ACI STRUCTURAL JOURNAL TECHNICAL PAPER

Title no. 99-S1

Displacement-Based Design of Reinforced Concrete Columns for Confinement

by Murat Saatcioglu and Salim R. Razvi

A displacement-based design procedure was developed for confinement of earthquake-resistant concrete columns. The procedure is based on experimentally observed and analytically computed relationships among the parameters of confinement. The amount, grade, spacing, and arrangement of transverse reinforcement; concrete strength and cover thickness; and the level of axial compression and drift ratio were considered as parameters of confinement. Static inelastic (pushover) analyses were conducted to generate a large volume of data, with due considerations given to concrete confinement, reinforcement strain hardening and buckling, anchorage slip, axial compression, and secondary deformations due to P-D *effect. Both normal-strength and high-strength concrete columns with circular and square cross sections were included. Improved design expressions were developed for column confinement utilizing both the current design criterion, which is based on column axial deformability, and the recommended design criterion, which is based on lateral deformability as expressed by column drift ratio.*

Keywords: column; ductility; confined concrete; displacement-based design; high-strength concrete; transverse reinforcement.

INTRODUCTION

Reinforced concrete columns built in seismically active regions are expected to undergo a large number of inelastic deformation cycles while maintaining overall strength and stability of the structure. This can be ensured by proper confinement of the core concrete. The confinement requirements of the ACI 318-99 Building Code[1] provide satisfactory designs in most applications.[2-4] The same requirements, however, may also result in unsatisfactory designs, leading to either unsafe or overconservative columns, which often lead to the congestion of reinforcement and related construction problems.[2-4] The code requirements were derived for normal-strength concrete columns and are not applicable to columns cast from high-strength concrete.

The state of knowledge on concrete confinement has improved substantially since the pioneering work of Richart, Brandtzaeg, and Brown[5,6] in 1928, which formed the basis for the ACI 318 design requirements.[1] During the last three decades, a large volume of experimental data has been generated, and a number of improved analytical models have been developed that describe the stress-strain behavior of confined concrete. A better understanding of design parameters has also been acquired, including those that are currently overlooked by the design practice. Test data have become available on high-strength concrete columns, making it possible to develop design provisions for such columns that are not currently included in the ACI 318 building code.[1] It is the objective of this paper to present improved design expressions for normal- and high-strength concrete column confinement on the basis of the current state of knowledge.

CURRENT DESIGN APPROACH

The design criterion adopted in ACI 318-99[1] for column confinement is based on the premise that confined columns should maintain their concentric capacities after the spalling of cover concrete. This is achieved by providing sufficient confinement to the core concrete to attain strength and ductility enhancements. The required volumetric ratio of transverse reinforcement was derived based on the strength gain in core concrete, which was assumed to be $(f'_{cc} - f'_{co}) = 4.1 f_\ell$, where f_ℓ represents uniform passive confinement pressure.[6] Equating the concentric capacity of cover concrete to the strength gain in core, the required volumetric ratio of transverse reinforcement that satisfies the ACI 318 performance criterion may be obtained as follows

Strength in cover concrete = strength gain in core concrete

$$0.85 f_c'(A_g - A_c) = 4.1 f_\ell (A_c - A_s) \tag{1}$$

The lateral pressure f_ℓ for a spirally reinforced circular column at yield is

$$f_\ell = \frac{2A_{sp} f_{yh}}{s h_c} \tag{2}$$

Substituting f_ℓ into Eq. (1) and dividing both sides by $(2.05\, f_{yh} A_c)$

$$0.415 \frac{f_c'}{f_{yh}}\left(\frac{A_g}{A_c} - 1\right) = \frac{4A_{sp}}{s h_c} - \frac{4A_{sp}A_s}{s h_c A_c} \tag{3}$$

$$\frac{4A_{sp}}{s h_c} = \frac{4\pi h_c A_{sp}}{\pi h_c^2 s} = \rho_s = 0.415 \frac{f_c'}{f_{yh}}\left(\frac{A_g}{A_c} - 1\right) + \frac{4A_{sp}A_s}{s h_c A_c} \tag{4}$$

Equation (4) was adopted by ACI 318[1] after dropping the last term and increasing 0.415 to 0.45. The code expression is shown in Eq. (5) for spirally reinforced circular columns where hoop tension results in near-uniform lateral pressure, which is consistent with the degree of strength enhancement considered.

ACI Structural Journal, V. 99, No. 1, January-February 2002.
MS No. 00-205 received September 6, 2000, and reviewed under Institute publication policies. Pertinent discussion will be published in the November-December 2002 *ACI Structural Journal* if received by July 1, 2002.

Murat Saatcioglu, *FACI, is a professor of structural engineering at the University of Ottawa, Ottawa, Canada. He is a member of ACI Committees 374, Performance-Based Seismic Design of Concrete Buildings; 441, Concrete Columns; and is the chair of 340, Design Aids for the ACI Building Code. His research interests include confinement of concrete columns.*

Salim R. Razvi *is an assistant professor at UAE University, United Arab Emirates. He received his MASc and PhD from the University of Ottawa. His research interests include column confinement and performance of high-strength concrete columns.*

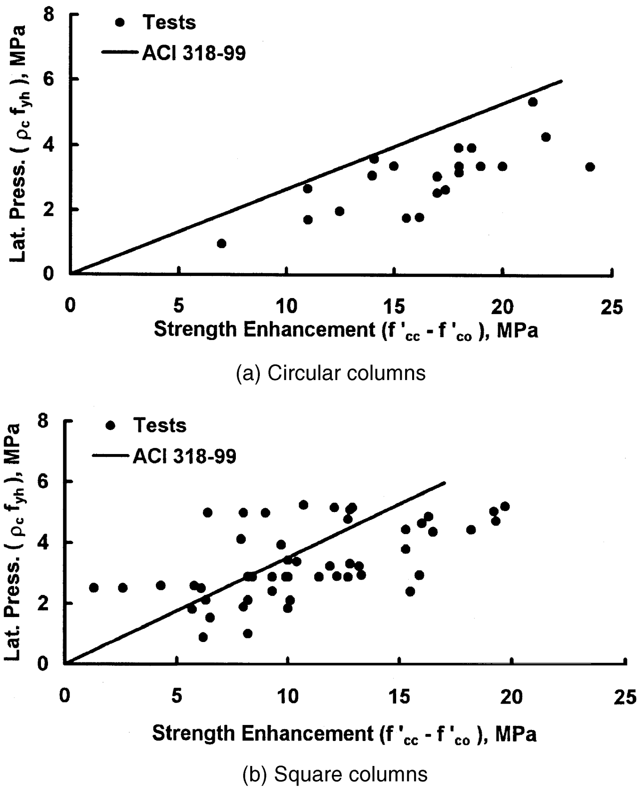


Fig. 1—Comparisons of experimental data with ACI 318 requirements.[1]

$$\rho_s = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_{yh}} \tag{5}$$

With the aforementioned simplification, the strength enhancement in core becomes $(f'_{cc} - f'_{co}) \approx 3.8f_\ell$. The lateral pressure f_ℓ for a spirally reinforced circular column can be written in terms of the area ratio of transverse reinforcement ρ_c in each cross-sectional direction

$$f_\ell = \frac{2A_{sp}f_{yh}}{b_c s} = \rho_c f_{yh} = \frac{1}{2}\rho_s f_{yh} \tag{6}$$

The strength enhancement based on ACI 318 is compared with experimental values obtained by Sheikh and Toklucu[7] and Mander, Priestley, and Park[8] in Fig. 1(a). The comparison indicates that Eq. (5) does not provide a good correlation with experimental data, producing over-conservative quantities of transverse reinforcement for spirally reinforced circular columns. This is attributed to the constant multiplier 3.8 used in relating strength gain to confinement pressure, although test results reported in the literature indicate a variable multiplier that is a function of lateral pressure f_ℓ.[9,10]

In large columns, the ratio of cross-sectional area to confined core area (A_g/A_c) may approach unity. In this case, Eq. (5) results in very small values of volumetric ratio. Therefore, a lower-bound expression is provided by setting a limit on the A_g/A_c ratio. This translates into Eq. (7)

$$\rho_s = 0.12\frac{f_c'}{f_{yh}} \tag{7}$$

The confinement steel requirements for square and rectangular columns are based on an arbitrary extension of the aforementioned requirements, while recognizing that rectilinear reinforcement is not as effective as circular reinforcement. The code expression for the required area of rectilinear reinforcement is obtained from Eq. (5), based on the premise that rectilinear reinforcement is 3/4 as effective as circular spirals. This implies that 1/3 more steel is needed in square and rectangular columns to attain deformabilities usually expected from spirally reinforced circular columns. The increased steel requirement, when expressed in terms of the area of lateral reinforcement, translates into Eq. (8) with the corresponding strength enhancement of $(f'_{cc} - f'_{co}) \approx 2.8f_\ell$. The lower limit established is similar to that for circular spirals, and is shown in Eq. (9)

$$A_{sh} = 0.3sh_c\frac{f_c'}{f_{yh}}\left(\frac{A_g}{A_c} - 1\right) \tag{8}$$

$$A_{sh} = 0.09sh_c\frac{f_c'}{f_{yh}} \tag{9}$$

The previously mentioned requirements of the ACI 318 building code[1] are compared with the results of concentrically tested columns, obtained by Sheikh and Uzumeri,[11] Scott, Park, and Priestley,[12] Razvi and Saatcioglu,[13] and Abdulkadir,[14] in terms of lateral pressure $\rho_c f_{yh}$ and the resulting strength enhancement. The comparison, shown in Fig. 1(b), indicates poor correlation. This may be explained mostly by the differences in behavior resulting from different arrangements of reinforcement, which is a parameter that is not considered in Eq. (8). Researchers in the past showed that columns with the same amount and spacing of confinement reinforcement showed significantly different strength and deformability when confined by different arrangements of transverse reinforcement.[11,12,15,16] While a square column with four corner bars and tied with perimeter hoops shows the worst behavior, columns confined with well-distributed longitudinal reinforcement, laterally supported by crossties, overlapping hoops, or both, show significantly improved performance.[11,12,15,16] It is clear from the comparisons shown in Fig. 1 that the code expressions do not provide adequate representation of experimental observations for the performance criterion for which they were developed.

PROPOSED DESIGN APPROACH

A design procedure is proposed in this paper based on the confinement model developed by the authors.[16] Initially, the performance criterion used in the ACI code is adopted, while recognizing important interactions among the design parameters that are currently overlooked in ACI 318-99.[1] More specifically, the tradeoff between the volumetric ratio and reinforcement arrangement is considered. Tie spacing and

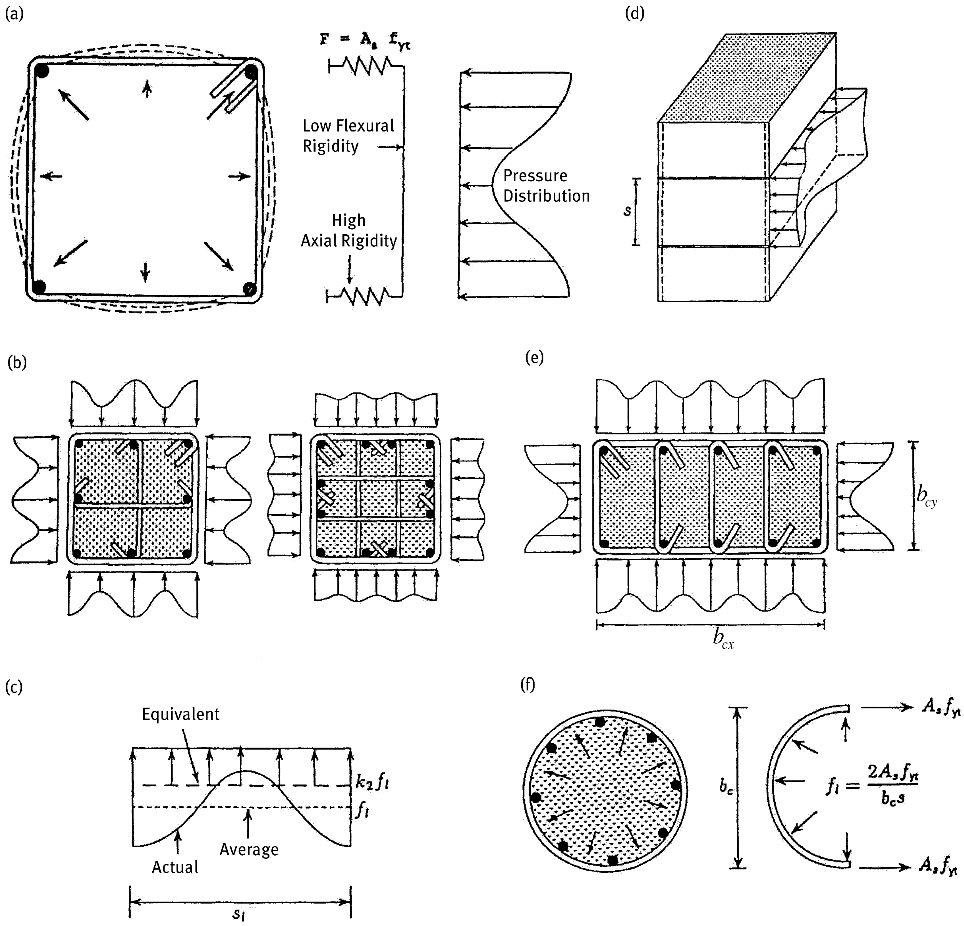


Fig. 2—Development of confinement pressure:[16] (a) and (b) square sections with different arrangements; (c) actual, average, and equivalent pressures; (d) pressure distribution along column height; (e) rectangular section; and (f) circular section.

spacing of laterally supported longitudinal reinforcements are explicitly addressed. Confinement of high-strength concrete, with strength of up to approximately 130 MPa, is included. It is shown that the volumetric ratio of confinement reinforcement can be reduced for columns with efficient tie arrangements. The treatment of square and rectangular columns is particularly improved since the confinement steel requirements are based on a realistic analytical model, reflecting experimental observations rather than an arbitrary extension of the concepts derived for circular columns. The following expressions define strength enhancement in confined concrete based on the analytical model adopted

$$f'_{cc} = f'_{co} + k_1 k_2 f_\ell \tag{10}$$

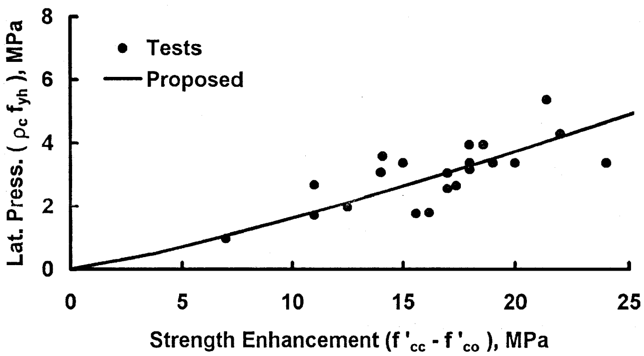
$$f_\ell = \frac{\sum A_s f_{yh}}{s b_c} \tag{11}$$

$$k_1 = 6.7(k_2 f_\ell)^{-0.17} \tag{12}$$

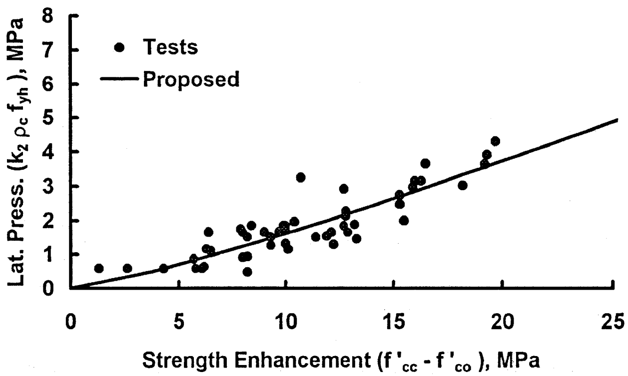
$$k_2 = 0.26\sqrt{\frac{b_c}{s}\frac{b_c}{s_\ell}\frac{1}{f_\ell}} \tag{13}$$

where f_ℓ in Eq. (12) and (13) is in MPa. The in-place strength of concrete f'_{co} may be taken as equal to $0.85f'_c$,as per ACI 318-99,[1] and also as supported by test data. Coefficient k_1 reflects the relationship between uniform lateral confinement pressure and strength enhancement. This coefficient was found to vary with lateral pressure in previous experiments,[6,9,10] differing substantially from the constant value of 4.1 used in deriving the ACI 318 expressions. Coefficient k_2 relates the average lateral pressure f_ℓ to equivalent uniform pressure, and reflects the efficiency of confinement reinforcement. The efficiency improves with the uniformity of confinement pressure and reaches its full value when the lateral pressure is uniform, as is approximately the case in circular columns with closely spaced spirals for which $k_2 = 1.0$.

The passive confinement pressures generated by different arrangements of rectilinear reinforcement are illustrated in Fig. 2. As can be seen, the restraining action against lateral expansion becomes high at locations of cross-reinforcement where overlapping hoops, crossties, or both are tied to the longitudinal reinforcement. Hence, both the tie spacing s as well as the spacing of cross-reinforcement in the cross-sectional plane s_ℓ play important roles in the efficiency of reinforcement arrangement. These variables are incorporated into Eq. (13). A simplified version of the same equation, also applicable to high-strength concrete, has been suggested by the authors and is shown in Eq. (14)[17]



(a) Circular columns



(b) Square columns

Fig. 3—Comparisons of experimental data with proposed equation.

$$k_2 = 0.15\sqrt{\frac{b_c}{s}\frac{b_c}{s_\ell}} \tag{14}$$

Equation (10) through (14) can be used to derive new design expressions while maintaining the performance criterion adopted by ACI 318-99.[1] This criterion requires the concentric capacity of confined column core to be at least equal to the unconfined strength of the entire column section. Ignoring the area of concrete replaced by longitudinal reinforcement, the resulting expression can be written as follows

$$f'_{cc}A_c = f'_{co}A_g \tag{15}$$

$$(0.85f'_c + k_1k_2f_\ell)A_c = 0.85f'_cA_g \tag{16}$$

$$\frac{k_1k_2f_\ell}{0.85f'_c} = \frac{A_g}{A_c} - 1 \tag{17}$$

$$\frac{8(k_2\rho_cf_{yh})^{0.83}}{f'_c} = \frac{A_g}{A_c} - 1 \tag{18}$$

$$\rho_c = 0.0825\frac{(f'_c)^{1.2}}{f_{yh}}\frac{1}{k_2}\left(\frac{A_g}{A_c} - 1\right)^{1.2} \tag{19}$$

Equation (19) incorporates the parameters of confinement that play important roles on axial deformabilty, including the arrangement of reinforcement. It provides the required area ratio of transverse confinement reinforcement in each cross-sectional direction for circular, square, and rectangular sections. For circular spirals as defined in ACI 318-99,[1] k_2 = 1.0. For all other cases, k_2 can be computed using Eq. (14).

The confinement steel requirement specified by Eq. (19) is verified against experimental data in terms of lateral pressure $k_2\rho_cf_{yh}$ and the resulting strength enhancement in Fig. 3. The figure indicates that significant improvements are achieved by the proposed expression over the expressions given in ACI 318-99[1] and shown in Fig. 1.

For the majority of columns in practice, Eq. (19) may be simplified as follows

$$\rho_c = 0.2\frac{f'_c}{f_{yh}}\frac{1}{k_2}\left(\frac{A_g}{A_c} - 1\right) \tag{20}$$

High-strength concrete columns

Recent research on high-strength concrete columns indicates that the strength gain due to confinement is independent of concrete strength, although the percentage of strength gain becomes lower for higher-strength concretes.[18-20] Therefore, high-strength concrete columns require proportionately more confinement to attain deformabilities usually expected from earthquake-resistant columns. Tests on high-strength concrete columns also reveal that higher-grade reinforcement is effective in confining columns.[18-20] It was shown by the authors that the effectiveness of high-grade confinement steel under concentric compression depended on the amount and efficiency of transverse steel, while it also depended on the level of axial compression for columns subjected to lateral load reversals.[19-22] Transverse reinforcement with yield strengths of up to 1000 MPa was found to be effective under monotically increasing concentric compression when confined to conform to Eq. (19).[19,20] The same steel was effective under lateral deformation reversals when the accompanying level of axial load was approximately 40% P_o. The transverse steel was approximately 80% effective when the level of axial load was reduced to 20% P_o, developing approximately 800 MPa stress at peak column resistance.[22] Reinforcement with approximately 600 MPa yield strength was consistently effective in confining high-strength concrete columns. Therefore, until further experimental data become available, it may be prudent to limit the yield strength of transverse reinforcement in Eq. (19) and (20) to 600 MPa, which provides an increase of approximately 50% in the current limit of 400 MPa used in ACI 318.[1]

The applicability of Eq. (19) to high-strength concrete columns is verified against experimental data. Figure 4 shows the comparison of strength enhancement values obtained by tests and by Eq. (19) for both normal-strength and high-strength concrete columns. The test data on high-strength concrete were obtained by different researchers for concrete strengths ranging between 30 and 124 MPa.[20,21,23-28] The lateral pressure for these columns was based on recorded transverse steel stress at peak column resistance f_s whenever available, rather than yield strength, since the high-grade reinforcement used in some of the columns did not necessarily yield, which confirms the validity of the upper limit suggested in the previous paragraph. The comparison indicates good correlations with test data providing experimental evidence on the applicability of Eq. (19) to high-strength concrete columns.

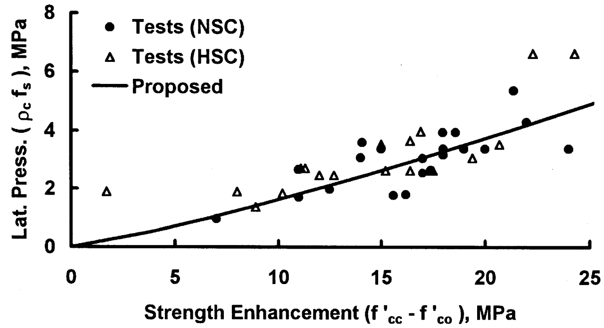
Displacement-based design

The design requirements discussed in the preceding section are based on the axial deformability of columns under concentric compression, and conform to the ACI 318[1] design criterion. This design criterion, however, is not representative of actual column behavior during seismic response. Columns of building and bridge structures experience lateral drift when subjected to seismic excitations. It has been shown by previous research that there is a direct correlation between lateral drift and concrete confinement.[29] Consequently, columns that experience significant lateral drift should be confined more stringently that those that are braced laterally by rigid structural walls. Lateral drift is not explicitly addressed in ACI 318-99[1] for column confinement. Instead, the confinement requirements were developed on the basis of axial deformability, with the implied understanding that columns deformable under concentric compression are also deformable under combined axial and lateral loading. This criterion does not permit the level of axial compression and/or the drift demand to be introduced as design parameters.

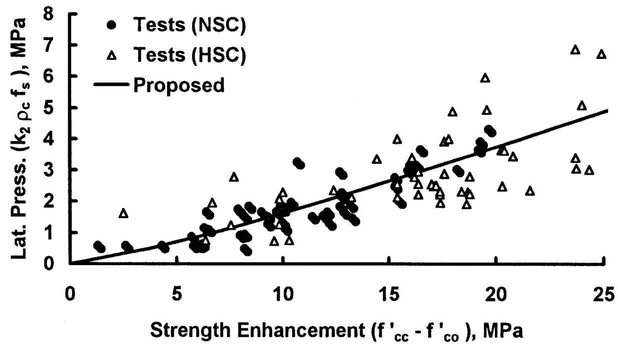
A displacement-based design approach is presented in this section, with lateral drift as the performance criterion. The design approach is based on computed drift capacities of columns with different levels of confinement and axial compression. The computation of drift was done using a computer program for static inelastic loading (pushover analysis)[30] that incorporates analytical models for concrete confinement,[16,17] steel strain hardening,[30] bar buckling,[30] formation and progression of plastic hinging,[29] and anchorage slip[31,32] (extension of reinforcement in the adjoining member). The analysis procedure also includes an option for second-order deformations caused by P-Δ effects. The analytical models, as well as the analysis procedure employed, had been verified extensively against experimental data.[16,17,29,31-33] Figure 3, 4, 5 and 6 illustrate sample comparisons of computed and measured response for the entire range of inelastic force-deformation relationships for both normal-strength and high-strength concrete columns. These sample comparisons, as well as those reported elsewhere,[27,29,37] provide experimental verification of the analysis procedure employed in deriving the design expressions.

The drift capacity was computed either at 20% strength decay in moment resistance or at the same level of decay in lateral force resistance. In the latter case, the decay included the portion that was caused by the P-Δ effect. The use of 20% strength decay as the failure criterion is consistent with that employed by previous researchers, since it is reasonable to accept some strength decay in columns of multistory multibay structures during seismic response before they can be considered to have failed.[3,38]

Extensive parametric investigation was conducted to establish the significance of design parameters on lateral drift.[29,37] The results were used to identify primary design parameters for column confinement while establishing relationships between axial load, confinement reinforcement, and drift capacity. It was concluded that the amount, grade, spacing, and arrangement of confinement reinforcement, as well as the level of axial compression, concrete strength, cover-core area ratio, and shear span-depth ratio played important roles on drift capacity, while the percentage of longitudinal reinforcement played a role of secondary importance. It was further concluded that similar drift capacities could be obtained from columns with similar geometry and reinforcement arrangement but different amounts of



(a) Circular columns



(b) Square columns

Fig. 4—Comparison of normal-strength concrete (NSC) and high-strength concrete (HSC) column tests with proposed equation.

confinement reinforcement and material strengths, as long as the $\rho_c f_{yh}/f_c'$ ratio remained constant, with certain limits placed on these design parameters. This indicates that the $\rho_c f_{yh}/f_c'$ ratio could be used as a design parameter for a wide range of material strengths, including high-strength concrete and high-grade reinforcement. Further verification of this point was done experimentally for concrete strengths up to 124 MPa and steel strengths up to 1000 MPa.[19-22] It was also established that the relationship between the required level of confinement and cover-core area ratio was approximately linear within the practical range of 0.2 to 0.8. Consequently, it was confirmed that columns having a constant $\rho_c f_{yh}/\{f_c'[(A_g/A_c) - 1]\}$ ratio would develop approximately similar drift capacities when other confinement parameters remained constant, irrespective of variations in individual parameters that make up this ratio.[29,37] This ratio, defined as r, is used in establishing the confinement steel requirements, as also used by ACI 318-99.[1]

$$r = \frac{\rho_c f_{yh}}{f_c'\left[\frac{A_g}{A_c} - 1\right]} \tag{21}$$

Figure 7 and 8 illustrate the variation of column drift capacity with coefficient r, defined in Eq. (21) for different levels of axial compression and efficiency of transverse reinforcement k_2. Drift ratios plotted in Fig. 7(a) and 8(a) were determined at 20% strength decay in lateral force capacity; these account for the decay in force resistance caused by the P-Δ effect. Hence, they are lower than those shown in Fig. 7(b) and 8(b), which were determined at 20% strength decay in moment capacity. These figures clearly indicate that

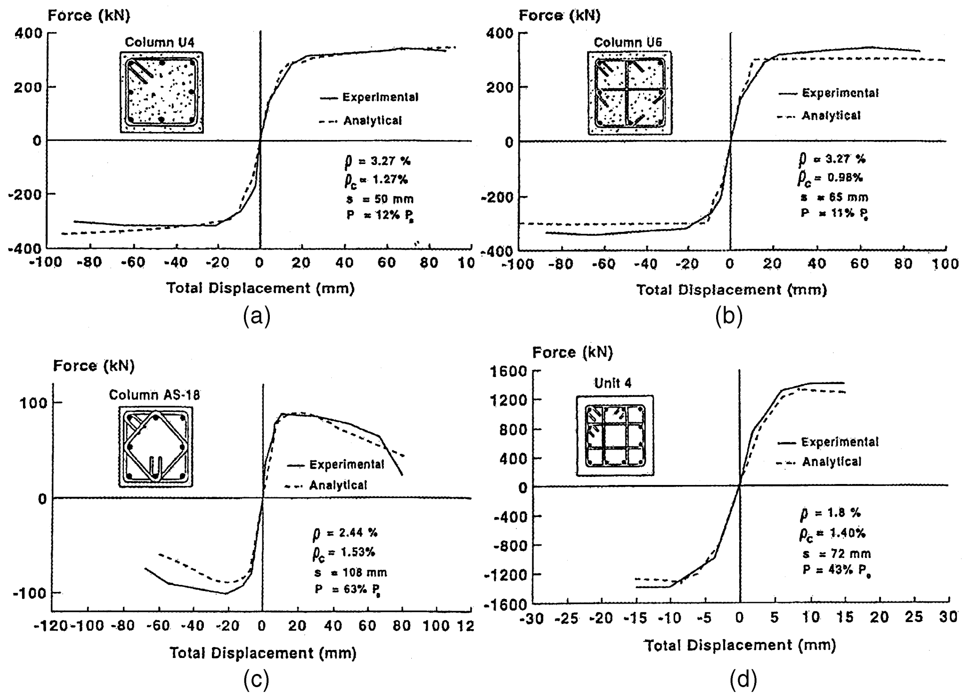


Fig. 5—Normal-strength concrete columns tested by: (a) and (b) Saatcioglu and Ozcebe;[34] (c) Sheikh and Khoury;[35] and (d) Park, Priestley, and Gill.[36]

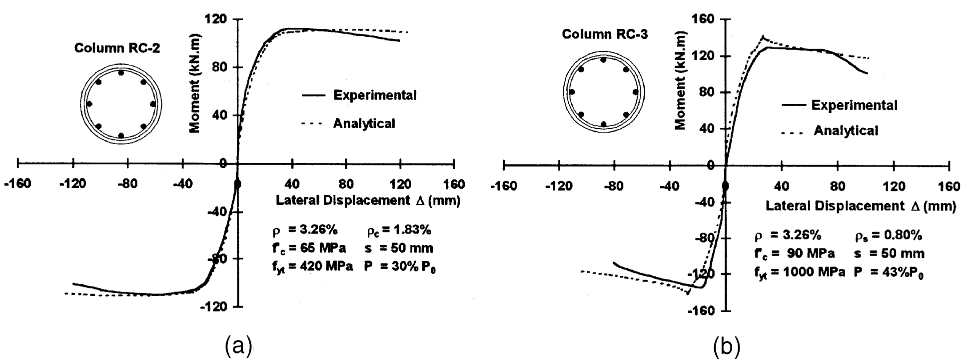


Fig. 6—High-strength concrete columns tested by Saatcioglu and Baingo.[22]

the lateral drift capacity (deformability) improves with increasing values of coefficient r and the efficiency of reinforcement arrangement k_2. They further indicate that the column drift capacity decreases with increasing axial compression. Therefore, higher percentage and/or higher grade and/or improved efficiency of transverse reinforcement are required for columns under higher compression. This implies that the confinement requirements may be relaxed for columns under lower levels of axial compression. Figure 7 and 8 also suggest that the confinement steel requirements should not only be a function of axial load level, but also the arrangement of reinforcement k_2. The relationships given in these figures suggest that the following approximation can be made between r and lateral drift ratio δ

$$r = 14\frac{1}{\sqrt{k_2}}\frac{P}{P_o}\delta \tag{22}$$

Substituting the value of r from Eq. (21) and solving for reinforcement ratio ρ_c

$$\rho_c = 14\frac{f_c'}{f_{yh}}\left[\frac{A_g}{A_c}-1\right]\frac{1}{\sqrt{k_2}}\frac{P}{P_o}\delta \tag{23}$$

Equation (23) relates the confinement parameters to drift capacity δ in the direction of confinement reinforcement when $P \geq 0.2P_o$. Figure 9 illustrates the correlation between drift capacities obtained by Eq. (23) and inelastic pushover analyses. Since the computed drift has been verified extensively against experimental data within the entire range of inelastic drift, as indicated previously and illustrated in Fig. 5 and 6, the computed drift values may be viewed as close representations of experimental values. Figure 9 indicates that Eq. (23) provides a good estimate of column drift capacity. Hence, it can be used to establish the confinement steel requirements of columns with different levels of drift demand.

Figure 7 and 8 were generated for columns with a shear span-depth ratio L/h of 2.5. This level is near the lower end of L/h ratios used in practice. A complete set of analyses was also conducted for columns with L/h = 5.0, representing the

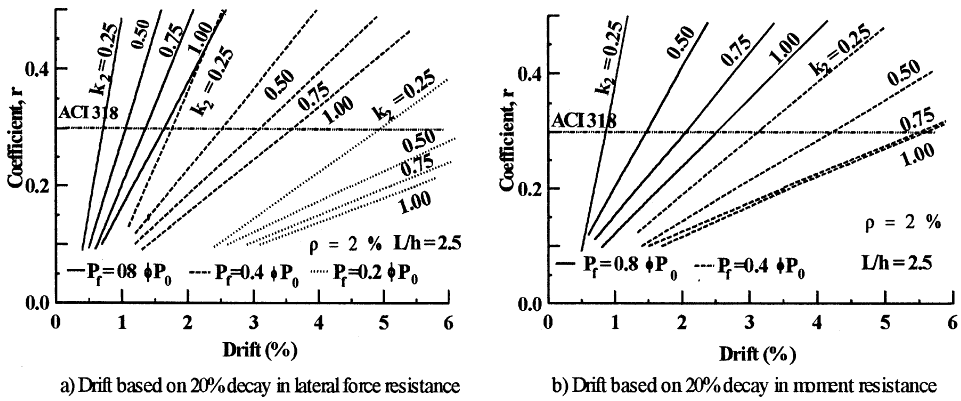


Fig. 7—Variation of drift capacity with confinement coefficient r *in square columns.*

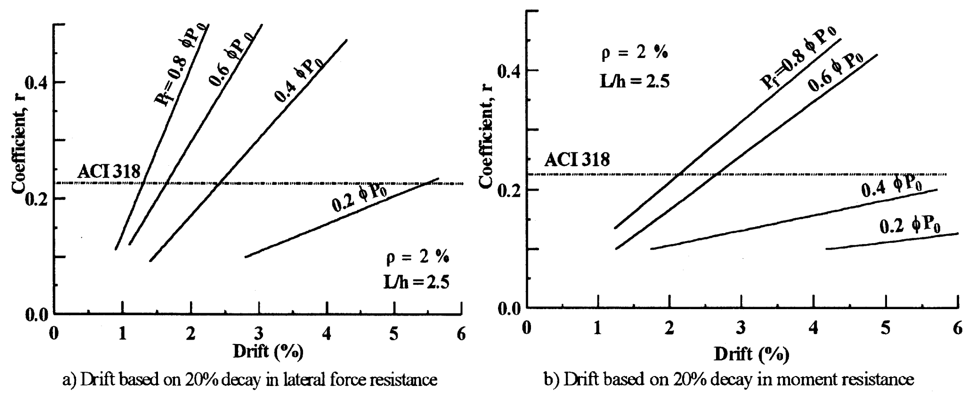


Fig. 8—Variation of drift capacity with confinement coefficient r *in spirally reinforced circular columns.*

higher end of the range used in practice.[37] When the strength decay due to P-Δ effect was included in the analysis, the L/h ratio did not show a pronounced effect on drift capacity. However, when the P-Δ effect was not considered, drift capacities were consistently higher for columns with higher shear span-depth ratios. In the cases considered, the drift capacity increased by approximately 75%, from L/h = 2.5 to L/h = 5.0. For design purposes, it is conservative to consider the aspect ratio that produces lower estimates of drift capacity. Hence, the results for L/h = 2.5 were used in developing the design expression given in Eq. (23).

The percentage of longitudinal reinforcement was also observed to have an influence on drift capacity.[37] This was expected because the increase in longitudinal steel content would increase the contribution of steel as a ductile material to overall column response and produce an increase in column deformability. Column analyses were conducted for 1, 2, and 4% longitudinal reinforcement. The results showed minor variations in drift capacity, with columns having higher percentage of longitudinal reinforcement exhibiting slightly higher drift capacities. The improvement obtained by doubling the amount of reinforcement was approximately 10%. Hence, the longitudinal reinforcement ratio ρ was not included as a parameter for confinement design. Instead, the results for an average reinforcement ratio of 2% were adopted.

The allowable story drift ratio (drift demand) specified by current building codes is limited to 2.0 to 2.5% for most concrete frame structures.[39-41] While Eq. (23) may be used for different drift ratio limits up to 4%, an expression

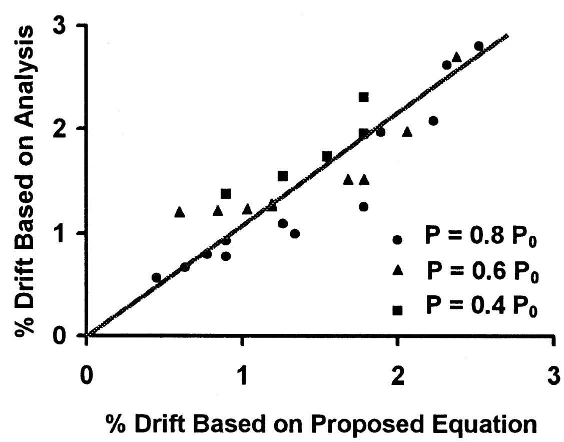


Fig. 9—Correlation of pushover analysis results with Eq. (23).

may be developed for a permissible drift ratio limit of 2.5%. When this drift level is substituted into Eq. (23), and the axial force ratio P/P_o is replaced with $P_u/\phi P_o$, a design expression can be derived as follows

$$\rho_c = 0.35\frac{f_c'}{f_{yh}}\left[\frac{A_g}{A_c}-1\right]\frac{1}{\sqrt{k_2}}\frac{P_u}{\phi P_o} \qquad (24)$$

The axial force P_u in the aforementioned expression represents the maximum axial compressive force that can possibly be applied on the column during a strong earth-

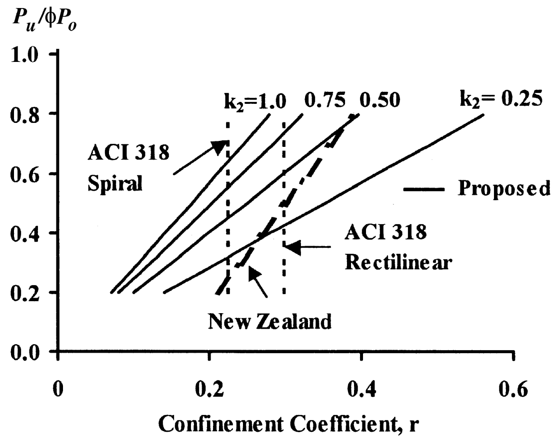


Fig. 10—Comparison of proposed confinement requirements with current American and New Zealand practices.

quake. This quantity corresponds to factored design axial compressive force in ACI 318 design practice.[1] When the capacity design approach is used, as in the case of the New Zealand practice,[42] P_u is computed at the formation of probable moment resistances at the ends of the framing beams when plastic hinges have formed at these locations. The capacity reduction factor ϕ may be taken as 0.90, as opposed to the 0.70 and 0.75 currently recommended for tied and spiral columns in ACI 318 because of the improved ductility of properly confined columns. Equation (24) provides the area ratio of transverse reinforcement required in each cross-sectional direction. For circular spirals, ρ_c remains the area ratio of spiral reinforcement, which is the same in any one direction. The reinforcement ratio requirement given in the same equation approaches zero as the axial compression approaches zero. Therefore, a lower limit is placed on the design axial compressive force, illustrated as follows

$$\frac{P_u}{\phi P_o} \geq 0.2 \tag{25}$$

Furthermore, as discussed previously, it was concluded in the parametric study[37] that the use of the cover-core area ratio as a design parameter had limitations. Therefore, the following limit, also used in ACI 318,[1] may be placed on this ratio

$$\frac{A_g}{A_c} - 1 \geq 0.3 \tag{26}$$

Comparisons with current practice

Equation (24), which is based on the proposed displacement-based design procedure, is compared with the requirements of ACI 318-99[1] and the New Zealand Code NZS 3101.[42] The confinement steel requirements of ACI 318 and NZ 3101 are reproduced as follows in terms of the area ratio of transverse reinforcement ρ_c

ACI 318-99 (spiral)

$$\rho_c = 0.225\frac{f_c'}{f_{yh}}\left[\frac{A_g}{A_c} - 1\right] \tag{27}$$

ACI-318-99 (rectilinear)

$$\rho_c = 0.3\frac{f_c'}{f_{yh}}\left[\frac{A_g}{A_c} - 1\right] \tag{28}$$

NZS 3101 (1982)

$$\rho_c = 0.3\frac{f_c'}{f_{yh}}\left[\frac{A_g}{A_c} - 1\right]\left[0.5 + \frac{1.25P_e}{\phi f_c' A_g}\right] \tag{29}$$

The comparison is made by using the confinement coefficient *r* that is obtained by dividing both sides of the previous equations by $f_c'/f_{yh}[A_g/A_c - 1]$. The axial force ratios of $\{P_u/\phi P_o\}$ and $\{1.25P_e/\phi f_c' A_g\}$ in Eq. (24) and (29), respectively, are approximately equal for a longitudinal column reinforcement ratio of 3% and can be assumed to be the same for the purpose of comparison. Figure 10 provides the comparison of proposed displacement-based design approach with current North American and New Zealand practices. The comparison indicates that the ACI 318[1] approach, which is not a function of the level of axial compression, produces overconservative designs for spirally reinforced columns and some columns with rectilinear reinforcement, especially when the level of axial compression is low. For columns with poor reinforcement arrangement (low k_2 value), the ACI 318 requirements can be unsafe when the axial load level is above approximately 40% of column concentric capacity P_o. The New Zealand approach recognizes the effect of axial load, but ignores the effect of reinforcement arrangement. It produces overconservative designs relative to the proposed approach for columns with superior arrangements of reinforcement (high k_2). Often, overconservative designs translate into the congestion of reinforcement cage and concrete placement problems.

SUMMARY AND CONCLUSIONS

Design expressions were developed for confinement steel requirements of earthquake-resistant concrete columns. Two different performance criteria were adopted for this purpose: 1) the ACI 318 criterion based on axial deformability; and 2) a displacement-based design criterion based on lateral drift. Design expressions were developed for both performance criteria. The expression for the latter criterion is based on static inelastic (pushover) analysis of columns, which was verified experimentally. The proposed expressions incorporate the effects of reinforcement arrangement and higher strength of steel and concrete, and also incorporate the effect of axial force for a displacement-based design. The expressions provide significant improvements over the existing practice, as evidenced by experimental verifications.

NOTATION

A_c = area of core concrete within perimeter transverse reinforcement (center-to-center, except in Eq. (5), (9), (27), and (28), where it is measured out-to-out)
A_g = gross area of column concrete section
A_s = area of longitudinal steel reinforcement, except in Fig. 2 and Eq. (11), where A_s is defined as either A_{sh} or A_{sp}
A_{sh} = area of transverse reinforcement within spacing *s* and perpendicular to dimension h_c
A_{sp} = area of spiral reinforcement
b_c = core dimension, center-to-center of perimeter tie
d_s = diameter of spiral reinforcement
f_c' = concrete cylinder strength
f_{cc}' = strength of confined core concrete
f_{co}' = in-place strength of unconfined concrete in column ($f_{co}' \sim 0.85 f_c'$)
f_ℓ = passive lateral confinement pressure provided by reinforcement
f_s = stress in transverse steel at peak column resistance
f_{yh}, f_{yt} = yield strength of transverse reinforcement

h = column sectional dimension
h_c = core dimension perpendicular to transverse reinforcement under consideration (center-to-center of perimeter reinforcement)
k_1 = lateral pressure coefficient, defined in Eq. (12)
k_2 = confinement efficiency parameter, defined in Eq. (14)
L = column shear span
P = axial compressive force on column
P_e, P_u = maximum axial compressive force on column during earthquake
P_o = nominal concentric compressive capacity of column
r = confinement coefficient, defined in Eq. (21)
s = center-to-center spacing of transverse reinforcement along column height
s_ℓ = center-to-center spacing of longitudinal reinforcement, laterally supported by corner of hoop or hook of crosstie
δ = lateral drift ratio, defined as horizontal displacement divided by height
ϕ = capacity reduction factor
ρ = longitudinal reinforcement ratio
ρ_c = area ratio of transverse confinement reinforcement, $\rho_c = A_{sh}/h_c s$
ρ_s = volumetric ratio of transverse reinforcement

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