Toward the Performance-Based Design of Confined Concrete

by S.A. Sheikh and Y. Li

Synopsis: This paper summarizes results from a comprehensive research program that aims at developing rational guidelines for the design of confinement reinforcement in concrete columns. The first part of the paper briefly introduces an analytical model for confined concrete in tied columns. The model is based on the results of testing 24 square columns with various tie configurations under concentric compression. The second part presents results from square columns tested under cyclic flexure and shear, and constant axial load simulating earthquake loads. The specimens tested included normal-strength concrete (NSC) and high-strength concrete (HSC) columns confined by steel and NSC columns confined by fiber-reinforced polymers (FRP). Performance-based procedures for the design of confinement reinforcement in these columns are proposed in light of the experimental results and analytical models. The design procedures incorporate various ductility parameters that include energy dissipation capacity, ductility factors, and cumulative ductility indices in addition to the type, amount, and configuration of the confinement reinforcement and the level of axial load. The areas in which further research is needed are also discussed.

Keywords: columns; confined concrete; confinement; ductility; earthquake; energy dissipation; fiber-reinforced polymers; lateral reinforcement

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INTRODUCTION

Research in the area of concrete confinement dates back to 1903 when Considère first introduced the use of spirals as confinement reinforcement in concrete columns. Over the last century, a large number of experimental and analytical studies have been carried out to study the behavior of confined concrete. These studies have significantly improved the understanding of confined concrete and resulted in the development of many stress-strain models and design procedures. Despite of these research efforts, how to design and detail confinement reinforcement remains a somewhat puzzling issue. The confinement requirements of the current ACI Code (ACI 318-02) and Canadian Code (CSA A23.3-94) are still based on the philosophy that the axial load carrying capacity of a column should be maintained after spalling of the cover concrete. In practice, however, the confinement is required to produce ductile behavior of the columns subjected to a combination of forces (Sakai and Sheikh 1989). Hence a rational design approach should relate the ductile behavior of a column to the confinement requirements, with due considerations given to those factors that have significant effects on column ductility.

To establish rational guidelines for the design and detailing of confinement reinforcement for confined columns, Sheikh and Uzumeri initiated a comprehensive research program on concrete confinement in 1970s. At the early stage of this program, 24 square columns with various tie configurations were tested under concentric compression, and an analytical stress-strain model was developed for confined concrete in tied columns (Sheikh and Uzumeri 1980; Sheikh and Uzumeri 1982). At the second stage, a series of tests (Sheikh and Yeh 1990; Patel and Sheikh 1992; Sheikh and Khoury 1993; Sheikh et al. 1994) were conducted on tied columns subjected to flexure and axial loads. Based on these results, Sheikh and Khoury (1997) proposed a performance-based procedure for the design of confining steel in tied columns. Bayrak and Sheikh (1997; 1998) further conducted tests on high-strength concrete (HSC) tied columns and proposed modification to this procedure to make it applicable to HSC square columns. Following these efforts, Sheikh and Yau (2002), Iacobucci et al. (2003), and Memon and Sheikh (2002) conducted tests on FRP-confined columns and Li and Sheikh (2003) developed a procedure for the design of confining FRP in square columns following the design philosophy proposed by Sheikh and Khoury (1997).

This paper presents the significant results from this research program following a critical review of the confinement provisions in current North American codes. Some of the areas in which further research is needed are also discussed. Full details of these studies may be seen in the literature.

CODES' PROVISIONS FOR CONFINEMENT

ACI 318-02 Code and CSA A23.3-94 Code

The provisions for confinement reinforcement are similar in both the codes. The basic philosophy behind these provisions is that the increase in strength of the core concrete due to confinement should offset the loss in strength caused by spalling of the cover concrete, thus maintaining the axial load carrying capacity of the columns. For circular columns, both Codes specify that the minimum volumetric ratio of spiral steel, ρ_s, shall be given by the larger amount given by Equations 1 and 2.

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

$$\rho_s = 0.12 f_c' / f_{yh} \tag{2}$$

Eq. (1) was derived on the basis of the strength gain of core concrete due to confinement as suggested by Richart et al. (1929),

$$f_{cc} = f_{cp} + 4.1 f_l, \tag{3}$$

while the lower limit provided by Eq. (2) is mainly applicable to large columns in which A_g/A_c is less than 1.27.

The Codes' requirements for the confinement reinforcement in tied columns are expressed in terms of the total cross sectional area of rectilinear ties, with the implied efficiency of rectilinear ties ranging from 67 to 75 percent of that of spirals. The total cross sectional area of ties is given by the larger amount from Equations 4 and 5:

$$A_{sh} = 0.3 s h_c \left(\frac{A_g}{A_c} - 1\right)\frac{f_c'}{f_{yh}}, \tag{4}$$

$$A_{sh} = 0.09 s h_c f_c' / f_{yh} \tag{5}$$

The confinement requirements of the ACI 318-02 and the CSA A23.3-94 Codes only aim to maintain the axial load capacity of a column section after spalling of the cover concrete. These provisions ignore the most important parameter, ductility, when a column is subjected to seismic loading. Since the primary objective of concrete confinement is to enhance the ductility of columns, the rational design of confinement reinforcement should take into account both strength and ductility. Moreover, many experimental studies (Sheikh and Uzumeri 1980; Sheikh and Yeh 1990; Patel and Sheikh 1992; Sheikh and Khoury 1993; Sheikh et al. 1994) have confirmed that both the level of axial load applied to the column and the steel configuration have great effects on the column behavior. Column ductility decreases with the increase in axial load. Given the same amount of longitudinal and transverse reinforcement and the same level of axial load, column ductility varied significantly from one steel configuration to another. Column sections designed in accordance with the codes' provisions thus display behavior that varies from very ductile to brittle depending on the arrangement of steel and the nature of loads.

BEHAVIOR OF CONFINED CONCRETE

Experimental program

To study the behavior of confined concrete in tied columns, Sheikh and Uzumeri (1980) tested 24 square columns under increasing concentric compression to failure. All columns were 305 mm square and 1.96 m long. Sections were gradually enlarged to 305 × 508 mm at both ends to avoid failure in the end zones. Variables studied were the distribution of longitudinal steel around the core perimeter and the resulting tie configuration, the amount of longitudinal steel, and the amount, spacing, and characteristic of lateral steel. The longitudinal steel content varied between 1.72% and 3.67% of the gross section area, while the amount of lateral steel ranged from 0.76% to 2.39% of the core volume. The core area was 267 × 267 mm measured from the centerline of the exterior tie.

The test results indicated that the distribution of the longitudinal steel around the core perimeter and the resulting tie configuration have a significant effect on the behavior of confined concrete. Strength and ductility of the concrete increased as the number of laterally supported longitudinal bars increased. Moreover, reduction in tie spacing and an increase in the confining steel content resulted in an increase in concrete strength and significant improvement in the ductility.

Sheikh and Uzumeri Model for tied columns

Based on their results, Sheikh and Uzumeri (1982) proposed an analytical model for confined concrete in tied columns. The stress-strain curve for this model is shown in Fig.1, which consists of a second-degree parabola and three straight lines. The curve can be defined completely by four parameters, namely f_{cc}, ε_{s1}, ε_{s2}, and ε_{s85}.

One of the important features of this model is the concept of the concrete effectively confined within the concrete core defined by the centerline of the perimeter ties. As illustrated in Fig. 2, the model assumes that at the tie levels, the separation between the effectively confined concrete and the unconfined concrete is in the form of a series of arcs spanning between the laterally supported bars with an initial tangent slope of 45 degrees. Between tie levels, the area of the effectively confined concrete core reduces and is minimum midway between two tie sets. For square sections with uniformly distributed longitudinal steel, the area of the effectively confined concrete core at the critical section, A_{ec}, can be calculated from

$$A_{ec} = \left(B^2 - \frac{nC^2}{5.5}\right)\left(1 - \frac{s}{2B}\right)^2$$

(6)

The strength gain factor K_s is calculated as

$$K_s = 1.0 + \frac{B^2}{140P_{occ}}\left[\left(1 - \frac{nC^2}{5.5B^2}\right)\left(1 - \frac{s}{2B}\right)^2\right]\sqrt{\rho_s f_s'} \tag{7}$$

Strain values to define the complete stress-strain curve can be calculated from the following equations.

$$\varepsilon_{s1} = 80K_s f_c' \times 10^{-6} \tag{8}$$

$$\varepsilon_{s2} = \left\{1 + \frac{248}{C}\left[1 - 5.0\left(\frac{s}{B}\right)^2\right]\frac{\rho_s f_s'}{\sqrt{f_c'}}\right\}\varepsilon_{oo} \tag{9}$$

$$\varepsilon_{s85} = 0.225\rho_s\sqrt{(B/s)} + \varepsilon_{s2}$$

(10)

According to this model, for the same amount of steel in the column, better distribution of longitudinal steel around the core perimeter and smaller tie spacing would result in larger A_{ec} and higher strength and ductility of concrete. The model was applied to predict the results of the tests reported by various researchers. The comparison between the experimental and analytical results showed good agreement.

PERFORMANCE-BASED DESIGN OF CONFINING STEEL IN COLUMNS

Experimental program

Test setup and procedure — To investigate the behavior of confined concrete columns under earthquake loads, a series of tests have been conducted at the University of Houston and the University of Toronto. The specimens tested included normal-strength concrete (NSC) and HSC columns confined by steel and NSC columns confined by fiber-reinforced polymers (FRP). Fig. 3 shows the test setup used. To facilitate the direct comparison of the column behavior, all the columns tested were 1.47m long with a 510 × 760 × 810 mm stub that represented a beam-column joint or a footing. The square columns had a cross section of 305 × 305 mm while the circular columns had a 356 mm diameter. All specimens were tested horizontally under cyclic shear and flexure while subjected to constant axial load to simulate earthquake loads. The lateral displacement excursion regime consisted of one cycle to a displacement of 0.75 Δ_1 followed by 2 cycles each of Δ_1, 2 Δ_1, 3 Δ_1, and so on until the specimen was unable to sustain the applied axial load. Analytical yield displacement Δ_1 was the lateral deflection corresponding to the estimated maximum lateral load along a load-deflection line that represented the initial stiffness of the column without the effect of axial load.

Ductility Parameters — In evaluating the seismic performance of the columns, 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 of the M-Φ loop.

Design procedure for NSC tied columns

Sheikh and Khoury (1993) and Sheikh et al. (1994) tested eleven square columns under simulated earthquake loads. The variables examined were the level of axial load, the concrete strength, and the amount and configuration of the lateral ties. The details and ductility parameters of the columns are listed in Table 1. From these results, Sheikh and Khoury (1997) found a reasonable correlation between different ductility parameters, as shown in Fig. 5. Data from nine specimens that were tested under similar conditions were used in the construction of this figure. From the best-fit curves, it can be shown that for $\mu_{\Phi 80}$ of 16, the values of $N_{\Phi 80}$ and E_{80} are 64 and 575, respectively. A column section with this level of deformability was defined as highly ductile. With a $\mu_{\Phi 80}$ value of 8 to 16, the section was defined as moderately ductile and the low ductility column has $\mu_{\Phi 80} < 8$. With this correlation between ductility parameters, the results of the columns tested by Sheikh and Yeh (1990) and Patel and Sheikh (1992) that were tested under monotonic flexure, as listed in Table 1, were also used to derive the procedure for the design of confining steel in tied columns with concrete strength up to 55 MPa.

Effect of axial load level — Increased axial load reduces ductility significantly (Sheikh and Yeh 1990; Sheikh and Khoury 1993; Sheikh et al. 1994). 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. For columns with different f_c', however, the comparison using $P/f_c'A_g$ may not remain valid (Sheikh and Khoury 1997). Hence in the design procedure P/P_o instead of $P/f_c'A_g$ was used to measure the level of axial load.

Effect of a change in axial load on the column behavior can be evaluated by comparing the moment-curvature responses of Specimens AS-3 and AS-17 (Fig. 6), which are almost identical in every other regard. Increase in axial load from $0.5P_o$ to $0.63P_o$ resulted in significantly less ductile behavior. Curvature ductility factor $\mu_{\Phi 80}$ was reduced by about 45%.

Steel configuration — The effectiveness of confining steel primarily depends on the area of the effectively confined concrete and the distribution of confining pressure, which, in turn, are highly affected by the distribution of longitudinal and lateral steel and the extent of lateral restraint provided to the bars (Sheikh and Uzumeri 1980; Sheikh and Uzumeri 1982; Sheikh and Yeh 1990). With larger number of longitudinal bars laterally supported by tie bends, the area of effectively confined concrete is increased considerably. Fig. 7 shows the moment-curvature responses of two specimens ES-13 and FS-9. These specimens and Specimen AS-17 are almost identical in all regards except steel configuration. Specimen AS-17 displayed more ductile behavior (see also Table 1) than Specimen FS-9 that in turn is tougher than Specimen ES-13.

Based on this concept and extensive experimental data (Sheikh and Uzumeri 1980; Sheikh and Yeh 1990; Patel and Sheikh 1992; Sheikh and Khoury 1993; Sheikh et al. 1994), Sheikh and Khoury (1997) divided steel configurations into the following three main categories (Fig. 8):

- Category I: Only single-perimeter hoops are used as confining steel
- 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: A minimum of three longitudinal bars are effectively supported by tie corners on each column face and hooks are anchored into the core concrete.

<u>Limiting conditions for steel configurations</u> — Sheikh and Khoury (1997) suggested that for earthquake design, columns should be designed and detailed with high or moderate ductility. Based on the experimental evidence, they suggested that Category I configuration not be used for high ductility columns. 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, tests (Sheikh and Yeh 1990; Sheikh and Khoury 1993; Sheikh et al. 1994) on columns with Section F (Fig. 8) under high levels of axial load showed that the use of 90 degrees hooks not anchored in the core provided sufficient restraint to the middle bars up to a certain stage of loading, but at large deformations the 90 degrees hooks tended to open, resulting in a loss of confinement. Therefore, it was 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 dose 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. 8.

<u>Proposed approach</u> — The relationship between the amount of lateral steel as recommended by the current ACI Code, $A_{sh,c}$, and the suggested amount of lateral steel A_{sh} was taken as:

$$A_{sh} = A_{sh,c} \cdot \alpha \cdot Y_p \cdot Y_\phi \tag{11}$$

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 discussed later in this section. Use of Category II configurations is subjected to the imposed limitations because some of the hooks are not anchored in the core. 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 sufficient ductility is exhibited (Sheikh and Yeh 1990; Sheikh and Khoury 1993; Sheikh et al. 1994). In the event of high axial load levels, the value of α would be much greater than unity; however such an application is not recommended and should be avoided.

With these values of α, Eq. (11) for sections with at least three longitudinal bars restrained on each face ($\alpha = 1$) reduces to

$$A_{sh} / A_{sh,c} = Y_p Y_\phi \tag{12}$$

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

$$Y_p = a_1 + a_2 (P / P_o)^{a_3} \tag{13}$$

$$Y_\phi = b_1 (\mu_\phi)^{b_2} \tag{14}$$

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 was assumed to be unity for highly ductile sections with $\mu_{\Phi 80}$ equal to or greater than 16. Specimens meeting this requirements are AS-3, AS-18, AS-19, AS-20H, A-3, and F-4.Using the results from these specimens, a least square 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 were then determined using the test results for those 16 specimens in which $\alpha = 1$. These included all the specimens with A and D configurations (Category III) and Specimens F-4 and F-12 (Category II) from Table 1. Specimen A-3 was not included in the analysis since its $\mu_{\Phi 80}$ 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 expressions for parameters Y_p and Y_Φ are given below:

$$Y_p = 1 + 13 (P / P_o)^5 \tag{15}$$

$$Y_\phi = (\mu_{\phi 80})^{1.15} / 29 \tag{16}$$

The correlation coefficients for Equations (15) and (16) are 0.99 and 0.93, respectively. The high coefficients indicate excellent agreement between the analytical and the experimental values. Based on the above, the amount of lateral steel in tied columns may be calculated using the following expression.

$$A_{sh} = \alpha\left\{1 + 13\left(\frac{P}{P_o}\right)^5\right\}\frac{\left(\mu_{\phi 80}\right)^{1.15}}{29}A_{sh,c} \tag{17}$$

The simplified versions of the expressions for Y_p and Y_Φ can be taken as

$$Y_p = 6\left(P/P_o\right) - 1.4 \geq 1.0 \tag{18}$$

$$Y_\phi = \mu_{\phi 80}/18 \tag{19}$$

and the required amount of lateral steel may be calculated as

$$A_{sh} = \alpha\left[6\frac{P}{P_o} - 1.4\right]\left[\frac{\mu_{\phi 80}}{18}\right]A_{sh,c} \geq \alpha\frac{\mu_{\phi 80}}{18}A_{sh,c} \tag{20}$$

Factor α is unity for Category III configurations and for Category II configurations as long as the prescribed limiting conditions are met; whereas α value for Category I configurations may be estimated by using the experimental results. Values for α were calculated using Eq. (17) for all the specimens with Configuration E. The average value of α is about 2.70.

The factor α for Category I configurations may also be estimated by adopting the concept of "effectively confined concrete core area" as shown in Fig. 2. The ratio between the area of effectively confined concrete and the total concrete area λ at tie level is given by:

$$\lambda = 1 - \frac{\sum_{i=1}^{n} C_i^2}{5.5B^2} \tag{21}$$

It may be reasonably assumed that the configuration parameter α is proportional to $1/\lambda$. 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. (21). For the specimens in which the longitudinal bars are uniformly distributed around the core perimeter, the λ values for Configurations A and O are 0.636 and 0.273, respectively. Hence, α_I =2.33. 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.

The above procedure, for the sake of simplicity, does not include tie spacing as an active parameter. However, it should be recognized 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 (or 3.4d_b to 7.2d_b). Experimental and theoretical evidence (Sheikh and

Uzumeri 1980; Sheikh and Uzumeri 1982) shows that hoop spacing plays a significant role in the mechanism of confinement. Larger hoop spacing will result in smaller area of effectively confined concrete in the core and may result in premature buckling of longitudinal bars. Hence for a conservative design the limit to the tie spacing is suggested to be the smallest of $B/3$, $6d_b$, and 200 mm.

Design procedure for FRP-confined columns

In recent years, FRP jacketing has emerged as a promising retrofitting technique to provide additional confinement to existing columns due to the lightweight, high strength, and excellent corrosion resisting property of FRP. As this technique has been increasingly used in the field, it is imperative to develop appropriate design procedure for practicing engineers to implement this new technology with confidence.

To investigate of behavior of FRP-confined columns, Iacobucci et al. (2003) and Memon and Sheikh (2002) tested 11 square FRP-confined columns under simulated earthquake loads. The test setup, loading sequences, testing procedures, and configurations of the specimens are similar to those used by Sheikh and Khoury (1993). The specimens were designed to model typical pre-1971 seismic design details and were externally retrofitted by different amounts of continuous carbon fiber-reinforced polymer (CFRP) or glass fiber-reinforced polymer (GFRP) wraps to provide additional confinement in the potential plastic hinge regions. The details and the ductility parameters of these specimens are listed in Table 2. Based on these results, Li and Sheikh (2003) have developed a procedure for the design of confining FRP reinforcement following the design philosophy proposed by Sheikh and Khoury (1997).

Ductility requirements for FRP-confined columns — To develop the ductility requirements for FRP-confined columns, the relationships between different ductility parameters of steel-confined columns are compared with those of FRP-confined columns in Fig. 5. The relationships of FRP-confined columns were constructed using the results of ten FRP-confined specimens as listed in Table 2. From Fig. 5, it can be seen that for a certain value of $\mu_{\Phi 80}$, the values of $N_{\Phi 80}$ and E_{80} of the FRP-confined columns are significantly higher than those of the steel-confined columns, which indicates that for dissipating equal amounts of energy, the curvature ductility factors of the FRP-confined columns are smaller than those of the comparable steel-confined columns. This may be attributed to the different curvature distributions in the plastic hinge regions and different plastic hinge lengths in these two types of columns. For steel-confined columns, the equivalent plastic hinge length is reported to be approximately equal to the dimension of the cross section (Sheikh and Khoury 1993; Sheikh et al. 1994), whereas the equivalent plastic hinge length of most of the FRP-confined columns is larger than the dimension of the cross section (Iacobucci et al. 2003; Memon and Sheikh 2002).

For steel-confined columns, a $\mu_{\Phi 80}$ value of 16 corresponds to $E_{80} = 575$, while a $\mu_{\Phi 80}$ value of 8 corresponds to $E_{80} = 123$. From the best-fit curve for FRP-confined columns, it can be shown that at $E_{80} = 575$, the corresponding value for $\mu_{\Phi 80}$ is 13.2; while at $E_{80} = 123$, the corresponding value for $\mu_{\Phi 80}$ is 8.2. Therefore it is suggested that