986) have been obtained from columns tested under low axial loads. A summary of the previous work is available elsewhere (Sakai and Sheikh 1989; Yeh and Sheikh 1988). In the experimental program, the results of which are presented here, the column specimens were tested under flexure while simultaneously subjected to large axial loads (Yeh and Sheikh 1988).

Experimental Program

Specimens

The test program consisted of 15 reinforced concrete columns with four different steel configurations (named A, E, D, and F) as shown in Figs. 2 and 3. The center-to-center distance between adjacent bars in any one column was constant. The cross-ties with 90° hooks at one end and 180° hooks at the other end were alternated as required in the ACI code ("Building"

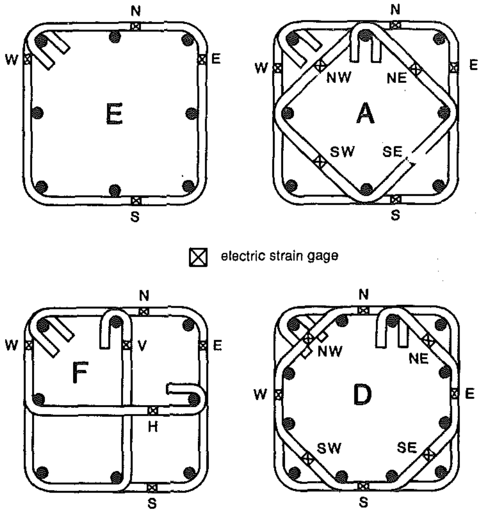


FIG. 2. Locations of Strain Gages in Different Tie Configurations

1983). All the columns were 12 in. (305 mm) square and 9 ft (2.74 m) long. The core size, as measured to the centerline of the perimeter tie, was 10.5 in. (267 mm) square.

The specimens were constructed within the following tolerance of the nominal dimensions: (1) Overall concrete dimensions of the section = ±1/8 in. (3 mm); (2) perimeter tie dimension = ±1/16 in. (1.6 mm); (3) placement of longitudinal steel = ±1/8 in. (3 mm); and (4) spacing of sets of ties = ±1/8 in. (3 mm).

The details of the lateral steel shown in Table 1 refer to the test region, which was the middle 3-ft (0.91-m) length of the specimens. Only deformed bars were used in the test region. To force failure in the well-instrumented test region of the specimen and to prevent premature shear failure outside

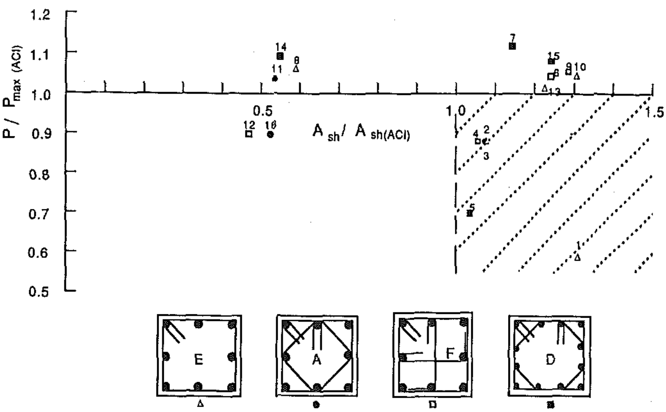


FIG. 3. Relationships between Test Parameters

TABLE 1. Details of Tested Specimens

Specimen (1)	Concrete strength (ksi) (2)	Longitudinal Steel: Number and size (in.) (3)	Longitudinal Steel: ρ (%) (4)	Transverse Steel: Size and spacing (in.) (5)	Transverse Steel: ρ_t (%) (6)	f_{yh} (ksi) (7)	P/f'_cA_g (8)	M_{max} (k-in.) (9)	M_{max}/M_{ACI} (10)	f'_s (ksi) (11)
E-2	4.55	8-#6	2.44	#4 at 4-1/2	1.69	70	0.61	1,498	1.075	57.3
A-3	4.61	8-#6	2.44	#3 at 4-1/4	1.68	71	0.61	1,750	1.231	70.8
F-4	4.67	8-#6	2.44	#3 at 3-3/4	1.68	71	0.60	1,755	1.218	70.8
D-5	4.53	12-#5	2.58	#3 at 4-1/2	1.68	71	0.46	1,810	1.260	70.8
F-6	3.95	8-#6	2.44	#4 at 6-13/16	1.68	70	0.75	1,286	1.151	25.5
D-7	3.80	12-#5	2.58	6 mm at 2-1/8	1.62	68	0.78	1,179	1.218	51.0
E-8	3.76	8-#6	2.44	#3 at 5	0.84	70	0.78	1,143	0.956	35.7
F-9	3.84	8-#6	2.44	#3 at 3-3/4	1.68	71	0.77	1,345	1.247	44.8
E-10	3.81	8-#6	2.44	#3 at 2-1/2	1.68	71	0.77	1,174	1.101	66.7
A-11	4.05	8-#6	2.44	6 mm at 4-1/4	0.77	68	0.74	1,196	0.970	61.6
F-12	4.85	8-#6	2.44	6 mm at 3-1/2	0.82	67	0.60	1,425	0.975	21.3
E-13	3.95	8-#6	2.44	#4 at 4-1/2	1.69	70	0.74	1,132	1.013	32.5
D-14	3.90	12-#5	2.58	6 mm at 4-1/4	0.81	67	0.75	1,031	1.011	23.0
D-15	3.80	12-#5	2.58	#3 at 4-1/2	1.68	71	0.75	1,190	1.171	31.9
A-16	4.92	8-#6	2.44	6 mm at 4-1/4	0.77	81	0.60	1,393	0.945	48.6

1 in. = 25.4 mm; 1 kip = 4.45 kn; 1 ksi = 6.9 MPa.

the test region, lateral reinforcement heavier than that used in the test region was provided in the end regions. This was achieved by reducing tie spacing or using larger-diameter ties and using configuration A in the end regions of specimens where tests regions were of configurations E and F. Minimum anchorage of ties conformed to the ACI code ("Building" 1983). An extension length of at least 8 d_b (d_b = diameter of tie bar) was used for 90° hooks, instead of 6 d_b, as recommended by the code. In the case of configurations A and D, 135° bends and at least 10 d_b extension lengths were used. In many circumstances, 2-1/2 in. (63.5 mm) minimum extension length controlled.

Fig. 3 shows the relationships between the test parameters and the ACI code ("Building" 1983) requirements for the level of axial load and the amount of lateral steel. The specimens that fall in the shaded area satisfy the code requirements. Considering the uncertainty of the forces during an earthquake and the arbitrary nature of the code limits, several columns were tested under axial forces exceeding the ACI limit by up to 11%.

Concrete

Normal weight concrete with a slump of 4 in. to 6 in. (102 mm to 152 mm) was used for all the specimens. Concrete was mixed in the laboratory with a target strength of 4,000 psi (27.6 MPa). The maximum size of the coarse aggregate was 3/8 in. (10 mm). All the specimens were cast horizontally. Strength of concrete was monitored with the help of compression tests on 6-in. × 12-in. (152-mm × 305-mm) standard cylinders. Strength of concrete on the day of the column test was determined from the strength-versus-age relation developed using the cylinder tests, which included cylinder tests conducted on the particular day.

Longitudinal Steel

Grade 60 (414 MPa) deformed bars of sizes No. 5 and No. 6 (16 mm and 19 mm) were used to provide longitudinal steel contents of between 2.44% and 2.58% of the gross cross-sectional area of the column. The tensile stress-strain curves of both sizes of steel bars shown in Fig. 4 are the average of at least three test results. Only those parts of the curves that were used for the analytical work for most columns are shown. Important properties of steel are also listed on the figure.

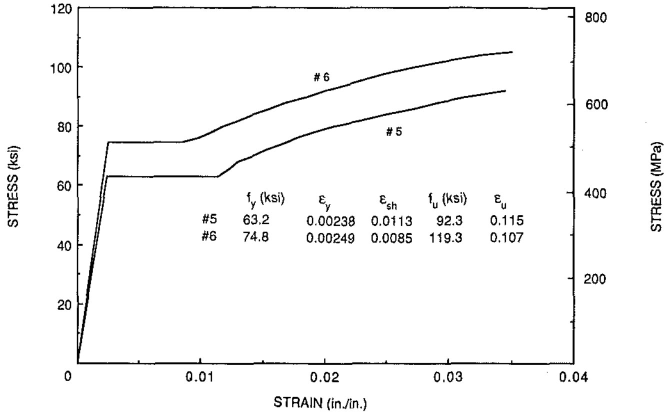
Tie Steel

Lateral reinforcement consisted of deformed No. 4, No. 3 (13 mm and 10 mm), and 6 mm (metric) bars and plain 6 mm bars of grade 60 steel. Tensile stress-strain curves for tie steel are shown in Fig. 5. Whereas No. 3 and No. 4 steel bars showed defined yield plateaus, 6 mm bars lacked a well-defined yield point. The yield point for the 6 mm steel was assumed to be the stress corresponding to an offset strain of 0.2%. Based on the least-squares method, nonlinear portions of the curves were represented by cubic equations.

Instrumentation

The test regions of all the specimens were instrumented to measure deformations in longitudinal steel, ties, and concrete. Specimens were tested horizontally such that the zone of maximum compressive strain was always on the top. Longitudinal steel strains along the depth of the member on both sides were measured by electrical resistance strain gages. Tie strains were also measured by strain gages that were laid on each leg of each tie in one selected set at the mid-length of the specimen. The locations of strain gages in different steel configurations are shown in Fig. 2. Longitudinal concrete strains in the core were measured by using LVDTs over a gage length of 10 in. (254 mm) at three locations along the depth of the section. These LVDTs were installed on the 5/16-in. (8-mm) diameter rods embedded horizontally through the sections without touching longitudinal steel.

Downward deflection was measured by LVDTs and dial indicators along



	f_y (ksi)	ε_y	ε_{sh}	f_u (ksi)	ε_u
#5	63.2	0.00238	0.0113	92.3	0.115
#6	74.8	0.00249	0.0085	119.3	0.107

FIG. 4. Stress-Strain Curves for Longitudinal Steels Within Effective Range

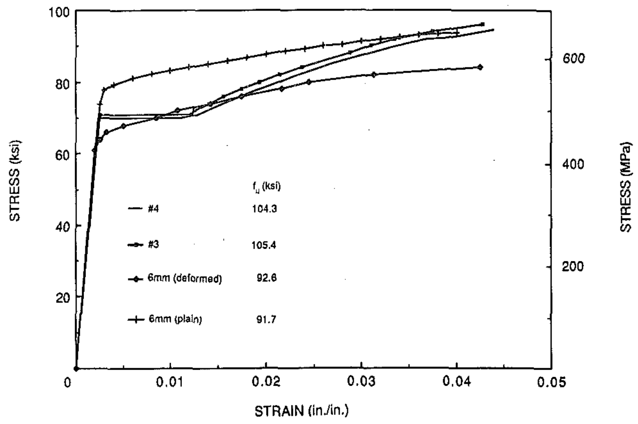


FIG. 5. Stress-Strain Curves for Lateral Steel

the specimen length. Deflection in the lateral direction was also monitored by dial gages along the specimen length to prevent off-center axial loading.

TESTING

All the specimens were tested under flexure to large inelastic deformations while simultaneously subjected to constant axial load. Two equal point loads were applied in the lateral direction at third points to produce a shear-free zone in the test region. Axial load was applied before the specimens were subjected to lateral loads.

Test Setup

Two special hinges were fabricated to be connected to the specimen ends using all threaded rods cast inside the specimen. A thin layer of plaster of paris was used between the hinges and the specimen. Each hinge consisted of two units, one having two high-capacity bearings and the other having three similar bearings. The two units were connected with a 3-in. (76-mm) diameter steel shaft, the central axis of which coincided with the axis of rotation of the hinge. The specimen was supported with the help of these shafts from the top beam of the test frame during the test. The overall test setup is shown in Figs. 6(*a* and *b*).

The axial load was applied using a 1,000-kip (4.5-MN) hydraulic jack and measured with a 1,000-kip (4.5-MN) load cell. To check alignment of the specimen, axial load was increased from 0 to 200 kips (890 kN) in 40-kip (178-kN) intervals. Readings from LVDTs, strain gages, and dial indicators were compared and any adjustments, if necessary, were made to achieve satisfactory alignment. The axial load was then increased to the maximum predetermined value. After a final check and any needed adjustments in the alignment, the specimen was unloaded and all the instruments were reset for the test.

Test Procedure

The axial load was applied in regular increments to reach a predetermined value and remained constant throughout the test. Readings from all the instruments were recorded after each increment with the help of an HP data acquisition system. The lateral load was applied with the help of a 146-kip (650-kN) MTS actuator. The displacement rate of the actuator, controlled by a function generator, was maintained at 0.04–0.05 in./min (1–1.25 mm/min) for the ascending part of the load-deformation curve. Depending on the behavior of the specimen, the loading rate was adjusted beyond the peak. For columns with ductile behavior, the rate was increased by a factor of 2. For columns with relatively brittle behavior, the rate of deformation was reduced to avoid sudden failure and to obtain the complete descending part of the load-deformation curves. Readings were recorded at frequent intervals by holding the deformation constant for a few seconds.

In several cases, tests were continued even after the lateral load dropped to zero and the axial load reduced below the fixed level. The level to which axial load dropped depended on the ductility and toughness of the column.

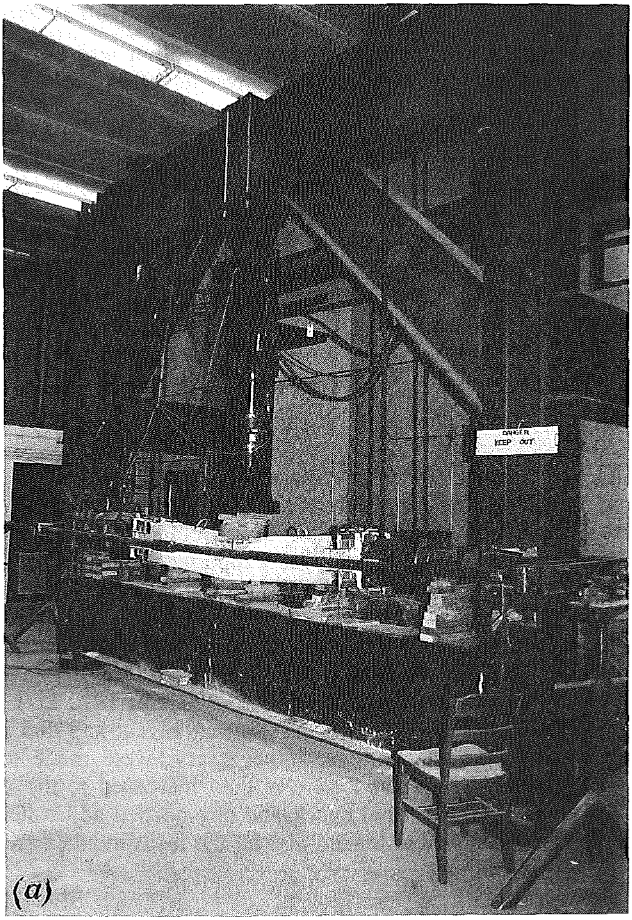


FIG. 6. (*a*) Test Setup and (*b*) Hinge Details

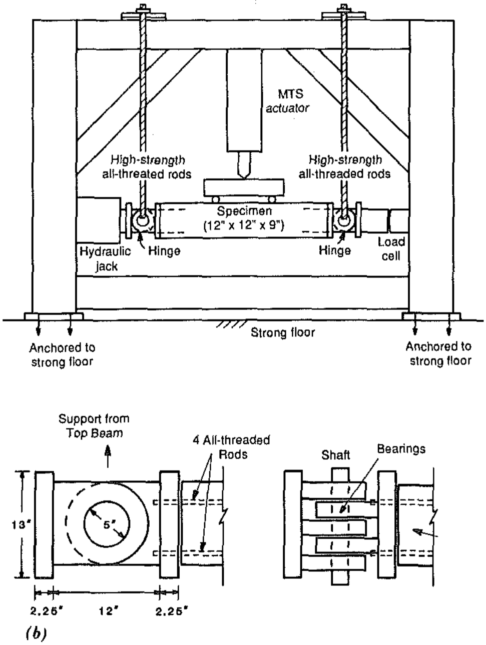


FIG. 6. **(*Continued*)**

An attempt to increase the jack pressure resulted in an increase in column deflection and a reduction in axial load. Although the readings were taken while the instruments remained effective, the main purpose of this part of the tests was to examine the column behavior visually. Most of the tests took approximately 3 hr to complete.

Results

The applied moment at the critical section consisted of two parts: (1) The moment caused by the lateral load, or the primary moment; and (2) the moment caused by the axial load, or the secondary moment. Initially, the primary moment was the major portion of the total moment, but as the lateral load started to drop beyond the peak, the secondary moment became the dominant part. Close to the end of the test, when the lateral load approached zero, almost the entire moment at the critical section was generated by the axial load. Table 1 lists the maximum moments (M_{max}) experienced by all the specimens, the ratio between M_{max} and the theoretical moment capacity (M_{ACI}) based on the unconfined concrete strength (f'_c), and the maximum stress in tie steel corresponding to the maximum moment. The M_{ACI} is based on the actual stress-strain curves for concrete and steel and the extreme fiber concrete strain is assumed to be 0.003.

The maximum strength enhancement beyond the M_{ACI} was obtained in columns that were well confined, with a large amount of lateral steel, and well-distributed, laterally supported longitudinal bars. In several columns that had about 0.8% lateral reinforcement ratios and were tested under a high axial load, the moment capacity was less than M_{ACI}. It appears that compressive strength of concrete in flexure varies with the level of axial load and is less than f'_c under high axial loads. Compressive strength of concrete has been found to depend on the state of stresses and strains acting on an element (Vecchio and Collins 1986). Flexural capacities of the column sections tested during this study were found to be much lower than those reported by Priestley and Park (1987), primarily because of the difference in the specimens. It is believed that heavy stubs used in those specimens played a major part in enhancing the strength of the adjacent sections. In addition, early mobilization of the lateral steel as a result of shear force and cyclic loading may have also contributed to the higher capacity.

In all the specimens, strain in the lateral steel in the compression zone of the critical section (gage N, Fig. 2) increased slowly with an increase in lateral load until crushing of concrete started at the top. This indicated a lack of need for concrete confinement at small deformations. The maximum lateral load approximately coincided with the start of crushing of cover concrete in most specimens. Beyond this point, tie strain increased rapidly, resulting in the yielding of steel. Yielding of tie steel has been marked on all the moment-curvature curves.

The lateral load-versus-deflection curves for 15 specimens are shown in Fig. 7. Before the lateral load reached the maximum value, the axial load was maintained at the predetermined level with relative ease. At larger deformations, the axial load needed to be adjusted more frequently, particularly in columns that were not well confined and were tested under high axial loads. For well-confined sections, the moment resistance increased after concrete cover was lost.

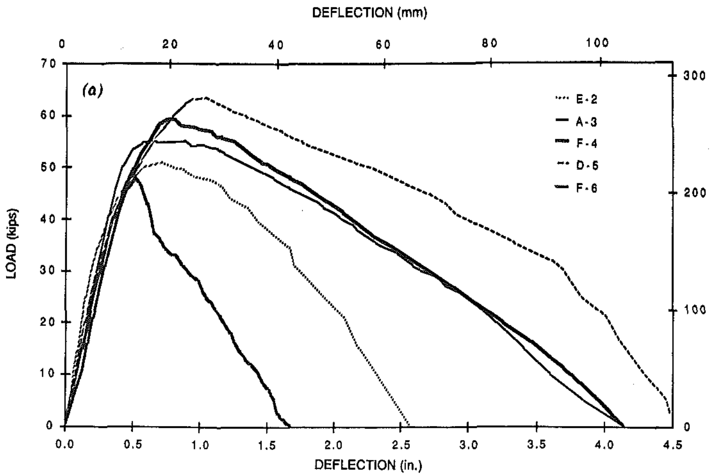


FIG. 7. Lateral Load versus Deflection at Mid-Point: (*a*) Lateral Load versus Deflection at Midpoint for Specimens E-2, A-3, F-4, D-5, and F-6; (*b*) Lateral Load versus Deflection at Midpoint for Specimens D-7, E-8, F-9, E-10, and A-11; (*c*) Lateral Load versus Deflection at Midpoint for Specimens F-12, E-13, D-14, D-15, and A-16

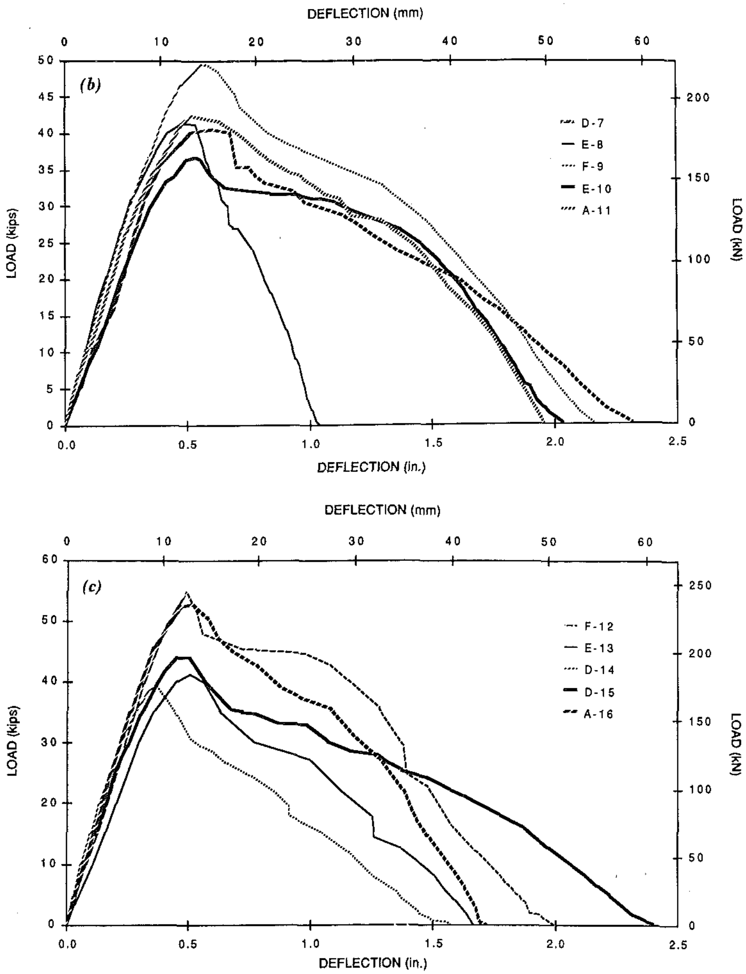


FIG. 7. ***(Continued)***

Effects of Different Variables

The effects of different variables are studied by comparing moment-curvature relations of the sections of those columns in which only one major variable differed significantly. These variables included distribution of longitudinal and lateral steel, including unsupported longitudinal bars and supplementary cross-ties, amount of lateral steel, spacing of ties, and level of axial load. It should be noted that variables other than the one studied in a particular comparison may also have some influence on the behavior of specimens, although every attempt was made to minimize that effect.

Distribution of Steel

Effects of the distribution of longitudinal and lateral steel are shown in Figs. 8–12. Three specimens in Fig. 8 had equal amounts of longitudinal

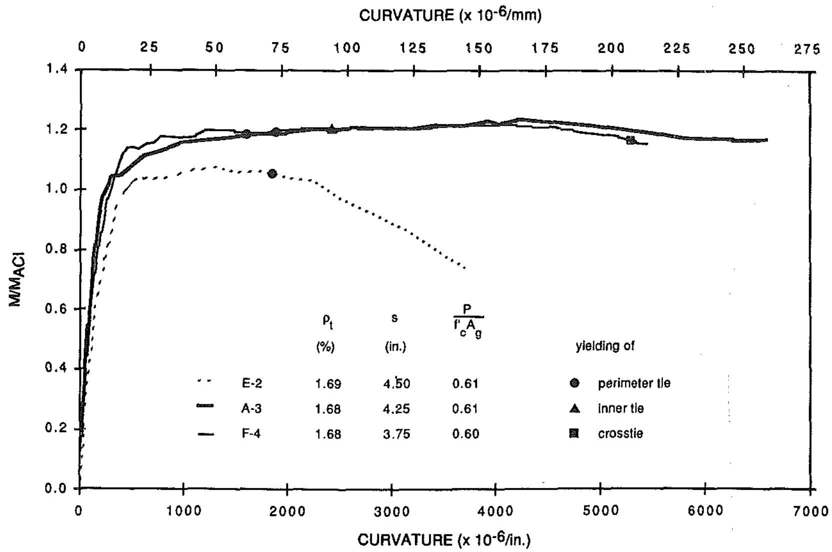


FIG. 8. Effect of Steel Configuration on Section Behavior

and lateral steel and almost equal tie spacing, and were tested under 400 kips (1,780 kN) of axial load. The superior performance of specimens A-3 and F-4 can be attributed to better confinement provided by the inner ties that support the middle longitudinal bars.

In the case of specimen E-2, lateral load dropped rapidly after yielding of tie steel, which accompanied crushing of core concrete, resulting in a rapid

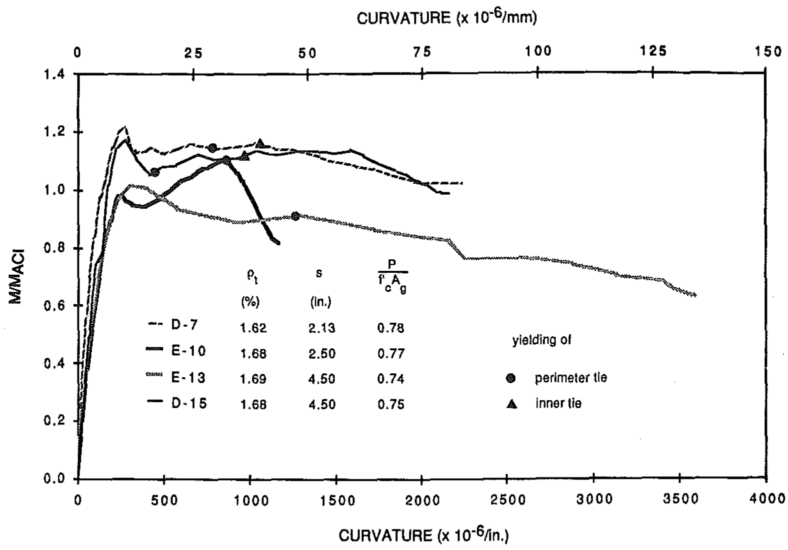


FIG. 9. Effects of Steel Configuration and Tie Spacing

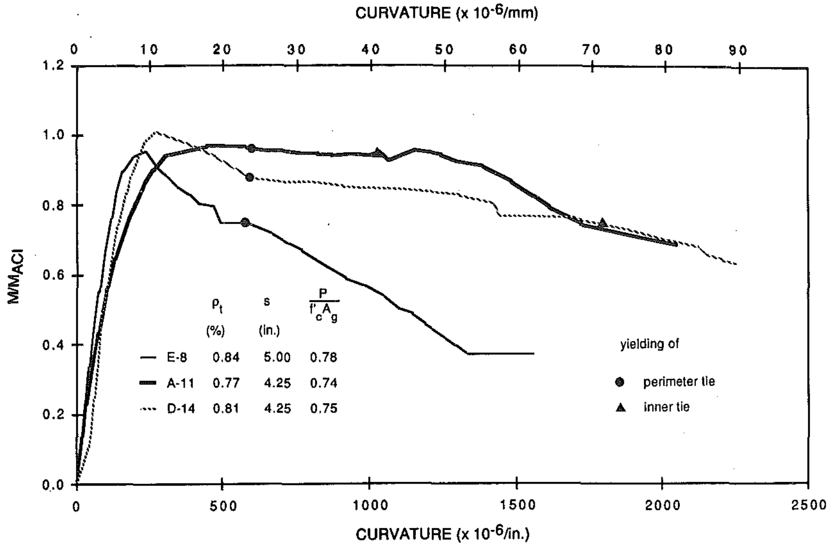


FIG. 10. ***Effect of Steel Configuration***

loss of moment capacity. Buckling of the middle longitudinal bar at the top appeared to have started just before this stage.

Unlike configuration E, the inner ties in configurations A and F provided the necessary restraint to the middle longitudinal bars and improved confinement of concrete, which resulted in a more gradual drop in lateral load beyond the peak. Stress in the inner ties was very low before yielding of

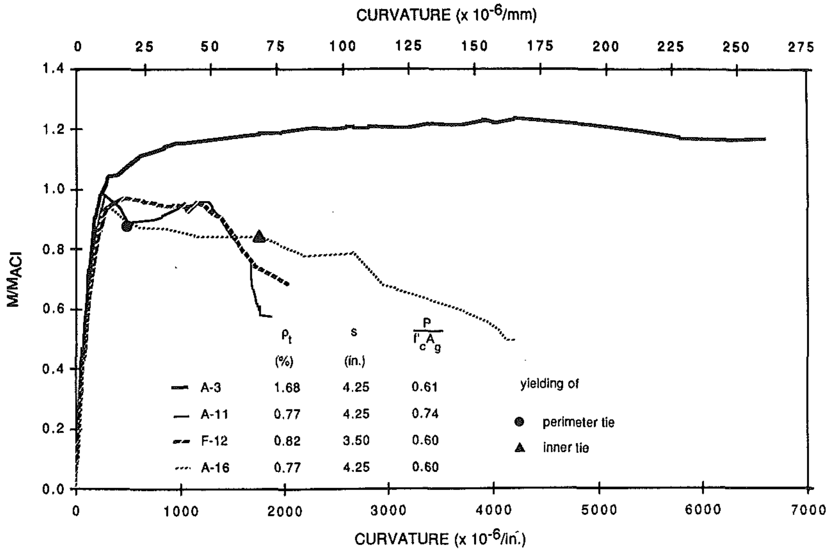


FIG. 11. Effects of Steel Configuration, Axial Load, and Amount of Lateral Steel

the outer ties and increased comparatively rapidly thereafter as the test progressed. This continued lateral confinement appears to be responsible for increasing moment capacity and ductility. After yielding of the tie steel, the moment dropped gradually. The tests were continued until after the lateral

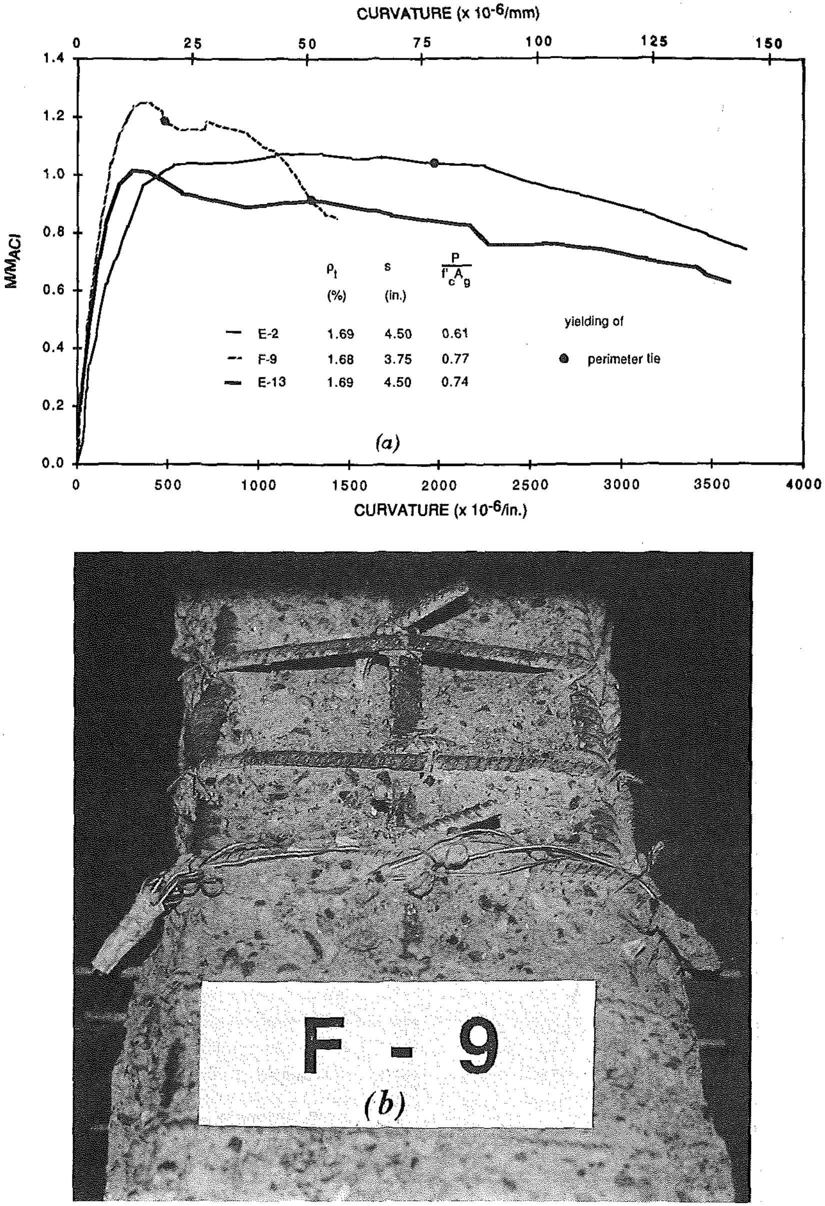


FIG. 12. (*a*) Effects of Steel Configuration and Axial Load; and (*b*) Opening of 90° Hooks in Specimen F-9

load dropped to zero. Specimen A-3 was quite stable and maintained the axial load only slightly less than the original value. At this stage, specimen F-4 was able to carry axial load considerably less than the originally applied force of 400 kips (1,780 kN), but was in a stable condition. The 90° hooks in specimen 4 showed signs of opening, but were able to provide effective confinement. However, it is believed that under reversed cyclic loading configuration A would provide better confinement compared to configuration F.

All the specimens in Fig. 9 were tested under high levels of axial load. Compared to the results shown in Fig. 8, it is obvious that high axial load resulted in a more pronounced drop in the moment when cover concrete started to crush or spall off. A comparison of the behavior of two columns in each set (D-15 versus E-13 and D-7 versus E-10) clearly indicates the superior performance resulting from better distribution of steel. Fig. 10 compares the behavior of sections in which the amount of lateral reinforcement was approximately one-half of that required by Appendix A of the ACI code. The axial load on the columns was about 4 to 10% higher than the maximum allowed by the code. Even under these adverse conditions, an appropriate distribution of steel can produce reasonably ductile behavior. However, it should be noted that the moment capacities of specimens E-8 and A-11 were less than the theoretical capacities based on unconfined strength. Comparison of specimens F-12 and A-16 in Fig. 11 shows a more ductile behavior of the specimen with configuration A. As soon as the stress in the cross-ties approached yield point, the 90° hooks started to open out and resulted in a sudden loss of confinement. It should be noted that the amount of lateral steel in these specimens was only about 50% of that required by the Appendix A of the ACI code and, compared to specimen F-4 (Fig. 8), yielding of cross-ties in specimen F-12 occurred at an earlier stage.

A comparison of specimens E-13 and F-9 in Fig. 12 also shows the unacceptable behavior of 90° hooks. With the amount of lateral steel slightly larger than that required by the code ("Building" 1983), the behavior of the specimen with configuration F is more brittle than that of specimen with configuration E. The axial load level in both specimens was slightly larger than allowed by the code ("Building" 1983). It appears that cross-ties in configuration F effectively confined the concrete at small deformations, resulting in larger enhancement of moment capacity compared to columns of configuration E. But at large deformations, when 90° hooks tend to open out, this enhanced capacity cannot be maintained by perimeter ties alone, thus resulting in a brittle failure. The performance of 90° hooks can be observed in Fig. 12(*b*), which shows the test region of specimen F-9 at the end of the test. From the comparison of the behavior of specimens E-13 and F-9, it appears that the use of cross-ties with 90° hooks may even be harmful, rather than beneficial, when columns are subjected to high axial loads.

Level of Axial Load

Effects of the level of axial load are shown in Figs. 11 to 14. Identical specimens in each of the four pairs were tested under different levels of axial load to evaluate the effects of axial load on the columns with varied steel configurations and other design parameters. Specimens A-11 and A-16 were tested under an axial load of approximately 430 kips (1,913 kN), which resulted in $P/f_c'A_g$ ratios of 0.74 and 0.60, respectively. Reduced ductility as a result of higher axial load level is obvious in specimen A-11. A more

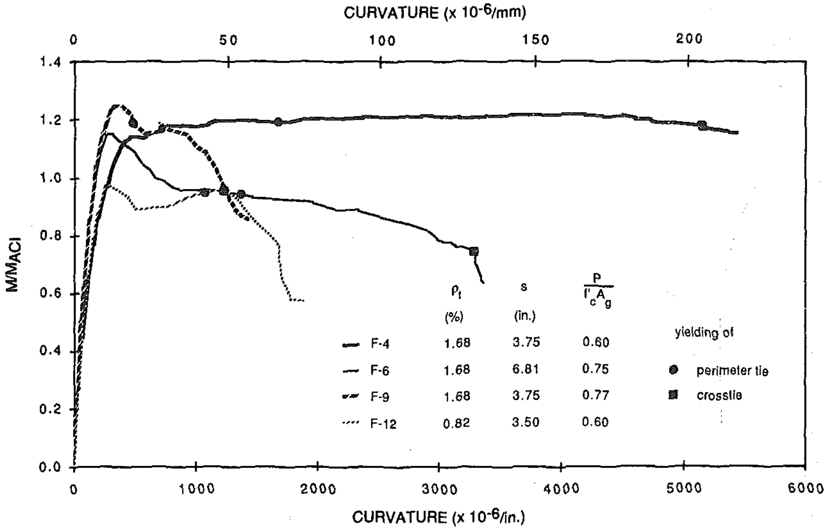


FIG. 13. Effects of Axial Load, Amount of Lateral Steel, and Tie Spacing

sudden drop in moment in the initial stages in specimen A-16 may be due to higher concrete strength. It should be noted that with the lateral steel contents much less than that suggested in the ACI code, a fairly ductiled column behavior can be achieved with appropriate detailing even under moderate to high levels of axial load.

Fig. 12 shows the effect of axial load on the behavior of columns with configuration E (E-2 versus E-13). Higher axial load resulted in slightly lower

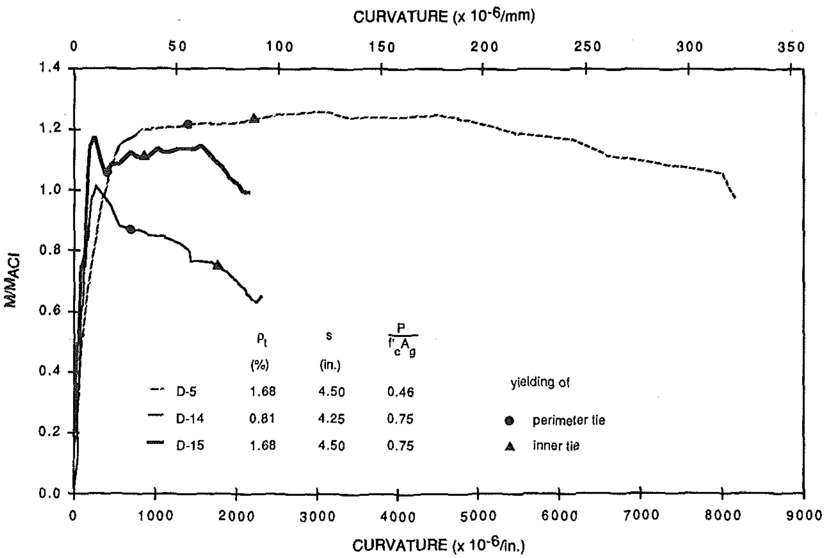


FIG. 14. Effects of Axial Load and Amount of Lateral Steel

moment capacity, but ductility was quite similar in both of the columns. In contrast to this, high axial load had a dramatic effect on the performance of specimens with configuration F [F-4 versus F-9 in Fig. 13(*a*)]. As explained earlier, a rapid drop in moment was caused by the opening of 90° hooks. Under high axial load, the effectiveness of 90° hooks to provide restraint to the longitudinal bars and to confine concrete for ductility is questionable.

Effect of axial load on the behavior of columns with configuration D can be evaluated from a comparison of specimens D-5 and D-15 in Fig. 14. Both ties yielded in both columns, but, under higher axial load, yielding of steel took place after a significant drop in the moment capacity, indicating that crushing of core concrete had started before lateral steel became effective.

Amount of Lateral Reinforcement

The effect of this variable can be evaluated by comparing the behavior of specimens A-3 and A-16 in Fig. 11, specimens F-4 and F-12 in Fig. 13, and specimens D-14 and D-15 in Fig. 14. Whereas one specimen in each pair contained lateral steel approximately equal to 50% of the amount required in Appendix A of the ACI code ("Building" 1983), the lateral steel content in the second column of each pair was approximately 10% more than the code requirement.

The almost identical ascending parts of the curves indicate that ties do not significantly influence the section behavior prior to the crushing of unconfined concrete. It is obvious that with lower lateral steel contents, the confining pressure is not sufficient to maintain the moment capacity of the specimens even when steel stress reached yield point in both perimeter and inner ties. It was also observed in these specimens that right after yielding of inner ties, the core concrete between inner and outer ties could not be maintained intact by the small amount of lateral steel and resulted in a rapid drop of moment capacity. A sudden drop in the moment capacity of specimen F-12 was caused by the opening of 90° hooks. The effect of reducing the amount of lateral steel is less severe in the case of configuration D compared to the other two configurations, despite the fact that specimens D-14 and D-15 were tested under higher levels of axial load.

Amount of lateral steel conforming to the Appendix A requirements of the ACI code ("Building" 1983) can provide effective confinement if laterally supported longitudinal bars are adequately distributed around the core perimeter. The amount of lateral steel can perhaps be reduced if certain steel configurations are used and limited ductility is needed. A certain minimum amount of lateral reinforcement is needed to develop the theoretical moment capacity of a section based on unconfined concrete strength.

Spacing of Ties

Comparisons of specimens D-7 and D-15, and E-10 and E-13 in Fig. 9, and specimens F-6 and F-9 in Fig. 13 show the effect of tie spacing on the moment-curvature behavior of the sections. In general, smaller tie spacing resulted in higher moment capacity. The effect of variation in tie spacing was the least in case of specimens with configuration D. In most cases, concrete cover in specimens with smaller tie spacing started to crush and spall off earlier and the cover was lost completely at a more rapid rate.

In configuration E specimens, reduced tie spacing resulted in better confinement of concrete until the tie was pushed out by the unsupported bars, resulting in a loss of confinement and a rapid drop of moment. In the case of larger tie spacing, this phenomenon was not observed. It appears that yielding of tie steel at a larger peak moment and earlier spalling of cover in the case of smaller tie spacing resulted in premature buckling of longitudinal steel bar and a loss of confinement. The moment level and, hence, the concrete stress in the specimen with larger tie spacing were comparatively lower. A loss of confinement, therefore, did not result in such a sudden drop in the moment. It should be noted that the moment capacity in specimen E-13 was approximately equal to the theoretical moment capacity based on unconfined concrete strength, whereas moment capacity of specimen E-10 was approximately 10% higher.

A similar phenomenon appears to have occurred in specimen F-9. Compared to specimen F-6, closer spacing of ties in specimen F-9 resulted in higher strength of concrete, and, hence, higher moment capacity. With better confinement in place, stress in the cross-ties approached yield point at higher moment level. The 90° hooks could not maintain this high stress level in the ties and opened out, resulting in a brittle failure. In specimen F-6, opening of the 90° hooks was delayed, because larger tie spacing resulted in lower stress in crossties.

In general, reduced tie spacing would result in an increased moment capacity of the section. Ductility would also improve unless anchorage of the lateral reinforcement was lost, which might result in a more brittle behavior with smaller tie spacing.

Moment Capacity and Concrete Strain

Effects of major variables on the moment capacity of sections with different steel configurations are summarized in Fig. 15. A straight line is drawn between two points representing two specimens with only one major variable between them. The moment capacity is nondimensionalized with respect to the theoretical capacity based on unconfined concrete strength (M_{ACI}). The trend clearly indicates the detrimental effects of high axial load on the sec-

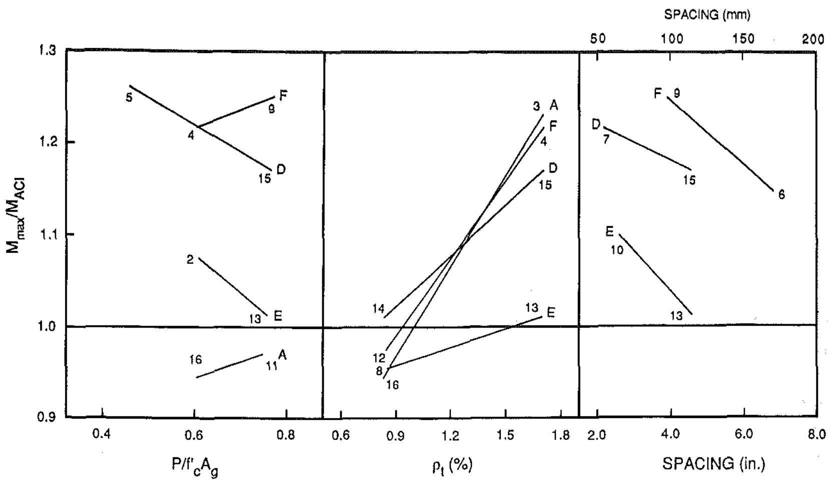


FIG. 15. Effect of Different Variables on Flexural Strength of Section

tions' moment capacity. Beneficial effects of reduced tie spacing and increased lateral steel content are also obvious. All the specimens that showed moment capacities less than M_{ACI} were reinforced with about 0.8% of lateral reinforcement, which is about 50% of that required in Appendix A of the ACI code. The variation in flexural capacity shown in Fig. 15 underlines the need to develop a rational method to calculate the flexural capacity of a section, particularly that of a well-confined section. A conservative estimate of flexural capacity is not always desirable. It can, in fact, be the cause of a brittle shear failure in the capacity design approach.

The deformation capacity of concrete is shown in Fig. 16, where the measured extreme fiber compressive strains at maximum moment and at 90% of the maximum moment on the descending part of the curve are plotted against different variables. Again, a straight line is drawn between two points representing two specimens. The crushing strain of cover concrete in most specimens was observed to be in the range of 0.003–0.0037. The low concrete strains in the specimens under high axial load correspond to the low stress developed in the lateral steel.

With the exception of specimen A-16, the data presented in Fig. 16 show that an increase in axial load and a reduction in the amount of lateral steel resulted in a reduced concrete strain. Corresponding to the maximum moment, concrete strain as high as 0.019 was observed in specimens A-3 and D-5. At 90% of the maximum moment, the maximum strain of 0.034 was observed in specimen A-3. Specimen E-8 displayed the least deformation capacity of concrete. Concrete strain in this specimen ranged between 0.002 and 0.003 when the moment in the specimen reduced from M_{max} to 0.9 M_{max}.

The curvature ductility factor ($\mu_{\emptyset}$) of the secion, defined as the ratio between $\emptyset_2$ and $\emptyset_1$, was computed for all the specimens. The value $\emptyset_1$ is the curvature corresponding to the maximum moment on a straight line joining the origin and a point corresponding to about 65% of the maximum moment on the moment-curvature curve. The curvature $\emptyset_2$ corresponds to 90% or 80% of the maximum moment experienced by the section on the descending

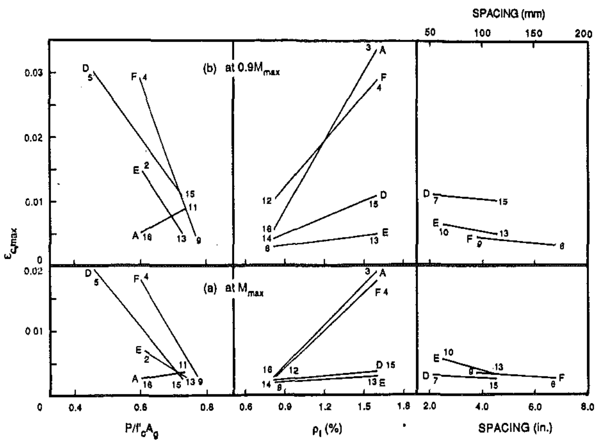


FIG. 16. Effect of Different Variables on Extreme Fiber Concrete Compressive Strain

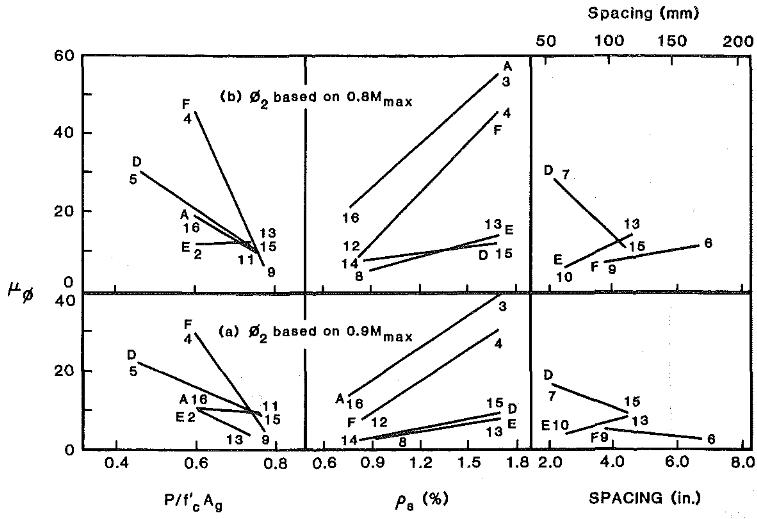


FIG. 17. Effect of Different Variables on Curvature Ductility

part of the M-$\varnothing$ curve. In the case of specimens A-3 and F-4, $\varnothing_2$ values were extrapolated, since the curves did not extend to 0.8 M_{max} before the lateral load dropped to zero.

Fig. 17 shows the dependence of curvature ductility factors on four major variables. Increase in axial load and a decrease in the amount of lateral steel resulted in significant reduction in the ductility of all column sections. The least effect of these variables is on configuration E, because the efficiency of confinement is very low in specimens with only four laterally supported longitudinal bars. Reduced spacing of ties resulted in higher ductility, except in specimens E-10 and F-9. The reasons for lower ductility in these specimens have been explained here. The ductility factors ranged from 3.1 in specimen E-8 to approximately 50 in specimens A-3 and F-4.

Summary and Conclusions

Design of confining steel in the U.S. codes is not related to the expected performance of the columns. In addition, there are several parameters that significantly affect the behavior of confined concrete, but are not considered by the codes. These include distribution of longitudinal and lateral steel, tie spacing, and the level of axial load in the case of combined axial load and flexure. These variables were investigated in this study, which included an experimental program involving 15 12-in. (305 mm) square and 9-ft (2.74 m) long columns. The specimens were subjected to an axial load first and then tested under increasing flexure to large deformations in the absence of shear in the critical region. The following conclusions can be drawn from this study.

As in the case of concentric compression, distribution of longitudinal and lateral steel plays an important role in the behavior of columns under axial load and flexure. A larger number of laterally supported longitudinal bars results in higher flexural strength and ductility. Reduced spacing of ties for the same amount of lateral steel would also result in higher strength and ductility if the anchorage of lateral steel can be assured. Maximum gain in flexural capacity due to confinement was observed to be 26% in the tests conducted. The maximum values of curvature ductility factor and the compressive concrete strain corresponding to the maximum moment were above 50 and 0.019, respectively.

Unsupported longitudinal bars, although effective in confining the concrete at small deformations, tend to buckle and push the ties outward at large deformations, resulting in a brittle behavior caused by a loss of confinement. A similar phenomenon was also observed for bars that were supported by 90° hooks, which opened at large deformations, particularly when the column was subjected to high axial load.

Higher axial load reduces strength and ductility of confined concrete sections very significantly. Several columns in which the amount of lateral reinforcement was about 50% of that required for seismic design did not even reach the theoretical moment capacity for unconfined sections, although the tie steel provided was more than that required for nonseismic design. It appears that the compressive strength of concrete in flexure reduces with an increase in the axial load.

An increase in the amount of lateral steel results in a significant improvement in flexural behavior of a section. Design of confining steel according to the ACI code provided reasonably ductile behavior of columns when axial load was less than $0.6 f'_c A_g$ and the steel was appropriately detailed. It appears that for less severe conditions and limited ductility demand, the code requirements can be relaxed, while for other conditions, the present code may produce unsafe design. The results from this study underscored the need to link the required amount of steel and the use of unsupported bars and 90° hooks to the level of axial load and the expected performance of a column.

Acknowledgments

The research reported here was supported by grants from the National Science Foundation and the Texas Advanced Technology Research Program. The experimental program was carried out at the Structures Laboratory of the Department of Civil Engineering at the University of Houston.

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