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A Performance-Based Approach for the Design of Confining Steel in Tied Columns

by Shamim A. Sheikh and Shafik S. Khoury

A review of the development over the years of the ACI Code provisions for confinement is presented. Based on the available experimental evidence, the current Code requirements for the amount of confinement steel in tied columns are critically evaluated. It was concluded that the behavior of columns designed according to the ACI Code may vary from unacceptably brittle to very ductile. While the amount of Code-required steel can be reduced in many cases, much larger amounts of lateral steel are needed in other cases. A new design procedure is proposed in which the amount of lateral steel required is a function of the column ductility performance. The lateral steel content increases with an increase in the level of axial load, and depends on steel distribution and the extent of lateral restraint provided to the longitudinal bars. For any specific steel configuration, the procedure lends itself to a simple design chart. The proposed method when applied to realistically-sized specimens tested by different investigators yielded excellent agreement with the experimental results.

Keywords: columns (supports); confined concrete; standards; structural design; tied columns.

INTRODUCTION

The need for ductile behavior of various structural components during a major earthquake has been demonstrated repeatedly during several seismic events. Although it is preferable to dissipate seismic energy by post-elastic deformations in beams, column hinging cannot be avoided entirely in most buildings during severe earthquakes. To achieve sufficient ductility in columns, their potential plastic hinge regions should be reinforced with appropriately designed and detailed longitudinal and lateral confining steel. Design provisions for confinement steel in various codes,[1-3] however, do not contain adequate quantitative relationships between the design parameters and column performance. For columns subjected to axial load beyond a certain limit, heavy confining steel is required by the North American Codes irrespective of the level of axial load. In addition, there is no consideration given to the distribution of longitudinal and lateral steel in a column which has been found to significantly affect the confinement mechanism.[4,5] As a result, the design may either be very conservative for columns with well-distributed steel and subjected to low levels of axial load or unsafe for columns in which only four corner bars are effectively supported by tie bends and axial load is large.

HISTORICAL REVIEW OF THE ACI CODE PROVISIONS FOR CONFINEMENT

The basic philosophy of the current ACI Code[1] requirements for confining steel is to maintain the axial load carrying capacity of the column after spalling of the cover concrete. This philosophy is obviously based on strength enhancement due to confinement. Ductility is not given due importance although it is implied that the lateral steel would enhance section and member ductility. The ACI design provisions for confining steel have changed over the past 30 years through six editions of the Code from 1956 through 1995. It is believed that some of the changes made from one edition to another are not suitable to provide sufficient ductility in reinforced concrete columns.

The 1956 and 1963 Codes—Neither of these Codes[6] contained any equations to calculate the amount of confining steel in tied columns. Both Codes required that at least #2 bars (6.4 mm) be used for ties that are spaced apart not more than 16 longitudinal bar diameter, 48 tie diameter, and the least section dimension. No mention was made of ductility, confinement, or plastic hinge. However, the following equation was suggested to calculate the volumetric ratio of circular spiral reinforcement

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$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_{yh}} \tag{1}$$

where A_g = gross area of section, A_c = area of the concrete core measured to the outside diameter of spiral, f_c' = compressive strength of concrete, and f_{yh} = yield strength of lateral steel.

The only significant difference between the two editions of code was that related to steel detailing. While the 1956 Code required all the longitudinal bars to be laterally supported by tie bends, this requirement was considerably relaxed in the 1963 edition (see Fig. 1) in which unsupported middle bars were permitted as long as the clear distance between an unsupported bar and a supported bar did not exceed 6 in. (152 mm). This change was primarily based on experiments in which the ultimate column strength was the only criterion used. Since no attempt was made to evaluate ductility, this relaxation in interior ties appeared plausible. Whereas the change was technically sound for most steel arrangements, allowing perimeter ties only for all situations is not appropriate as it is now well-known that columns with only four corner bars supported by tie bends may fail in a brittle manner.[7-9] The single perimeter tie is not able to support the middle longitudinal bars effectively after cover spalling; these bars would buckle and push the ties outward, thus releasing a considerable amount of confinement. The 1956 Code provided very efficient steel detailing which would have provided excellent confinement with small tie spacing. Since 1963, this provision of the Code has not changed.

The 1971 and 1977 Codes—The special provisions for seismic design were introduced in the 1971 edition of the Code in Appendix A and were retained without any substantial changes in the 1977 Code. The importance of ductility was outlined, and related significant terms were defined. The plastic hinge was defined as the region where ultimate section moment capacity may be developed and maintained while the inelastic deformation is increased significantly. The concept of "Strong Column-Weak Beam" was introduced in an attempt to prevent column hinging. The volumetric ratio of spiral ρ_s was given as in Eq. (1) with the lower limit provided by Eq. (2) that will be applicable to large columns in which A_g/A_c is less than 1.27.

$$\rho_s = 0.12\frac{f_c'}{f_y} \tag{2}$$

The spiral pitch was limited to 3 in. (76 mm) and the maximum center-to-center spacing between ties was 4 in. (102 mm). The minimum cross sectional area of tie (one leg) was specified as

$$A_{tie} = \frac{l_h \rho_s s}{2} \tag{3}$$

where l_h = the maximum unsupported length of the perimeter tie, s = tie spacing, and ρ_s is the larger of the value calculated from Eq. (1) and (2). In the format of the current Code requirements,[1] the amount of tie steel for square columns with only perimeter ties can be written as

$$A_{sh} = 0.45 s h_c\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yh}}, \quad \geq 0.12 s h_c \frac{f_c'}{f_{yh}} \tag{4}$$

where $A_{ch} = A_c$, A_{sh} = the total cross sectional area of rectilinear steel perpendicular to dimension h_c ($A_{sh} = 2A_{tie}$), and $h_c = l_h$. Eq. (4) assumes that for columns with square perimeter hoops only, the efficiency of rectilinear confining steel is 50 percent of that of spirals. For the case of square hoop with one supplementary crosstie in each direction, the implied efficiency of the rectilinear ties is increased to 66 percent of that of spirals. Application of lateral pressure on the concrete core at larger number of points results in better confinement;[4] therefore the Code's assumption was quite rational. The minimum bar size allowed for ties was also increased from #2 (6.4 mm) as specified in the 1963 Code, to #3 (9.5 mm) for longitudinal bars #10 (31.8 mm) or smaller and at least #4 (12.7 mm) for #11 (34.9 mm) or larger longitudinal bars.

The 1983 Code—The maximum tie spacing was changed from 4 in. (102 mm) to the smaller of 4 in. (102 mm) and one-quarter of the minimum section dimension. While the requirements for the amount of spiral reinforcement were

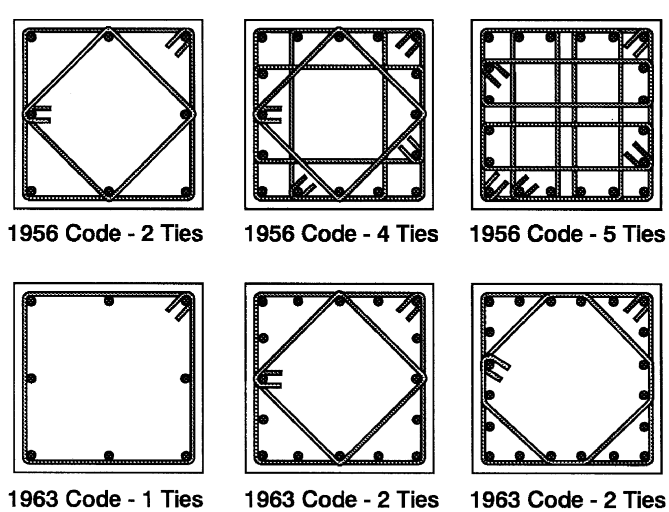


Fig. 1—Reduction of interior ties in 1963 ACI Code

similar to those specified in the previous edition [Eq. (1) and (2)], the total cross sectional area of rectilinear lateral steel (including crossties) was given by

$$A_{sh,c} = 0.3sh_c\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_y}, \qquad (5)$$
$$\geq 0.12sh_c\frac{f_c'}{f_y}$$

It can be seen that this requirement is similar to that given by Eq. (4) except for the numerical coefficient 0.45 which has been reduced to 0.3. No clear explanation for this change from the 1977 Code was provided. From Eq. (1), (2), and (5), it can be shown that for columns with square perimeter ties, the efficiency of ties as confining steel varied from 50 to 75 percent of that of spirals (Fig. 2). As for spirals, the lower limit in Eq. (5), which is applicable to columns in which $A_g/A_{ch} \leq 1.4$ sets the minimum confinement for the purpose of ductility.

The Code also required that this lateral steel be distributed over regions where inelastic action is considered to be likely. The length of this region was defined to be above and below each connection and on both sides of any section where flexural yielding is likely to occur, or in other words, where plastic hinges are expected. The use of crossties with a 180 deg hook at one end and a 90 deg hook at the other end was allowed for the first time to provide ease of construction. Efficiency of the 90 deg hooks in confining the concrete core and preventing the premature buckling of longitudinal bars has proven to be somewhat doubtful, especially under high axial load levels.[7-9]

The 1989 and 1995 Codes—In view of the importance of providing reinforced concrete structures with adequate toughness to respond inelastically under severe seismic attacks, Appendix A was moved to form Chapter 21 in the main body of the 1989 Code.[1] Eq. (1), (2), and (5) remained unchanged except that the factor 0.12 in Eq. (5) was changed to 0.09. This change was based on the observed behavior of tied columns, which had properly detailed hoops and crossties, and made the relative efficiency of rectilinear lateral reinforcement reasonably uniform for all sizes of columns (Fig. 2). Fig. 3 shows a comparison of the behavior of columns with different steel arrangements tested under concentric compression.[10,11,14] It is clear that the relationship between the efficiencies of circular and rectilinear lateral steel is not as simple as assumed in the code. The following points can be made in this regard: 1. The current code equations assume that all steel configurations in tied columns result in similar column behavior. Extensive experimental evidence indicates that section ductility and strength varied significantly from one configuration to another[7-12] under axial load only as well as under combined axial load and bending moment; 2. The Code philosophy of maintaining axial load capacity of a section after spalling of the cover concrete ignores the most important parameter, ductility, when a column is subjected to axial load and flexure. A measure of ductility should be included in the design equations; 3. The axial load level in the column has a great effect on the column behavior.[7-9,12,13] Since considerably high levels of axial load are permitted by many codes for seismic design of columns,

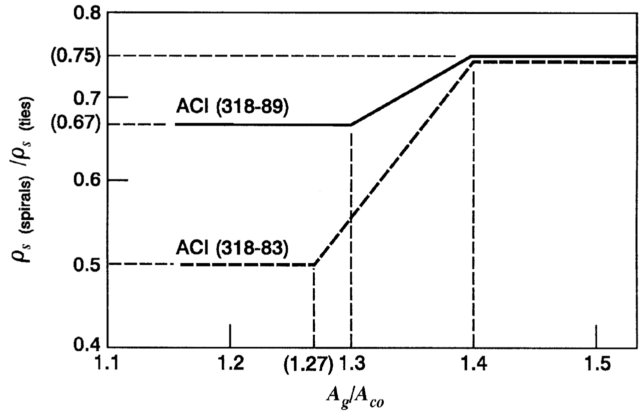


Fig. 2—Comparison of effectiveness of spirals and rectilinear ties

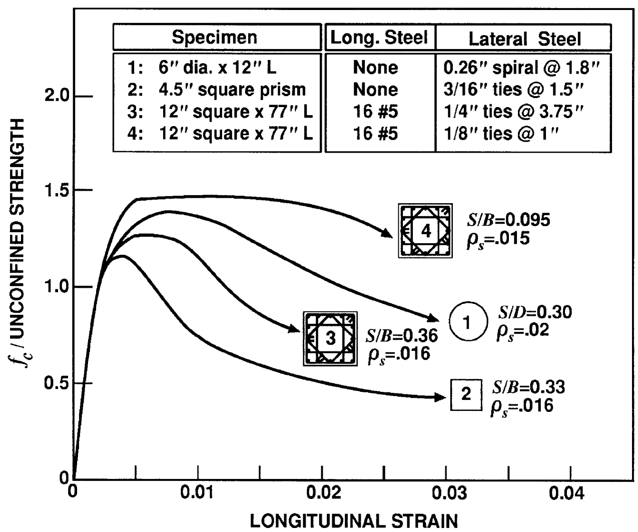


Fig. 3—Comparison of circular and rectilinear confinement

the detrimental effects of higher axial loads must be compensated for by using larger amounts and more efficient configuration of lateral steel. The confinement equations in the 1982 NZS Code[2] were almost similar to Eq. (5) except for the additional multiplication factor of $0.5 + 1.25(P_e/\phi f_c' A_g)$ where P_e is the column axial load, and ϕ is the strength reduction factor. At values of $P_e/\phi f_c' A_g$ less than 0.4, the ACI Code was more conservative than the 1982 NZS Code and for higher axial load, the NZS Code requires more lateral steel than the ACI Code, up to 50 percent more for $P_e/\phi f_c' A_g$ equal to 0.75. Neither code attempted to quantitatively relate the required amount of lateral steel to column performance. In the 1995 version of the New Zealand Code,[2] the amount of lateral steel is related to the level of axial load and is aimed at producing highly ductile columns.

RESEARCH SIGNIFICANCE

In the light of the previous research and ensuing conclusions about the inability of the existing ACI Code provisions for confinement to provide columns with an adequate level of ductility in many circumstances, the need for a procedure for the design of confining steel becomes clear. Such a procedure that relates confinement parameters to the column performance is proposed here on the basis of results from an extensive experimental program. This procedure includes the effects of two additional variables which are not considered in the current Code equations and have been proven[4,5,7-13] to significantly affect the confinement effectiveness and consequently the column behavior. These variables are the level of axial load and the steel configuration. In the proposed method the required amount of confining steel is increased with an increase in ductility demand. The proposed procedure lends itself to a design chart which should be of interest to researchers, and practitioners engaged in the design of ductile moment-resisting reinforced concrete frames.

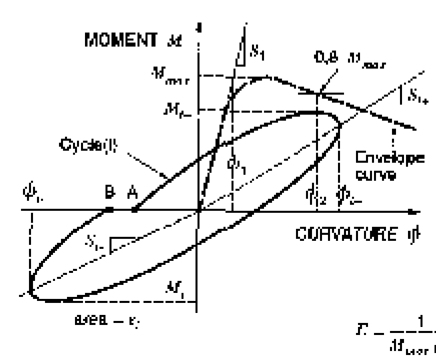
PROPOSED APPROACH

The proposed approach was developed using the experimental results of twenty-nine large-sized specimens reported elsewhere.[7-9,14] Initial development of this design procedure is given in Ref. 15. Some background data and the procedure is briefly explained here.

Column performance—In evaluating the column performance and studying the effects of different variables, ductility and toughness parameters defined in Fig. 4 were used. These include curvature ductility factor μ_ϕ, cumulative ductility ratio N_ϕ, and energy-damage indicator E. Wherever used, subscripts t and 80 indicate, respectively, the value of the parameter until the end of the test (total value) and the value until the end of the cycle in which the moment is dropped to 80 percent of the maximum value. Energy parameter e_i represents the area enclosed in cycle i by the M-ϕ loop. All other terms are defined in Fig. 4 except L_f and t which represent the length of the most damaged region and section depth of the specimen, respectively. The energy-damage indicator E is similar to the one proposed by Ehsani and Wight[16] for force-deflection curves. Table 1 lists the available ductility parameters for all the specimens considered in this analysis.

In order to relate various ductility parameters, energy index E_{80} and cumulative ductility ratio $N_{\phi 80}$ are plotted against curvature ductility factor μ_ϕ in Fig. 5. Data from nine similar specimens that were tested under similar conditions with constant axial load and cyclic lateral loads were used in the construction of this figure. A reasonable correlation exists between the parameters in the figure. For μ_ϕ of 16, the values for $N_{\phi 80}$ and E_{80} are 64 and 575, respectively. A column section with this level of deformability is defined as highly ductile. With a μ_ϕ value of 8 to 16, the section is defined as moderately ductile and the low ductility column has $\mu_\phi < 8$. With this correlation between ductility parameters, the specimens tested under monotonic flexure (last 15 specimens in Table 1) could also be considered in the analysis. In typical columns of framed structures curvature ductility factor μ_ϕ and displacement ductility factor μ_Δ are directly related. Assuming an elasto-plastic section response and constant curvature over an equivalent plastic hinge length (L_p), the main variables that affect the relationship between μ_ϕ and μ_Δ are L_p, the column length L between the point of maximum moment and the point of contraflexure and the type of lateral load applied. The equivalent plastic hinge length has been found to depend on several factors such as length L, section size, and longitudinal bar diameter, but it is unaffected by parameters that comprise confining steel.[7,8,13,17] Since confinement of concrete in columns will only affect μ_ϕ directly, curvature ductility rather than displacement ductility is therefore used as a parameter in the proposed procedure. For drift-based design story/column drift can then be easily calculated using μ_ϕ and plastic hinge length for specific geometric and loading conditions.

Axial load level—Increased axial load reduces ductility significantly.[7-9] The level of axial load is generally measured by indices $P/f_c'A_g$ and P/P_o. For columns with similar f_c', both these indices provide similar comparison. However, for different f_c' values in columns the comparison using $P/f_c' A_g$ may not remain valid. Fig. 6 shows moment-curvature responses of 4 columns. Effect of a change in axial load on the column behavior can be evaluated from Specimens AS-3 and AS-17, which are almost identical in every other regard. Increase in load from 0.66 $f_c'A_g$ to 0.77 $f_c'A_g$ resulted in a sig-



$$\phi_i = \frac{1}{2}(\phi_{i-} + \phi_{i+})$$
$$M_{i,max} = \frac{1}{2}(M_{i+} + M_{i-})$$
$$S_i = \frac{1}{2}(S_{i-} + S_{i+})$$
$$\mu_\phi = \frac{\phi_2}{\phi_1}$$
$$N_\phi = \sum_{i=1}^{i=n} \frac{\phi_{i}}{\phi_1}$$
$$E = \frac{1}{M_{max}\phi_1} \sum_{i=1}^{i=n} e_i \frac{L_f}{t} \left(\frac{S_i}{S_1}\right)\left(\frac{\phi_i}{\phi_1}\right)^2$$

Fig. 4—Ductility parameters

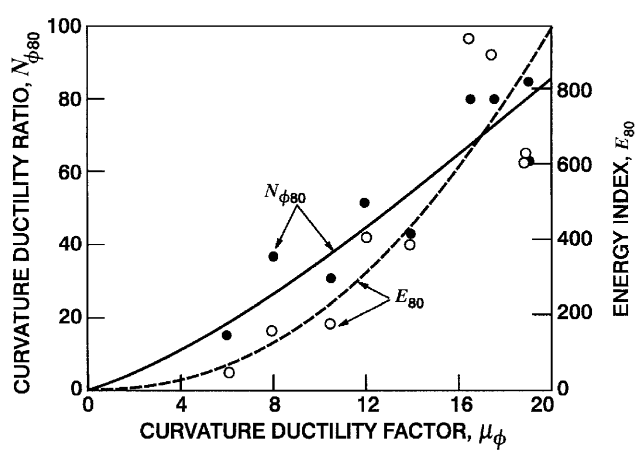


Fig. 5—Relationships between curvature and displacement ductility factors

nificantly less ductile behavior. Curvature ductility factor μ_ϕ was reduced by about 45 percent.

Specimens AS-3 and AS-3H contained the same amount of tie steel and were tested under similar axial loads as represented by index $P/f_c'A_g$. Specimen AS-3H made with higher concrete strength, f_c', displayed much lower ductility. Specimens AS-3 and AS-18H contained about 45 percent more tie steel than that required by the ACI Code[1] and both were tested under $P/f_c'A_g$ approximately equal to 0.6. Ductility and energy dissipation capacity of the higher strength concrete specimen is considerably lower. Specimen AS-17 is comparable to Specimen AS-18H from a point of view that both contained about 150 percent of the code-required tie steel contents and both were subjected to axial loads that were approximately 60 percent of the ultimate load capacity P_o. Moment-curvature responses and ductility parameters of these two specimens are reasonably similar. It can be concluded that the amount of tie steel required for a certain flexural response of columns for a given P/P_o is approximately proportional to the concrete strength.

Steel configuration—The effectiveness of confining steel primarily depends on the area of the effectively confined concrete and the distribution of confining pressure which are in turn highly affected by the distribution of longitudinal and lateral steel and the extent of lateral restraint provided to the bars.[4,5,9] Fig. 7 explains the concept of effectively confined concrete area within a column core in two different steel configurations.[5] With larger number of longitudinal bars laterally supported by tie bends, the area of effectively confined concrete is increased and the efficiency of confinement improves considerably. From a four-bar configuration to an eight-bar configuration the efficiency improvement is very large. Beyond the eight-bar configuration, the confinement efficiency does not increase as significantly with the increase in the number of laterally-supported longitudinal bars.

Fig. 8 shows moment-curvature responses of two column specimens ES-13 and FS-9. These specimens and Specimen AS-17 in Fig. 6 are almost identical in all regards except steel configuration. Specimen AS-17 displayed more ductile behavior (also see Table 1) than Specimen FS-9 which in turn is tougher than Specimen ES-13.

Based on this concept and extensive experimental data[4,7,8,9,12,14] steel configurations may be divided into the following three main categories (see Fig. 9):

- Category I: where only single-perimeter hoops are used as confining steel

Table 1—Details of specimens

Researchers	Specimen	f_c', ksi	Lateral steel: Spacing, in.	Lateral steel: ρ_s, percent	Lateral steel: f_{yh}, ksi	Lateral steel: $\frac{A_{sh}}{A_{shc}}$	Axial load level: $\frac{P}{f_c'}$	Axial load level: $\frac{P}{P_o}$	μ_ϕ	Ductility ratio: $N_{\phi 80}$	Ductility ratio: $N_{\phi t}$	Energy indicator: E_{80}	Energy indicator: E_t
Khoury and Sheikh[10]	AS-3	4.81	4.25	1.68	73.6	1.43	0.599	0.498	19.0*	63	74	610	753
	FS-9	4.70	3.75	1.68	73.6	1.46	0.761	0.628	8.0	37	44	154	163
	ES-13	4.72	4.50	1.69	67.3	1.34	0.758	0.626	6.0	15	26	53	110
	AS-17	4.54	4.25	1.68	73.6	1.52	0.765	0.626	12.0	52	58	402	443
	AS-18	4.75	4.25	3.06	67.3	2.41	0.768	0.636	17.5	80	92	897	1156
	AS-19	4.68	4.25	1.30	72.2	1.12	0.467	0.386	19.0	85	129	631	1230
	F-9H	8.45	3.75	1.68	73.6	0.81	0.641	0.615	5.0	—	—	—	—
	E-13H	8.36	4.50	1.69	67.3	0.76	0.631	0.605	2.5	—	—	—	—
Sheikh, Shah, and Khoury[11]	AS-3H	7.85	4.25	1.68	73.6	0.88	0.619	0.585	10.5	31	35	178	204
	AS-18H	7.93	4.25	3.06	67.3	1.44	0.639	0.605	14.0	43	59	384	458
	AS-20H	7.78	3.00	4.30	67.3	2.10	0.643	0.607	16.5	80	98	935	1170
Patel and Sheikh[16]	F-9L*	6.81	3.75	1.68	73.6	1.01	0.637	0.583	7.0				
	E-13L	6.95	4.50	1.69	67.3	0.91	0.649	0.597	5.0				
	A-17L	7.12	4.25	1.68	73.6	0.97	0.658	0.609	10.3				
Yeh and Sheikh[12]	E-2	4.55	4.50	1.69	70.0	1.45	0.611	0.443	10.0				
	A-3	4.61	4.25	1.68	71.0	1.44	0.603	0.492	28.5				
	F-4	4.67	3.75	1.68	71.0	1.41	0.595	0.491	21.3				
	D-5	4.53	4.50	1.68	71.0	1.39	0.460	0.387	20.0				
	F-6	3.95	6.81	1.68	70.0	1.65	0.747	0.578	10.3				
	D-7	3.80	2.13	1.62	68.0	1.52	0.777	0.618	16.0				
	E-8	3.76	5.00	0.84	70.0	0.87	0.776	0.600	3.5				
	F-9	3.84	3.75	1.68	71.0	1.72	0.769	0.589	6.2				
	E-10	3.81	2.50	1.68	71.0	1.73	0.766	0.585	5.2				
	A-11	4.05	4.25	0.77	68.0	0.72	0.737	0.576	8.3				
	F-12	4.85	3.50	0.82	67.0	0.63	0.601	0.499	9.4				
	E-13	3.95	4.50	1.69	70.0	1.67	0.738	0.571	12.3				
	D-14	3.90	4.25	0.81	67.0	0.73	0.748	0.600	7.3				
	D-15	3.90	4.50	1.68	71.0	1.61	0.748	0.600	12.3				
	A-16	4.92	4.25	0.77	81.0	0.71	0.600	0.500	13.2				

* Lightweight aggregate concrete specimens

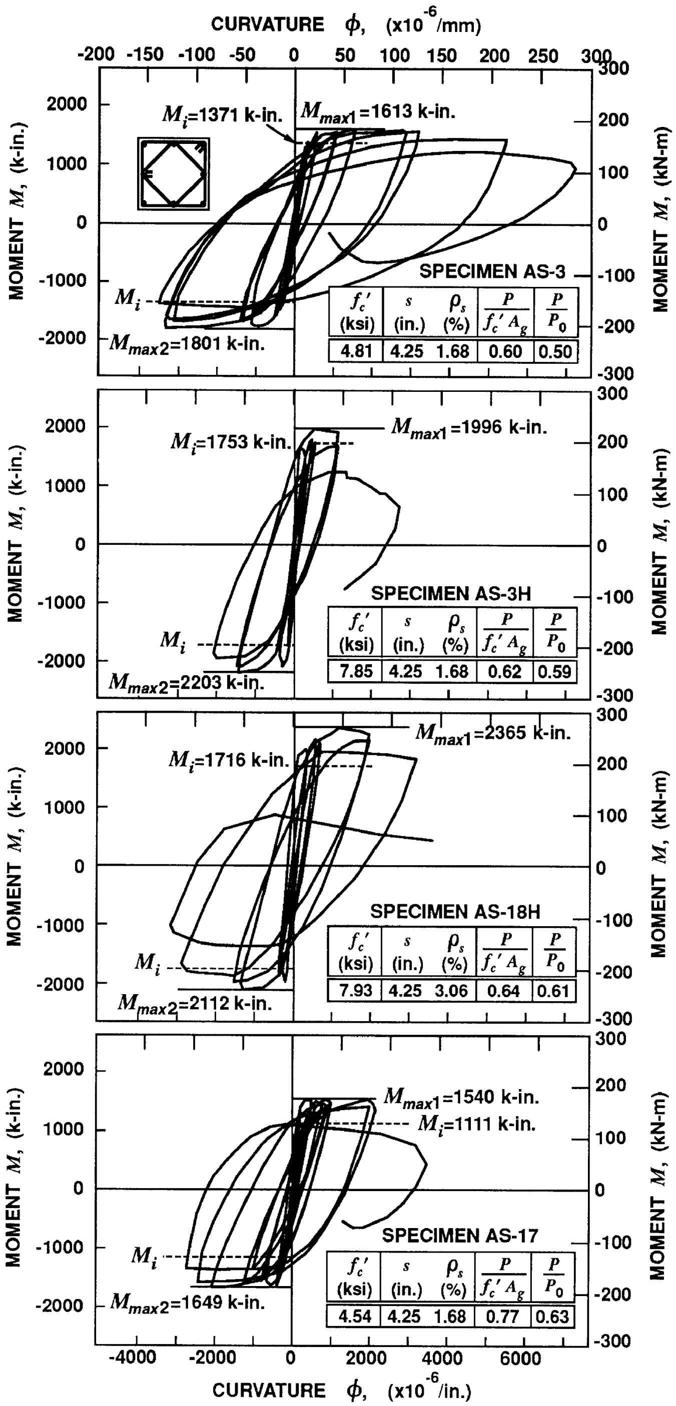


Fig. 6—Effects of axial load and concrete strength

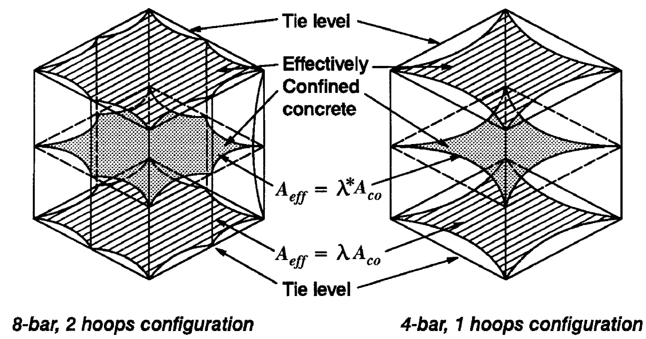


Fig. 7—Concept of effectively confined concrete area

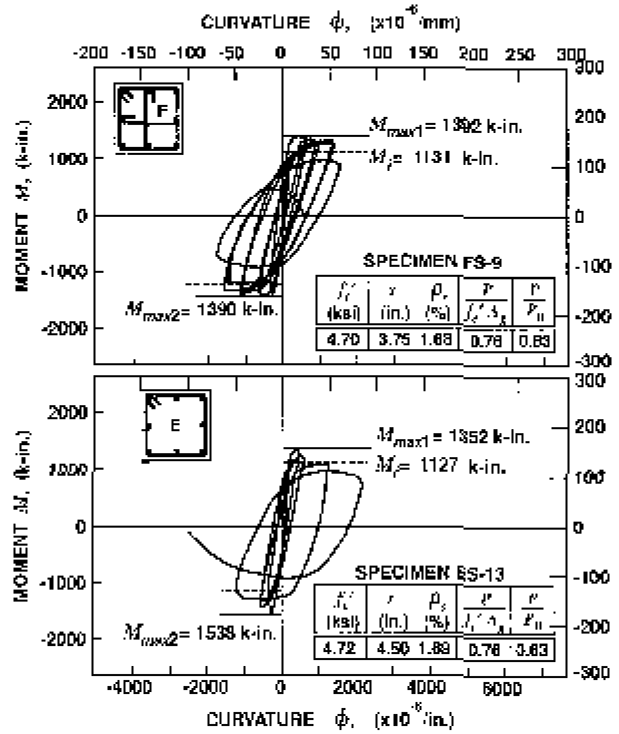


Fig. 8—Effect of steel configuration

- Category II: in addition to the perimeter hoops supporting four corner bars, at least one middle longitudinal bar at each face is supported at alternate points by hooks that are not anchored in the core. At other points the supporting hooks are anchored in the core.
- Category III: in which a minimum of three longitudinal bars are effectively supported by tie corners on each column face and hooks are anchored into the core concrete.

Limiting conditions for steel configurations—For earthquake design, it is believed that only the two top categories of ductility, high and moderate, are needed to be discussed here. Among the specimens tested during this research program none of the specimens with Configuration E (Category I) resulted in high section ductility factor (μ_ϕ). The axial load level in these specimens was high but in other studies (e.g., Ref. 12) columns with E sections tested under low axial load level ($P < 0.3P_o$) also showed unsatisfactory behavior. Based on the observed experimental performance and the analytical evidence (Fig. 8), Category I configuration is not recommended for high ductility columns.

Specimen ES-13, which contained 34 percent more steel than required by the Code, showed very poor behavior with $\mu_\phi = 2.5$. Also under medium level of axial load (about 0.4-$0.45P_o$), Specimen E-2 with 1.45 times the Code required steel exhibited μ_ϕ of only about 10. Therefore, the use of Configuration E in moderately ductile columns should be limited to lower range of axial load ($P < 0.40P_o$). For conservative design, the Category I configurations are recommended for moderate ductility columns only if the applied axial load is less than the balanced load P_b.

With regard to Category II configurations, the effectiveness of hooks not anchored in the core has been a controversial issue. Although some researchers concluded that the supplementary crossties allowed columns to perform in a ductile manner,[12,18,19] the axial load levels in these tests were low (about $0.1P_o$ to $0.3P_o$). Recent research[7-9] has shown that the use of 90 deg hooks in Section F (Fig. 9) may provide sufficient restraint to the middle bars up to a certain stage of loading, but at large deformations the 90 deg hooks tend to open, and the restraint provided to the bars becomes ineffective resulting in a loss of confinement. None of the reported specimens showed satisfactory performance under high levels of axial load even when lateral steel content was in excess of the Code requirements (see Table 1). Although Specimen F-4 indicates very ductile performance, the same section in Specimen F-9 under higher axial load level shows undesirable behavior for seismic resistance. It should be noted that the 90 deg hooks were not always in the zone of maximum deformation due to the monotonic nature of flexural loading in this set of specimens. Therefore, high apparent ductility in some specimens may not be repeatable. Accordingly, it is recommended that the use of Category II configurations to produce high-ductility columns be limited to cases with low levels of axial load. These columns can be used for moderate ductility if axial load does not exceed $0.4P_o$.

The limiting conditions under which the three categories of steel configurations may be reliably used for moderate and high ductility columns are outlined in Fig. 10. It should be emphasized here that some configurations under such conditions may require a higher amount of lateral steel than other configurations.

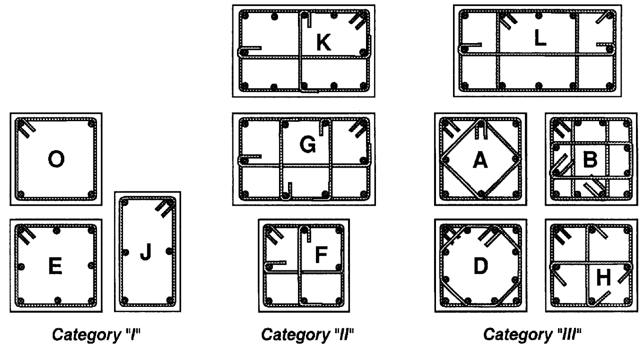


Fig. 9—Categories of steel configurations

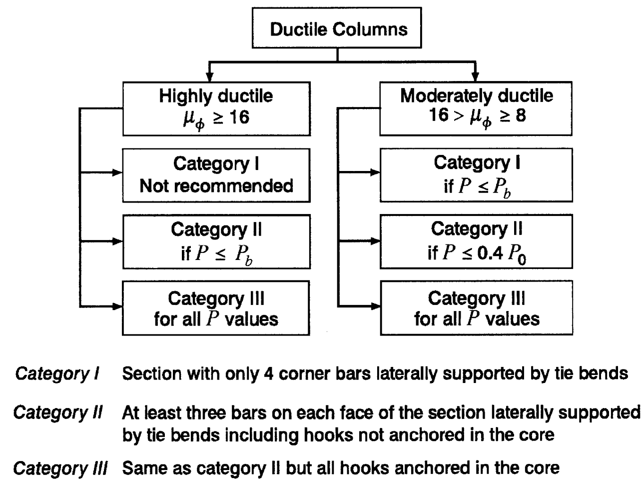


Fig. 10—Limiting conditions for steel configurations

AMOUNT OF CONFINING STEEL

General form of proposed equation

The relationship between the amount of lateral steel as recommended by the current Code $A_{sh,c}$ and the suggested amount of lateral steel A_{sh} is taken as:

$$A_{sh} = (A_{sh,c})Y \tag{6}$$

where Y is a factor expressed as

$$Y = \alpha Y_p Y_\phi \tag{7}$$

where α is a parameter that accounts for the confinement efficiency including configuration and the lateral restraint provided to the longitudinal bars. Parameters Y_p and Y_ϕ take into account the effect of axial load level and the section ductility demand, respectively.

Parameter α—Parameter α is assumed to be equal to unity for Category III configurations. This factor is expected to be greater than unity for Category I configurations even for their use under limiting conditions prescribed earlier. For such a case, the value for α is estimated in a later section. Use of Category II configurations is subjected to imposed limitations because some of the hooks are not anchored in the core as previously discussed. It is reasonable to assume a value of α equal to unity for these configurations in situations where opening of these hooks does not take place until after sufficient ductility is exhibited.[7-9] In the event of high axial load levels, the value of α would be much greater than unity; however such an application should be avoided.

Development of expressions for parameters Y_p *and* Y_ϕ—Eq. (6) and (7) for sections with at least three longitudinal bars effectively restrained on each face ($\alpha = 1$) reduce to

$$\frac{A_{sh}}{A_{sh,c}} = Y_p Y_\phi \tag{8}$$

After investigating several possible forms of expressions for Y_p and Y_ϕ, the following simple forms were selected,

$$Y_p = a_1 + a_2\left(\frac{P}{P_o}\right)^{a_3}, \text{ and} \tag{9}$$

$$Y_\phi = b_1(\mu_\phi)^{b_2} \tag{10}$$

where a_1, a_2, a_3, b_1, and b_2 are constants to be determined empirically.

As a starting point, since the two parameters Y_p and Y_ϕ are independent of each other, the value of Y_ϕ is assumed to be unity for highly ductile sections with μ_ϕ equal to or greater than 16. Specimens meeting this requirement are AS-3, AS-18, AS-19, AS-20H, A-3 and F-4. Using the results from these specimens, a least squares analysis was performed to find constants a_1 and a_2 for selected values of a_3 that ranged from 1 to 6. Corresponding to each chosen value of a_3, and consequently obtained values for a_1 and a_2, the constants b_1 and b_2 in the expression for Y_ϕ (Eq. 10) were then determined using the test results for those 16 specimens in which $\alpha = 1.0$. These included all the specimens with A and D configurations and Specimens F-4 and F-12 from Table 1. Specimen A-3 was not included in the analysis since its μ_ϕ was unusually large compared with other similar specimens.

Minimization of the total cumulative error for all the 16 specimens was the only criterion used to select the final values of the empirical constants. The cumulative error e^2 was calculated as

$$e^2 = \sum_1^{16} (Y_{\exp} - Y_{pred})^2 \tag{11}$$

where $Y_{exp} = A_{sh}/A_{sh,c}$, and $Y_{pred} = Y_p\, Y_\phi$. The best fit curves from this analysis are shown in Fig. 11 and 12. The expressions for parameters Y_p and Y_ϕ are given below

$$Y_p = 1 + 13\left(\frac{P}{P_o}\right)^5 \tag{12}$$

and

$$Y_\phi = \frac{(\mu_\phi)^{1.15}}{29} \tag{13}$$

The correlation coefficients for Eq. (12) and (13) are 0.99 and 0.93, respectively. The high coefficients indicate excellent agreement between the analytical and the experimental values. The cumulative error e^2 over the 16 specimens used in the final analysis was 0.519 yielding as average error of 0.032 per specimen. The parameter Y_ϕ, when checked for highly ductile column sections, was found to be almost unity for the average value of μ_ϕ equal to 18.5.

Final form of design equation—Based on the above, the amount of lateral steel in tied columns may be calculated using the following expression

$$A_{sh} = \alpha\left\{1 + 13\left(\frac{P}{P_o}\right)^5\right\}\frac{(\mu_\phi)^{1.15}}{29}A_{sh,\,c} \tag{14}$$

Factor α is unity for Category III configurations and for Category II configurations as long as the prescribed limiting conditions are met. However, for Category I configurations, the α value is greater than unity.

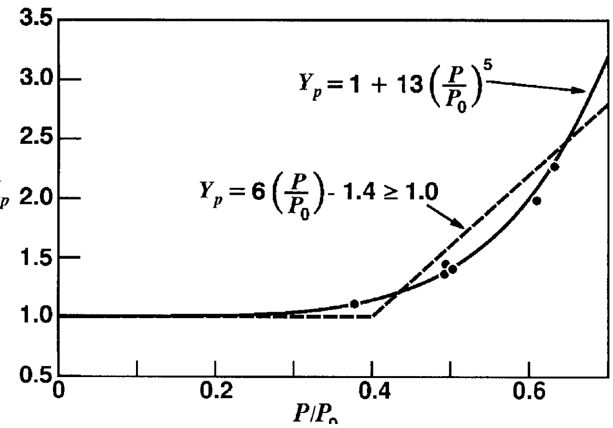


Fig. 11—Required amount of tie steel as affected by axial load

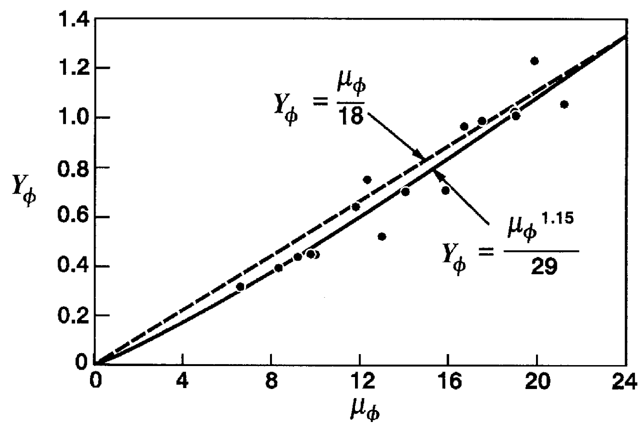


Fig. 12—Required amount of tie steel as affected by curvature ductility factor

The above procedure is applied to all those specimens tested during this program in which longitudinal bars were effectively supported laterally. Comparison between the analytical and the experimental curvature ductility factors is shown in Fig. 13. The correlation coefficient is 0.94 with an average difference between the test and the predicted values less than 10 percent. As mentioned above, minimization of the total cumulative error was the only criterion used in the development of Eq. (14). No attempts were made to minimize the error in individual columns.

Fig. 11 and 12 also show simplified versions of equations for Y_p and Y_ϕ as given below

$$Y_p = 6\frac{P}{P_o} - 1.4 \geq 1.0 \tag{15}$$

$$Y_\phi = \frac{\mu_\phi}{18} \tag{16}$$

Eq. (15) provides a conservative estimate for Eq. (12) for most of the axial load range up to P/P_o equal to 0.65. It should be noted that the allowable axial load for tied columns is $0.56P_o$ (1). Eq. (16) gives a slightly conservative alternative to Eq. (13). Considering Eq. (15) and (16), Eq. (14) can be rewritten as:

$$A_{sh} = \alpha\left[6\frac{P}{P_o} - 1.4\right]\left[\frac{\mu_\phi}{18}\right]A_{sh,c} \geq \alpha\frac{\mu_\phi}{18}A_{sh,c} \quad (17)$$

Design for Category I configurations—The parameter α may be estimated in this case by using the experimental results. Values for α were calculated using Eq. (14) for all the specimens with Configuration E from Table 1 and are listed in Table 2. The average value of α is about 2.70 which implies that the amount of lateral steel needed in sections with Configuration E to attain a specific ductility demand may be two to three times that required for sections with Configuration A. The experimental M-ϕ relationships reported previously[8] confirm this finding.

The factor α for Category I configurations may also be estimated by adopting the concept of "effectively confined concrete core area"[5] as shown in Fig. 7. The ratio between the area of effectively confined concrete and the total concrete area λ at tie level is given by

$$\lambda = 1 - \frac{\sum_{i=1}^{n} C_i^2}{5.5A_{co}} \quad (18)$$

where A_{co} = the core area enclosed by the center line of perimeter hoop; C_i is the base of the curve representing the area which is not effectively confined; and n = the number of these curves.

It may be reasonably assumed here that the configuration parameter α is proportional to 1/λ. Since α = 1 for Category III configurations, α for Category I configurations (α_I) may be written as $\alpha_I = \lambda_{III}/\lambda_I$ where λ_{III} and λ_I can be calculated using Eq. (18). For the specimens in which the longitudinal bars are uniformly distributed around the core perimeter, the λ values for Configurations A and O (Fig. 9) are 0.636 and 0.273, respectively. Hence, α_I = 2.33. The ductility of Section E with 8 longitudinal bars, 4 corner bars, and 4 unsupported middle bars was sometimes observed to be even worse than that of Section O with only four corner bars.[20] It may be reasonable, therefore, to conclude that the factor α for Category I configurations may range from 2.3 to 2.7. An average value of 2.5 is thus assumed for all configuration types in this category.

As suggested before, the use of Category I configurations may be reliable only for moderate ductility columns under axial load level below the balance point. For this low axial load, $1 + 13(P/P_o)^5 \approx 1.0$. Taking α equal to 2.5, Eq. (14) and (17) reduce to Eq. (19) and (20), respectively.

$$A_{sh} = \frac{(\mu_\phi)^{1.15}}{11.5}A_{sh,c} \quad (19)$$

$$A_{sh} = \frac{\mu_\phi}{7.2}A_{sh,c} \quad (20)$$

Based on Eq. (19) and (20), it can be stated that for axial load below the balance point, the current ACI Code steel may be sufficient to provide μ_ϕ of about 7 to 8 for sections with Configuration E. However, for a moderately ductile column with μ_ϕ = 12, the required amount of lateral steel should be 50 percent higher than that required by the Code.

Design chart

On the basis of the proposed equations [Eq. (14) and (17)], a design chart is constructed in Fig. 14 for columns in which a minimum of 3 longitudinal bars are effectively supported laterally in each face (Category III configurations). The amount of required lateral steel increases with an increase in the axial load level and an increase in the ductility demand. Three ductility zones as discussed earlier are indicated in the figure which shows that the Code prescribed amount of tie steel may be adequate to provide high ductility columns only if the applied axial load is less than $0.4P_o$, and moderate ductility columns under higher axial loads as long as at least three longitudinal bars are effectively supported laterally on each column face. Under high levels of axial load, the Code required amount of lateral steel may not be sufficient to meet high ductility demand. The same figure can be used for Category II sections as long as the limiting conditions shown in Fig. 10 are satisfied. For columns with Category I configura-

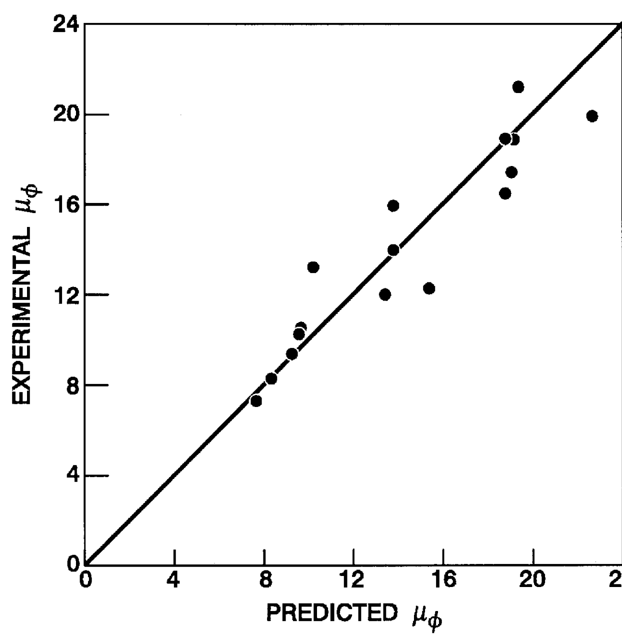


Fig. 13—Comparison of experimental and predicted curvature ductility factors

Table 2—Calculated α values for specimens with Configuration E

Specimen	ES-13	E-13H	E-13L	E-2	E-8	E-10	E-13
Y_p	2.25	2.05	1.99	1.22	2.01	1.89	1.79
Y_ϕ	0.27	0.10	0.22	0.49	0.15	0.23	0.62
α	2.20	3.73	2.09	2.43	2.95	3.99	1.51

tions ($\alpha = 2.5$), the code-provided amount of lateral steel will be insufficient to provide even moderately ductile columns under axial loads exceeding balanced load ($P/P_o \approx 0.3$).

Effect of hoop/tie spacing

Experimental and theoretical evidences show that hoop spacing plays a significant role in the mechanism of confinement.[4,5] Larger ratio of hoop spacing s to core width B will result in smaller area of effectively confined concrete in the core (Fig. 7). The procedure presented here for the sake of simplicity does not include tie spacing as an active parameter. However, it should be noted that the test data on which the equations are based were obtained from specimens in which tie spacing varied from $0.20B$ to $0.43B$. In this practical range of spacing, the confinement mechanism has reasonably high efficiency. Another important reason to limit spacing is to avoid premature buckling of longitudinal bars when a column is subjected to seismic excursions in the inelastic range. In the specimens considered here, tie spacings varied between $3.4d_b$ and $7.2d_b$ where d_b is the bar diameter. In this range of s/d_b, premature buckling of the longitudinal bars can generally be avoided.[2,13] The proposed procedure can be used to design the confining steel for a given column performance as long as the tie spacing is less than $0.43B$ and $7d_b$. For a conservative design the limit to the tie spacing is suggested to be the smallest of $B/3$, $6d_b$ and 200 mm (8 in.).

APPLICATION OF THE PROPOSED DESIGN APPROACH

The proposed equation is applied to a 700 mm (27.6 in.) square column previously reported by Park et al.[21] The results are presented graphically in Fig. 15 and are compared with the ACI and NZS Code requirements as well as the lateral steel required according to Park.[22] For comparative purposes, the analytical equation proposed here was converted to be a function of ($P/f_c'A_g$) instead of (P/P_o).

Under high axial loads, the steel required for highly ductile columns according to Park is significantly less than that based on the proposed equation. However, for curvature ductility factor μ_ϕ equal to 10, the requirements according to Park are somewhat conservative compared with the proposed curve. The amount of steel required according to the ACI Code is inadequate to achieve $\mu_\phi = 10$ under high axial loads even for well-configured columns. The 1982 NZ code requirements for lateral steel produced highly ductile to moderately ductile columns for most of the axial load range. In the 1995 version of the NZ code, the effect of axial load is more severe compared with the 1982 version. The required amount of lateral steel, according to the 1995 NZ code, is similar to that propposed by Park[22] for $\mu_\phi = 20$.

Eq. (14) is also applied to six specimens reported by Muguruma and Watanabe[23] and by Azizinamini et al.[24] Details of these specimens are given in Table 3 which also compares the analytical values of curvature ductility factors with the experimental values. The comparison shows excellent agreement. The only difference may be noticed in Specimen AL-2. It is believed that the actual value of μ_ϕ for this specimen is greater than the experimental value listed in Table 3 since the M-ϕ envelope curve for this specimen was reported only

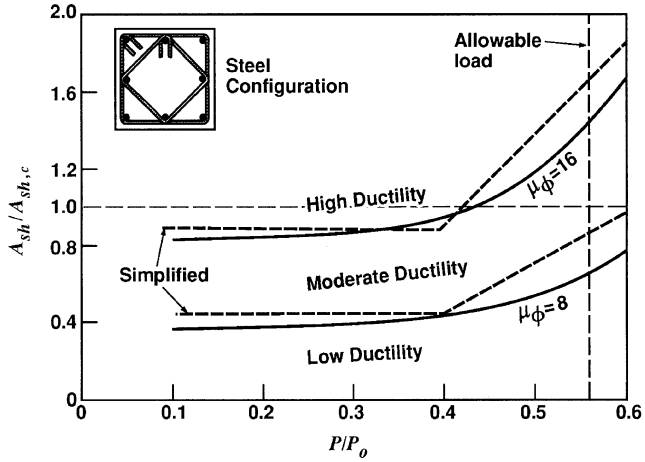


Fig. 14—Lateral steel requirements

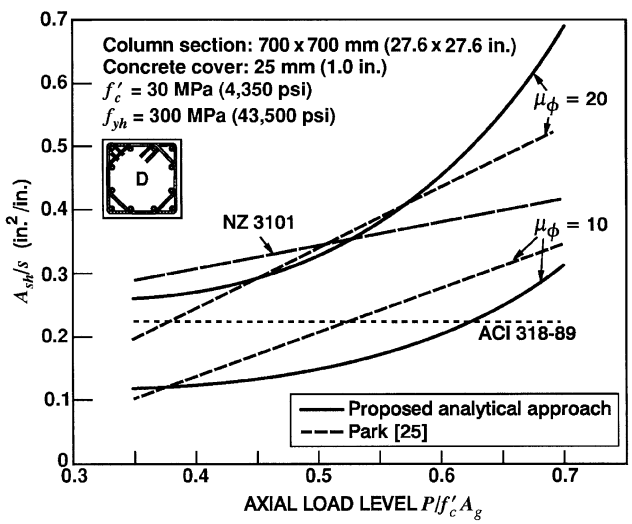


Fig. 15—Application of the design procedures

up to the maximum moment. The post-peak descending part of the curve was not provided.

SUMMARY AND CONCLUSIONS

The development over the years of the ACI Code provisions for the design of confining steel in tied columns is presented. It appears that some of the changes made from one edition of the Code to the next are not suitable to provide ductile columns. The current Code requirements are evaluated based on experimental evidences, and the areas in which the requirements need modification are explored.

Although extensive experimental evidence indicates that ductile behavior of confined concrete columns depends significantly on the level of axial load as well as the distribution of steel and the lateral support provided to the longitudinal bars, these factors are not considered in the current Code equations for confining steel. Ductility demand is also not given due importance in the Code design procedure. Therefore, columns designed according to the current Code may display brittle behavior under certain circumstances and may be overly conservative in terms of ductility in other cases. The required amount

Table 3—Application of proposed equations

Specimen	f_c', MPa	Lateral steel Size	@	Spacing, mm	f_{yh}, MPa	Longitudinal steel No.	Size	f_{y1}, MPa	Axial load level $\frac{P}{f_c'A_g}$	$\frac{P}{P_o}$	$\frac{A_{sh}}{A_{shc}}$	Results μ_ϕ, exp.	μ_ϕ, pred.
Specimens* reported by Mugurauma and Watanabe[26]													
BH-2	115.8	6 mm	@	35	792.3	12	ϕ13 mm	399.6	0.423	0.446	1.395	20.3	20.9
AH-2	85.7	6 mm	@	35	792.3	12	ϕ13 mm	399.6	0.629	0.632	1.885	15.6	15.6
BL-2	115.8	6 mm	@	35	328.4	12	ϕ13 mm	399.6	0.423	0.446	0.578	9.0	9.7
AL-2	85.7	6 mm	@	35	328.4	12	ϕ13 mm	399.6	0.629	0.632	0.781	4.1‡	7.3
Specimens† reported by Azizinamini et al.[27]													
NC-2	41.4	#4	@	102	414	8	#8	414	0.20	0.20	0.888	16.8	16.7
NC-5	41.4	#4	@	102	414	8	#8	414	0.30	0.30	0.888	16.4	16.3

* Column section = 200 × 200 mm; core section = 176 × 176 mm; configuration = Type D
† Column section = 18 × 18 in.; core section = 14.5 × 14.5 in.; configuration = Type A
‡ In Ref. 26, the M-ϕ envelope curve for this specimen was reported only up to the maximum moment. The post peak descending part of the curve was not given.

of lateral steel should be a function of axial load level, steel configuration, and the expected ductile performance.

To account for the factors discussed above, a procedure for the design of confining steel in tied columns is proposed. Three categories of steel configurations are outlined. Limiting conditions under which these categories of steel configurations may reliably be used are suggested. A sample design chart is also presented which can be used to determine the amount of lateral steel required for a specific ductility level in a column that contains a minimum of three laterally-supported longitudinal bars in each face and the steel is uniformly distributed. A comparison between the amount of lateral steel as suggested by the new approach and the Code requirements indicates that under moderate-to-high levels of axial loads, the Code requirements may not be sufficient even in well-configured columns to meet the high ductility demand. However, at low axial load levels ($P \leq 0.4P_o$), the code requirements may be relaxed. For steel configurations in which only four corner bars are adequately restrained laterally, the ACI Code design will produce columns with inadequate ductility for most of the axial load range. The proposed equation when applied to realistically-sized specimens tested by different investigators yielded excellent agreement with the test data.

ACKNOWLEDGEMENTS

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