

INTERNATIONAL SYMPOSIUM ON CONFINED CONCRETE



Editors:
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PREFACE

In June of 2004, an International Symposium on Confined Concrete (ISCC-2004) was held at Hunan University, Changsha, China. The technical content of the symposium was sponsored by Joint ACI-ASCE Committee 441, Reinforced Concrete Columns. Major sponsoring agencies for ISCC-2004 included the American Concrete Institute (ACI), Pacific Earthquake Engineering Research Center (PEER), the National Natural Science Foundation of China (NSFC), the Japan Concrete Institute (JCI), and the Architectural Institute of Japan (AIJ).

The objective of the symposium was to provide an open forum for researchers around the world to exchange information and to thoroughly discuss topics related to confined concrete modeling, testing, design implementation, and related issues. More than 70 researchers representing nine countries attended the symposium. The symposium attracted more than 60 technical papers, including several keynote and invited papers. All of the papers presented were reviewed in accordance with the policies of the American Concrete Institute, and 28 were accepted for publication in this compilation. The editors thank the reviewers for their professional and timely review opinions. To all those whose submissions could not be included in the special publication, the editors extend the sincere appreciation for their presence at the symposium and their hard work. The cooperation of the authors in the careful and timely revision of their papers in accordance with reviewers' comments is greatly appreciated.

The guidance and assistance of ACI staff in finalizing the special publication are gratefully acknowledged. Particular appreciation is extended to Hunan University for hosting the symposium, and to the University of Southern California and the University of California at Davis for co-organizing the symposium.

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Performance-Based Design and Seismic Analysis

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 Houry (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.

$$\rho_s = 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_{yh}} \quad (1)$$

$$\rho_s = 0.12 f'_c / f_{yh} \quad (2)$$

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

$$f_{cc} = f_{cp} + 4.1 f_t, \quad (3)$$

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:

$$A_{sh} = 0.3 s h_c \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_{yh}} \quad (4)$$

$$A_{sh} = 0.09 s h_c f'_c / f_{yh} \quad (5)$$

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

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

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

$$\epsilon_{s1} = 80K_s f'_c \times 10^{-6} \quad (8)$$

$$\epsilon_{s2} = \left\{ 1 + \frac{248}{C} \left[1 - 5.0 \left(\frac{s}{B} \right)^2 \right] \left[\frac{\rho_s f'_s}{\sqrt{f'_c}} \right] \right\} \epsilon_{oo} \quad (9)$$

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

(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 $510 \times 760 \times 810$ 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 $0.75 \Delta_I$ followed by 2 cycles each of Δ_I , $2 \Delta_I$, $3 \Delta_I$, and so on until the specimen was

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unable to sustain the applied axial load. Analytical yield displacement Δ_l 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.

Limiting conditions for steel configurations — 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.

Proposed approach — 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:

$$A_{sh} = A_{sh,c} \cdot \alpha \cdot Y_p \cdot Y_\phi \quad (11)$$

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

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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

$$A_{sh} / A_{sh,c} = Y_p Y_\phi \quad (12)$$

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

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

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

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:

$$Y_p = 1 + 13 (P / P_o)^5 \quad (15)$$

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

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.

$$A_{sh} = \alpha \left\{ 1 + 13 \left(\frac{P}{P_o} \right)^5 \right\} \frac{(\mu_{\phi 80})^{0.15}}{29} A_{sh,c} \quad (17)$$

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

$$Y_p = 6(P/P_o) - 1.4 \geq 1.0 \quad (18)$$

$$Y_\phi = \mu_{\phi 80} / 18 \quad (19)$$

and the required amount of lateral steel may be calculated as

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

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:

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

It may be reasonably assumed that the configuration parameter α is proportional to $1/\lambda$. Since $\alpha = 1$ 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, $\alpha_I = 2.33$. 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

for FRP-confined columns, the behavior of a column section with $\mu_{\phi 80} = 13$ can be considered as highly ductile. The section with a $\mu_{\phi 80}$ value of 8 to 13 can be defined as moderately ductile and the low ductility column has $\mu_{\phi 80} < 8$.

Amount of confining FRP— The effect of the amount of confining FRP on column ductility can be evaluated by comparing the moment vs. curvature behavior of Specimens ASC-4NS, ASC-3NS, and ASC-5NS, as presented in Fig. 9. These three columns were tested under an axial load of $0.56P_o$ and were confined by 1, 2, 3 layers of CFRP, respectively. As can be seen from Fig. 9, the behavior of the columns improved progressively as the amount of confining CFRP increased. These three columns were able to sustain 8, 11, and 15 load cycles, respectively. The enhancements in curvature ductility of the columns were approximately proportional to the amount of confining FRP provided. Comparisons of the behavior of Specimens ASC-2NS and ASC-6NS that were tested under an axial load of $0.33P_o$ and the behavior of the GFRP-confined columns (Table 2) also lead to the same conclusion.

Axial load level— The effect of the level of axial load on column behavior can be evaluated by comparing the behavior of Specimens ASC-2NS and ASC-4NS (Fig. 9), which were almost identical in every other respect except for the different levels of axial load applied. An increase in axial load from $0.33P_o$ in ASC-2NS to $0.56P_o$ in ASC-4NS resulted in significantly less ductile behavior. Curvature ductility factor was reduced by about 36 percent. Similar observations can be made by comparing the behavior of Specimens ASC-6NS and ASC-3NS and that of Specimens ASG-2NSS and ASG-4NSS.

Proposed approach— From the above discussion, it is evident that for square FRP-confined columns, the column ductility increases as the amount of confining FRP increases, whereas an increase in axial load level reduces column ductility. These effects are similar to those in steel-confined columns. For square columns confined by FRP, the lateral confining pressure provided by the FRP, f_l , can be calculated as

$$f_l = 2 \cdot n \cdot f_{FRP} / h \quad (22)$$

Since the actual average strains of the FRP at the failed sections at column failure are difficult to measure, the corresponding confining pressures at column failure are difficult to obtain. However, from the reported experimental results listed in Table 2, it was found that a reasonable relationship existed between the curvature ductility factors of the columns and the theoretical maximum lateral confining pressure, $f_{l,max}$, provided by the FRP, which can be defined as

$$f_{l,max} = 2 \cdot n \cdot f_u / h \quad (23)$$

The $f_{l,max}$ of the ten specimens were calculated and are listed in Table 2. The relationship between the curvature ductility factor and theoretical maximum lateral confining pressure provided by the FRP is shown in Fig. 10. In this figure, the theoretical maximum lateral confining pressures are normalized with respect to the concrete

compressive strength of the columns, which can be defined as the confinement ratio for the FRP-confined columns.

Fig. 10 indicates that, for columns confined by FRP, the curvature ductility factor, $\mu_{\phi 80}$, increases almost linearly with the increase in the confinement ratio for a certain level of axial load. This relationship can be expressed by the following equation:

$$f_{l,max} / f'_c = \gamma \cdot Y_p \cdot Y_{\phi} \quad (24)$$

Substituting Eq. (23) into Eq. (24) and rearranging the equation give

$$n \cdot f_u = \beta \cdot f'_c \cdot h \cdot Y_p \cdot Y_{\phi} \quad (25)$$

where $\beta = \gamma/2$. In the application of Eq. (25) and the expressions for Y_p and Y_{ϕ} (Eq. [15] and Eq. [16]) for steel-confined columns to the aforementioned FRP-confined columns, it was found that the expressions for Y_p and Y_{ϕ} for steel-confined concrete columns were applicable to FRP-confined columns. Substituting the expressions for Y_p and Y_{ϕ} into Eq.(25) gives the following form of the design equation:

$$n \cdot f_u = \beta \cdot h \cdot f'_c \cdot \left\{ 1 + 13 \left(\frac{P}{P_o} \right)^5 \right\} \cdot \frac{(\mu_{\phi 80,in})^{1.15}}{29} \quad (26)$$

The experimental results of the ten FRP-confined columns were used to calculate the value for β . The average value of β was found to be 0.25 with the standard deviation of 0.03.

Design procedure for HSC tied columns

The procedure developed by Sheikh and Khoury (1997) was derived from the test results of the columns with concrete strength up to 55 MPa. To check the applicability of this procedure to HSC columns, Bayrak and Sheikh (1997; 1998) tested eight HSC and ultrahigh-strength concrete (UHSC) specimens 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). Concrete strength varied between 71.7 and 102.2 MPa. The effectiveness of two tie configurations, i.e. Type A and Type E in Fig. 8, was examined. The details and the ductility parameters of these specimens are listed in Table 3.

Effects of confinement parameters on column behavior — Test results indicate that HSC and UHSC columns with concrete strength up to 102 MPa can be made to behave in a ductile manner under high levels of axial load, provided that sufficient amount of lateral reinforcement is used in an efficient configuration. As in the case of

NSC specimens, an increase in the amount of lateral reinforcement significantly improved the cyclic behavior of HSC and UHSC specimens; whereas an increase in axial load caused substantial reductions in column ductility. The specimens with Type E steel configuration behaved in a considerably less ductile manner than the comparable specimens with Type A configuration. From the results, Bayrak and Sheikh (1998) also found that specimens behaved in similar manner as long as they had the same reinforcement configuration and the same $R_{A/P}$ ratio, which was defined as $R_{A/P} = (A_{sh}/A_{sh,c}) / (P/P_o)$.

Confinement reinforcement requirements for HSC columns — Bayrak and Sheikh (1998) checked the accuracy of the design procedure suggested by Sheikh and Khoury (1997) for the HSC and UHSC test data. It was found that the average of all eight ductility predictions made by Eq. (17) was 37% larger than the average experimental results. Standard deviation from the mean was 29%. To improve the predictions, a regression analysis was performed using the results from the eight specimens listed in Table 3. In the application of Eq. (17) to specimens having concrete strengths varying between 72 and 102 MPa, it was found that constants a_1 , a_2 , and a_3 in Eq. (13) remained the same and constants b_1 and b_2 in Eq. (14) changed. In other words, the effect of axial load on the lateral reinforcement demand is not influenced significantly by concrete strength. Section ductility demand measured by $\mu_{\phi 80}$, on the other hand, is influenced by concrete strength. This is because higher strength concrete specimens have lower deformability and energy absorption capacity initially. These properties improve considerably during the latter part of the displacement excursions. Curvature ductility factor $\mu_{\phi 80}$ does not reflect the total behavior of a columns section. Instead, it focuses only upon the part of a moment-curvature relationship in which the strength loss is smaller than or equal to 20% of the maximum bending moment. For NSC columns b_1 and b_2 were found to be equal to 1/29 and 1.15, respectively. Using the results of the eight specimens in Table 3, a least-square analysis yielded in $b_1 = 1/8.12$ and $b_2 = 0.82$. With these new constants, the average of the predicted curvature ductility ratios is roughly equal to the average of the experimental values, and the standard deviation from the mean is 10%. Furthermore, the value of configuration efficiency factor α for E-type HSC and UHSC columns is calculated to be equal to 1.35. The following equation summarizes the suggested design procedure for the design of confinement reinforcement for HSC columns with f'_c in the range of 55 MPa to over 100MPa:

$$A_{sh} = \alpha \cdot \left\{ 1 + 13 \left(\frac{P}{P_o} \right)^5 \right\} \cdot \frac{(\mu_{\phi 80})^{0.82}}{8.12} \cdot A_{sh,c} \quad (27)$$

RESEARCH NEEDED FOR CIRCULAR COLUMNS

It is generally recognized that the confinement efficiency of circular lateral steel is higher than that of rectilinear ties. Li and Sheikh (2003) conducted a review on the available experimental results on circularly confined columns tested under simulated earthquake loads. This review shows that only limited test data is available in the

literature. Moreover, the test setups, loading histories, instrumentations, and ductility parameters used in the available experimental investigations were different from one program to another, making it difficult to evaluate the results on a common platform. Yau and Sheikh (2002) tested four circular steel-confined columns and eight FRP-confined columns under cyclic flexure and shear, and constant axial load. Two of the steel-confined columns meet the ACI 318-02 Code's confinement requirements and were tested under different levels of axial load. The test results of these two columns indicated that an increase in axial load resulted in reduced ductility and deformability of the columns. The ACI 318-02 Code's confinement requirements for circular columns are not sufficient to provide ductile behavior under an axial load level of $0.54P_o$. A reexamination of these provisions is needed. However, due to the limitations of the available test data, it is difficult to suggest a rational design procedure at this stage. More data is needed to investigate the effects of various confinement parameters on the behavior of circular columns and develop rational design procedure.

SUMMARY AND CONCLUSIONS

The test results from an extensive research program that aims at investigating the behavior of confined concrete columns have been briefly summarized. An analytical model for rectilinear confinement of concrete is briefly presented that highlights the effects of the distribution of longitudinal and lateral steel on column behavior. The paper also outlines procedures for the design of steel and FRP confinement reinforcement in square columns. Design of steel reinforcement is provided for normal as well as high strength concrete columns while only normal strength concrete columns were considered for FRP confinement. The procedures directly relate the confinement requirements to a column's ductile performance, the level of axial load applied, and the configuration of reinforcement. A review of the available research on circularly confined columns suggests that further test data is needed to evaluate the effects of different variables on the performance of these columns, and develop and corroborate analytical models and design procedures.

NOTATIONS

- A_c = cross sectional area of the core concrete, mm^2
- A_{ec} = area of effectively confined concrete core at critical sections, mm^2
- A_g = gross cross-sectional area of column, mm^2
- A_{sh} = total cross-sectional area of rectilinear ties perpendicular to dimension h_c , mm^2
- B = center-to-center distance between the outer legs of perimeter ties, mm
- C = center-to-center distance between laterally supported longitudinal bars, mm
- d_b = diameter of longitudinal bars, mm
- E = energy damage indicator
- f'_c = compressive strength of concrete as measured from standard (150×300 mm) cylinders, MPa
- f_{cc} = compressive strength of confined concrete, MPa
- f_{cp} = compressive strength of plain concrete, MPa
- f_{FRP} = tensile stress of FRP, MPa

f_l	= lateral confining pressure, MPa
f_s'	= stress of lateral reinforcement, MPa
f_y	= yield strength of longitudinal steel, MPa
f_{yh}	= yield strength of lateral steel, MPa
f_u	= ultimate tensile strength of FRP, MPa
h	= cross-sectional dimension of cross section, mm
h_c	= cross-sectional dimension of concrete core measured center-to-center of rectilinear ties, mm
K_s	= strength gain factor
L_f	= length of most damaged region of column, mm
n	= layers of FRP laminates or number of arcs
N_ϕ	= cumulative ductility ratio
P	= applied axial load on column, kN
P_b	= balance load, kN
P_o	= unconfined axial load capacity of column = $0.85f_c'(A_g - A_c) + A_s f_y$, kN
P_{oc}	= $0.85f_c'(B^2 - A_s)$, kN
c	= spacing of rectilinear ties, mm
Y_p	= parameter to take into account the effect of axial load level on column ductility
Y_ϕ	= parameter to take into account section ductility demand
α	= confinement efficiency factor
β	=
ϵ_{oo}	= strain corresponding to the maximum stress in plain concrete
ϵ_s	= strain of confined concrete
ρ_s	= ratio of the volume of total lateral steel to the volume of concrete core

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Table 1 — Details and ductility parameters of NSC steel-confined specimens

Researchers	Specimen	f'_c (MPa)	Lateral steel			$\frac{P}{P_o}$	$\mu_{\Phi 80}$	$N_{\Phi 80}$	E_{80}
			s (mm)	ρ_s (%)	f_{yh} (MPa)				
Sheikh and Khoury (1993)	AS-3	33.2	108	1.68	507	0.50	19.0*	63	610
	FS-9	32.4	95	1.68	507	0.63	8.0	37	154
	ES-13	32.5	114	1.69	464	0.63	6.0	15	53
	AS-17	31.3	108	1.68	507	0.63	12.0	52	402
	AS-18	32.8	108	3.06	464	0.64	17.5	80	897
	AS-19	32.3	108	1.30	498	0.39	19.0	85	631
	F-9H	58.3	95	1.68	507	0.62	5.0	—	—
Sheikh et al. (1994)	E-13H	57.6	114	1.69	464	0.61	2.5	—	—
	AS-3H	54.1	108	1.68	507	0.59	10.5	31	178
	AS-18H	54.7	108	3.06	464	0.61	14.0	43	384
Patel and Sheikh (1992)	AS-20H	53.6	76	4.30	464	0.61	16.5	80	935
	F-9L*	47.0	95	1.68	507	0.58	7.0		
	E-13L	47.9	114	1.69	464	0.60	5.0		
Sheikh and Yeh (1990)	A-17L	49.1	108	1.68	507	0.61	10.3		
	E-2	31.4	114	1.69	483	0.44	10.0		
	A-3	31.8	108	1.68	490	0.49	28.5		
	F-4	32.2	95	1.68	490	0.49	21.3		
	D-5	31.2	114	1.68	490	0.39	20.0		
	F-6	27.2	173	1.68	483	0.58	10.3		
	D-7	26.2	54	1.62	469	0.62	16.0		
	E-8	25.9	127	0.84	483	0.60	3.5		
	F-9	26.5	95	1.68	490	0.59	6.2		
	E-10	26.3	64	1.68	490	0.59	5.2		
	A-11	27.9	108	0.77	469	0.58	8.3		
	F-12	33.4	89	0.82	462	0.50	9.4		
	E-13	27.2	114	1.69	483	0.57	12.3		
	D-14	26.9	108	0.81	462	0.60	7.3		
	D-15	26.9	114	1.68	490	0.60	12.3		
	A-16	33.9	108	0.77	558	0.50	13.2		

• Lightweight aggregate concrete specimens

Table 2 — Details and ductility parameters of NSC FRP-confined specimens

Researchers	Specimen	f'_c (MPa)	Layers & type of FRP	$\frac{P}{P_o}$	$\mu_{\Phi 80}$	Ductility ratio		Energy indicator		$f_{i,max}^*$ (MPa)
						$N_{\Phi 80}$	$N_{\Phi t}$	E_{80}	E_t	
Iacobucci et al. (2003)	AS-1NS ^Δ	31.4	0	0.33	5.3	8	24	11	66	0
	ASC-2NS	36.5	1CFRP	0.33	11.6	61	73	352	466	6.3
	ASC-3NS	36.9	2CFRP	0.56	10.9*	56*	56	326*	326	12.6
	ASC-4NS	36.9	1CFRP	0.56	7.4*	24*	24	79*	79	6.3
	ASC-5NS	37.0	3CFRP	0.56	15.6*	109*	109	1083*	1083	18.9
	ASC-6NS	37.0	2CFRP	0.33	16.7*	161*	161	1328*	1328	12.6
	AS-8NS ^Δ	42.3	0	0.56	2.6*	5.4*	5.4	7.9*	7.9	0
Memon and Sheikh (2002)	AS-1NSS ^Δ	42.4	0	0.56	2.6*	5.4*	5.4	7.9*	7.9	0
	ASG-2NSS	42.5	2GFRP	0.33	11.5	59	79	315	450	7.4
	ASG-3NSS	42.7	4GFRP	0.56	10.6*	55*	55	308*	308	14.8
	ASG-4NSS	43.3	2GFRP	0.56	7.1*	24*	24	97*	97	7.4
	ASG-5NSS	43.7	1GFRP	0.33	10.1	40	47	180	280	3.7
	ASG-6NSS	44.2	6GFRP	0.56	14.7*	135*	135	945*	945	22.2

* Reduction in capacity less than 20% for completed cycles

^Δ Control steel-confined specimens

Table 3 — Details and ductility parameters of HSC steel-confined specimens (Bayrak and Sheikh 1997; 1998)

Specimen	$R_{A/P}$	f'_c (MPa)	Lateral steel			$\frac{P}{P_o}$	$\mu_{\Phi 80}$	$N_{\Phi 80}$	E_{80}
			Size @ spacing (mm)	ρ_t (%)	f_{yh} (MPa)				
ES-1HT	2.3	72.1	15M @ 95	3.15	463	0.50	6.6	19	80
AS-2HT	3.3	71.7	10M @ 90	2.84	542	0.36	15.8	53	631
AS-3HT	2.4	71.8	10M @ 90	2.84	542	0.50	10.1	20	161
AS-4HT	3.7	71.9	15M @ 100	5.12	463	0.50	21.2	84	997
AS-5HT	2.3	101.8	10M @ 90 15M @ 90	4.02	542 463	0.48	9.6	27	144
AS-6HT	3.5	101.9	15M @ 76	6.74	463	0.49	14.0	54	450
AS-7HT	1.7	102.0	10M @ 94	2.72	542	0.48	7.2	14	25
ES-8HT	2.2	102.2	15M @ 70	4.29	463	0.50	6.7	14	33

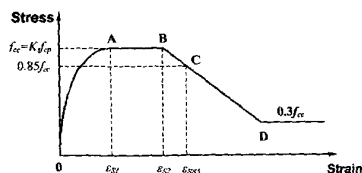


Figure 1 — Sheikh and Uzumeri (1982) model for confined concrete in tied columns

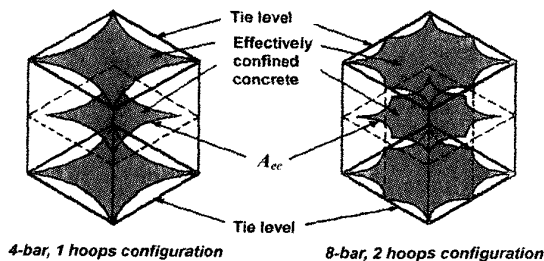


Figure 2 — Concept of effectively confined concrete area

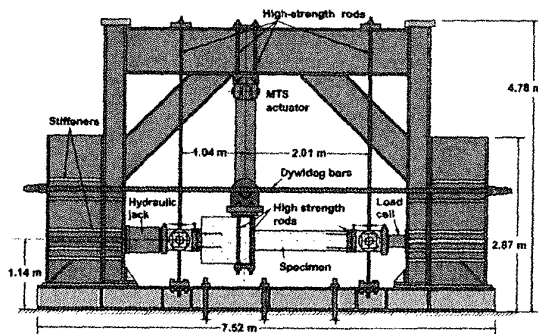


Figure 3 — Schematic of test setup

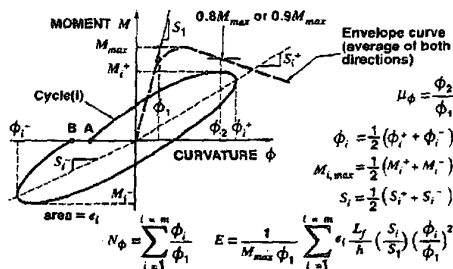


Figure 4 — Ductility parameters

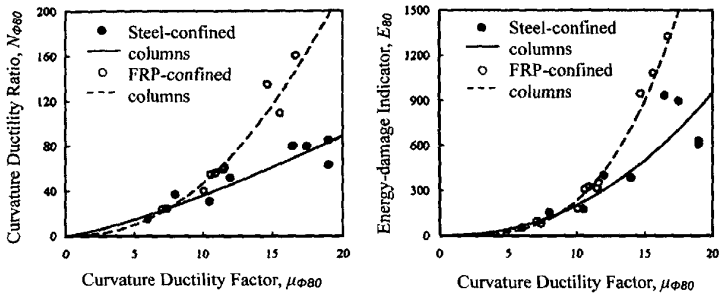


Figure 5 — Relationships between ductility parameters

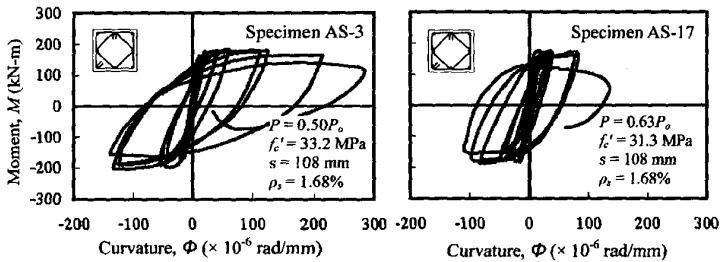


Figure 6 — Behavior of Specimens AS-3 and AS-17

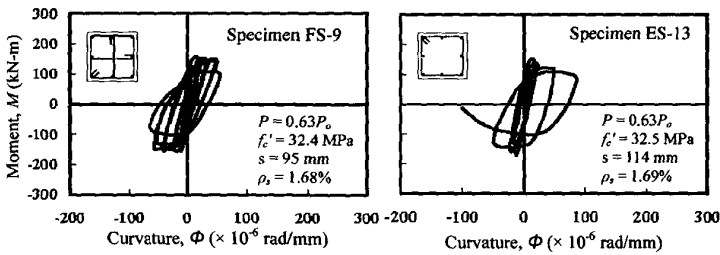


Figure 7 — Behavior of Specimens FS-9 and ES-13

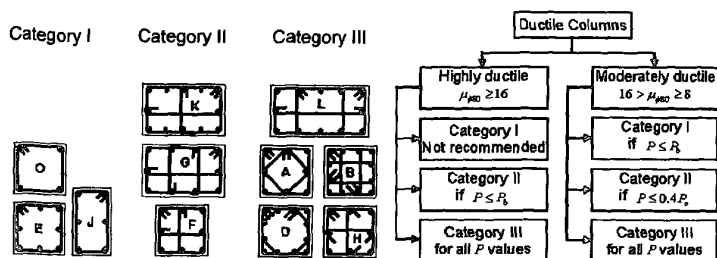


Figure 8 — Categories and limiting conditions for steel configurations

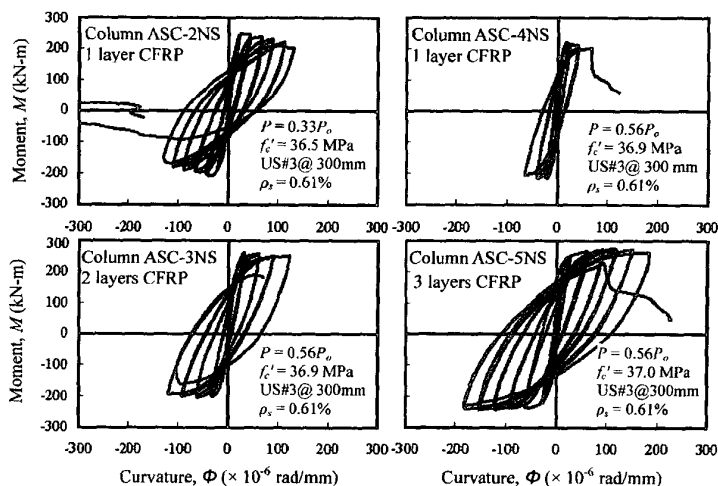


Figure 9 — Behavior of FRP-confined columns

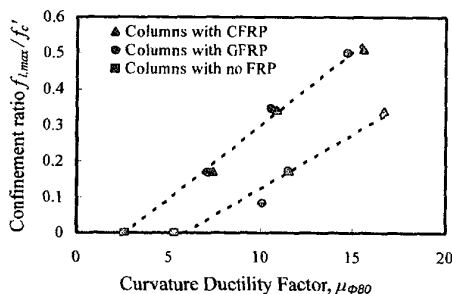


Figure 10 — Relationship between curvature ductility factor and confinement ratio

Seismic Performance of Confined Concrete Bridge Columns

by J.P. Moehle and D.E. Lehman

Synopsis: A current focus in earthquake engineering research and practice is the development of seismic design procedures whose aim is to achieve a specified performance. To implement such procedures, engineers require methods to define damage in terms of engineering criteria. Previous experimental research on bridge columns has focused on component failure, with relatively little attention to other damage states. A research program was undertaken to assess the seismic performance of well-confined, circular-cross-section, reinforced concrete bridge columns at a range of damage states. The test variables included aspect ratio, longitudinal reinforcement ratio, spiral reinforcement ratio, axial load ratio, and the length of the well-confined region adjacent to the zone where plastic hinging is anticipated. The experimental results are used to identify important damage states and to link those states to engineering parameters.

Keywords: bridges; columns; confined concrete; earthquake engineering; performance-based earthquake engineering; reinforced concrete

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INTRODUCTION

Most current seismic design codes do not require the engineer to explicitly assess performance. Instead, seismic design actions are specified through an analysis of the structure, the structure is proportioned to resist those actions, and prescriptive details are provided. Although damage is anticipated in future earthquakes, the extent of damage is not an explicit consideration in design. Furthermore, little attention is paid to performance levels other than life safety. New performance-based seismic design approaches aim to provide more direct consideration of a broader range of performance objectives to meet the needs of individual owners or society.

The Pacific Earthquake Engineering Research Center is developing an integrated methodology for performance-based earthquake engineering [Moehle, 2003], illustrated qualitatively in Figure 1. The methodology begins with representation of seismic demand through an Intensity Measure, simulation of soil-foundation-structure response in terms of Engineering Demand Parameters (such as displacement, acceleration, strain), computation of Damage Measures (such as cracking, spalling, reinforcement fracture) given the engineering demand parameters, and interpretation of the damage in terms of aggregated Decision Variables (repair cost, downtime, casualties) that are relevant to owners and decision-makers. The methodology requires a probabilistic description of each of the components shown in Figure 1, with uncertainties of the entire problem carried consistently from one step to the next.

A key part of the development of performance-based earthquake engineering is to relate damage to engineering demand parameters. An experimental program was developed to improve analytical capabilities for relating damage to engineering demand parameters. The program was limited to circular-cross-section, reinforced concrete bridge columns detailed for ductile flexural response. Damage measures of interest, because they relate to different degrees of repair effort, include crack width, onset of spalling, onset of reinforcement buckling and fracture, and loss of lateral load resistance. This paper describes the study and its principal findings.

EXPERIMENTAL PROGRAM

Overview

The experimental program was designed to obtain performance data for bridge columns having details typical of those currently in use in regions of high seismicity in the United States. The columns had circular cross sections and were reinforced with well-distributed longitudinal reinforcement and closely spaced spiral reinforcement (Figure 2). Columns were fixed to a stiff foundation and were proportioned so that flexure would dominate inelastic response under lateral loading. Column dimensions were selected to represent typical bridge column dimensions scaled to one-third of full scale.

Table 1 lists important properties of the test specimens. Each specimen has an alphanumeric designation. The last two numbers relate to the longitudinal reinforcement ratio while the preceding number(s) relate to the aspect ratio (length/diameter); e.g., Column 415 has aspect ratio of 4 and longitudinal reinforcement ratio of 0.015.

The ten columns are organized in five series (Table 1). Test Series LR included Columns 407, 415, and 430, the main variable being longitudinal reinforcement ratio. (Column 430 was detailed with bundled bars, while the others in this test series were not.) Test Series AR (Columns 415, 815, and 1015) studied effect of varying aspect ratio. Test Series AL (Columns 415 and 415P) had axial loads of $0.1A_g f'_c$ and $0.2A_g f'_c$, where A_g is the gross cross-sectional area of the column and f'_c is the specified concrete compressive strength. Test Series SR (Columns 415 and 415S) examined effects of doubling the spiral spacing. Lastly, Test Series CL included Columns 328T, 828T, and 1028T; this series featured variable spacing of the spiral along the length, with smaller spacing near the column end where inelastic flexural response was anticipated.

Specimen design and materials

Column spiral reinforcement was designed considering recommendations and requirements for shear and confinement (Caltrans 1991, ATC 1996, AASHTO 2000). The confinement requirements of AASHTO, ATC 32, and Caltrans are expressed by Equations (1), (2), and (3), respectively. Table 2 presents spiral reinforcement ratios obtained by these expressions for the test columns. With the exception of Column 415S, all of the columns meet or exceed these provisions.

$$\rho_s = 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_{yh}} \text{ but not less than } \rho_s = 0.12 \frac{f'_c}{f_{yh}} \quad (1)$$

$$\rho_s = 0.16 \frac{f'_c}{f_{yh}} \left(0.5 + \frac{1.25P}{f'_c A_g} \right) + 0.13(\rho_l - 0.01) \quad (2)$$

$$\rho_s = 0.12 \frac{f'_c}{f_{yh}} \left(0.5 + \frac{1.25P}{f'_c A_g} \right) \quad (3)$$

In the preceding expressions ρ_s is the spiral reinforcement ratio, ρ_l is the longitudinal reinforcement ratio, A_c is the cross-sectional area of the concrete core, P is the applied axial load, and f_{yh} is the specified yield strength of the spiral reinforcement.

The ATC 32 provisions require a well-confined region adjacent to expected plastic hinge locations, and allow the spiral reinforcement ratio to decrease outside that region. If column axial load is less than $0.3A_g f'_c$, the length of the well-confined region, L_c , must satisfy Equation (4).

$$L_c \geq \text{minimum of } (0.2L, D) \quad (4)$$

In the expression, L is the column length and D is the column diameter. The three columns of Test Series CL were designed using these recommendations. The well-confined region was equal to the column diameter for the shortest column and 20 percent of the column length for the two taller columns. Outside the well-confined region, the spiral reinforcement was designed to suppress shear failure.

Longitudinal reinforcement met ASTM A 706. Spiral reinforcement was 6 mm in diameter. As this reinforcement was not available with specified yield strength of 414 MPa (60 ksi), ASTM A 82 reinforcement was selected. The 28-day strength of concrete was specified as 28 MPa (4 ksi). Tests were done on 150mm by 300mm cylinders stored with the test columns. Table 3 summarizes the results.

Test details

Figure 3 shows a drawing of a column with an aspect ratio of 4 in the test apparatus. Similar apparatus was used for other tests. Manually controlled jacks applied axial load through a spreader beam attached to the top of a test column. A horizontally aligned actuator at the top of the test column applied the lateral displacement history.

Instrumentation monitored global response quantities (e.g., applied lateral load and displacement) as well as local ones (e.g., steel strains and column segment rotations). Local response instrumentation was concentrated where significant inelastic action was anticipated, and included external instruments to monitor shearing, bending, and expansion of local segments of the column (Figure 3) and foil strain gauges on the longitudinal and spiral reinforcement.

Axial load was maintained constant during testing. Figure 4 shows the target lateral displacement history. The history was based on nominal displacement ductility because histories based on drift or displacement would have resulted in very different inelastic demands for columns of different heights. Three cycles were run at each amplitude, followed by a single cycle at one-third of the preceding amplitude.

OBSERVED PERFORMANCE

The sequence of damage was similar for all columns. The most notable observations, in sequence of first occurrence, were concrete cracking, longitudinal reinforcement yielding, initial spalling of the concrete cover, complete spalling of the concrete cover, spiral fracture, longitudinal reinforcement buckling, and longitudinal reinforcement fracture. These damage states are described in the following text and in the photographs of Figures 5 through 9. The first occurrence of each damage state was identified for each of the force-displacement histories (see example in Figure 10) and values are summarized in Tables 4 and 5.

Cracking and yielding

Initial damage was in the form of horizontal cracks. Initially, crack spacing was equal to approximately half the column diameter (Figure 5). In general, new cracks were not observed for subsequent displacement cycles at the same displacement amplitude; instead, new cracking occurred mainly when displacement amplitude increased.

Initial yielding of longitudinal reinforcement was measured using a strain gauge attached to the extreme longitudinal bar immediately above the column-footing interface. The geometry of a circular-cross-section column is such that initial yielding occurs only in a small number of bars at the extremity of the section; therefore, softening of the load-displacement response occurs only gradually as yielding spreads to adjacent bars around the column circumference. Measured initial yield displacement, Δ_{y1} , is listed in Table 4.

Initial spalling and core damage

Initial spalling of concrete cover was recorded visually at the peaks during the first cycle at a given displacement amplitude (Figure 6). For displacement cycles after spalling but before apparent crushing of the core concrete, the extent of concrete spalling increased with increasing displacement amplitude but remained essentially constant for cycles at constant amplitude. After the core concrete had sustained apparent damage (defined when the inner spiral diameter was visible), cycling at constant amplitude resulted in additional visible damage. Figure 7 shows two columns after the height of spalling had stabilized. Height of the spalled region was greater for taller columns. Table 4 records the onset of initial cover and core spalling.

Bar buckling, bar fracture, and loss of lateral-load-carrying capacity

Typically, longitudinal bar buckling was observed after spalling had fully exposed the spiral and longitudinal reinforcement (Figure 8). The buckled portion of the bar always spanned several spiral spacings. The lateral displacement of the buckled bar increased during subsequent displacement cycles at a given displacement level, and spiral fracture occurred where it had been extended and kinked by the buckled bar (Figure 8).

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The length of the buckled region was similar for all columns and was centered approximately 8 percent of the column height above the base (Table 5).

Fracture of longitudinal bars occurred after bar buckling (Figure 8). The bars did not fracture in Column 430 (bundled bars) or Column 415S (reduced spiral reinforcement ratio), both contrary to expectations. Figure 9 shows conditions at end of testing.

RELATIONS OF DAMAGE AND ENGINEERING DEMAND PARAMETERS

The damage states identified in the preceding section are quantified in terms of engineering demand parameters (Figure 1) in the following paragraphs.

Cracking

Cracking is important in the context of performance-based earthquake engineering because residual cracks may require repair by epoxy injection. Some engineering criteria (e.g., ATC 32 1996) relate residual crack width to maximum strain in the longitudinal reinforcement.

Residual crack widths, w_{res} , were measured for the eight columns of Test Series LR, AR, and CL. Residual crack widths were measured at the end of the three cycles at a displacement level (zero displacement). In most cases, the residual force was small. For Test Series LR and AR, the measured residual crack widths were located approximately 150 mm (6 in.) and 300 mm (12 in.) above the top of the footing. For Test Series LR and AR, the measured residual crack widths were at a height of approximately 300 mm (12 in.). Corresponding maximum longitudinal strains were measured on the extreme bars using foil strain gauges glued to the longitudinal reinforcement.

Figure 11 shows the relation between residual crack width and maximum previous longitudinal reinforcement strain. Figure 12 shows cumulative distributions of residual crack widths exceeding 0.5 mm, 0.13 mm and 0.25 mm. For each curve, the abscissa indicates the percentage of the crack width data that exceed the value indicated in the legend. The corresponding ordinate indicates the maximum measured strain.

Concrete cover spalling and core crushing

Onset of spalling signals a point at which more costly, time-consuming, and disruptive repairs likely will be required. As crushing becomes more severe, damage to core concrete may occur, perhaps requiring more extensive repair measures (Lehman et al. 2001). The extent of spalling along column height also is important, as it determines the minimum length over which confinement by spiral reinforcement is required.

Concrete compressive strains were approximated using external deformation measurements. As shown in Figure 3, steel instrumentation rods passed through the column core at selected elevations. Vertically oriented displacement transducers spanned between rods at one elevation and those at the next and measured deformations along the span between rods, enabling derivation of average rotation, average curvature, and average strain relations between sets of rods. Actual peak local values would be equal to or greater than the averages along the gauge lengths. Table 4 presents the cycle displacement, average strains at the extreme compression fiber of the cross section at spalling, ϵ_{spall} , and the average strains in the core concrete (defined at the inside diameter of the spiral) when core crushing was apparent, ϵ_{core} .

Initial spalling occurred over a wide range of strains (-0.0039 to -0.011), with a mean value of -0.066 and COV of 0.33 (Table 4). ATC 32 (1996) cites an extreme fiber concrete compression strain of -0.004 before spalling, which is approximately equal to the mean minus one standard deviation of the recorded data. Figure 13 presents distribution and cumulative probability curves for the data.

Spalling did not initiate at the interface between the column and footing, but instead at a distance h_{spall} above the interface. The reported strain values correspond to the elevation h_{spall} . Figure 14a shows the relation between spalling strain and h_{spall} . The trend is for ϵ_{spall} to increase with decreasing h_{spall} . One explanation is that footing concrete may be confining column concrete close to the footing. It also is possible that spalling strain was influenced by moment gradient – for steep moment gradients in columns with small aspect ratios, sections with lower strain demand can confine adjacent sections with higher demand. As shown in Figure 14b, columns with higher moment gradients tended to have larger initial spalling strains. Other study parameters such as axial load, reinforcement ratio, and confinement ratio did not have significant influence on spalling strain.

Onset of core damage was defined as the point at which the inside diameter of the spiral reinforcement was fully visible. Strains corresponding to initial crushing of the core concrete are indicated in Table 4. Figure 15 shows the distribution and cumulative probability curves. For the range of the study parameters, there was no apparent effect of axial load, reinforcement ratio, or aspect ratio.

Figure 16 plots maximum spall height (highest location of spalled concrete) as a function of column aspect ratio and axial load ratio. Maximum spall height increases approximately linearly with aspect ratio and increases with axial load. Moehle et al (1996) proposed Equation (5) to define length of confined region.

$$L_c = D, \text{ but not less } \frac{P}{A_g f_c} L \quad (5)$$

The first term recognizes that kinematics requires inelastic response to spread over a length of column related to the diameter. The second term is based on the assumption that confinement is required over full height for pure compression, and a fraction of full

height for lower axial load. Equation (5) is plotted for three axial load ratios in Figure 18. It may be acceptable that Equation (5) specifies confinement over height less than the maximum spall height, as full confinement is not required at the extremity of the spalled region. Reduced confinement is adequate at that height, as demonstrated by the results for Column 1028T

Bar buckling, bar fracture, and loss of lateral-load-carrying capacity

Degradation of column moment strength may adversely affect seismic response and may signal the onset of bridge column failure. For the columns tested, loss of strength resulted from bar buckling, spiral fracture, and longitudinal bar fracture, usually in this order. The onset of this type of damage led rapidly to significant loss of lateral-load strength. An arbitrary but convenient point to represent failure, then, is when lateral-load strength drops by more than 20 percent of the peak value.

Table 5 reports average tensile strain at the center of the extreme longitudinal bar and average compressive strain at the extreme fiber of the core at time of failure. (Column failure was not achieved for Column 828.) Both compressive and tensile strains were obtained using the external instrumentation attached to rods. Average tensile strains were obtained between rods located at $0.25D$ and $0.5D$ from the top of the foundation, where D is the column diameter. The measurement between 0 and $0.25D$ was not used because it primarily measured slip of the reinforcement from the foundation.

Previous research has shown that cyclic history has a stronger influence on bar buckling than on damage states such as initial spalling (Kunnath et al. 1997). This is partly because tensile fracture is influenced by prior buckling, which in turn is influenced by the maximum prior tensile strain in the bar. The displacement histories used in the experimental program described here were intended to be similar for all columns tested. Therefore, the results may not be generally applicable. Consequently, cumulative probability curves are not presented for these damage states.

Some procedures for evaluation and design (e.g., ATC 1996) define failure according to the compressive strain at which the hoop reinforcement ruptures. These procedures are based on models developed from pure compressive tests of confined concrete cross sections in which hoop rupture due to concrete dilation was a predominant failure mode (Mander et al. 1981). Under reversed cyclic loading it is more common for hoop rupture to be controlled by local strains induced as the longitudinal reinforcement buckles and bears against the hoops. New models for compressive strain limits, based on longitudinal reinforcement buckling, are needed.

CONCLUSIONS

Performance-based earthquake engineering requires procedures to translate engineering demand parameters to damage measures, as well as structural details that

deliver required performance efficiently. An experimental study investigated performance of ten spiral-confined, circular-cross-section columns. Primary test variables were column aspect ratio, longitudinal reinforcement ratio, spiral reinforcement ratio, axial load ratio, and length of the well-confined region. The columns were subjected to constant axial load and cyclic lateral loads. Within the limitations of the test program, the following conclusions were reached:

1. The progression of damage was similar for all columns: Concrete cracking, longitudinal reinforcement yielding, cover spalling, core crushing, longitudinal reinforcement buckling, spiral fracture, and (in most cases) longitudinal reinforcement fracture. In all cases, spiral fracture results from longitudinal bar buckling, and not solely from overall dilation of the core concrete.
2. Residual crack widths increased with increasing maximum measured longitudinal reinforcement strains. Residual crack width was not measurable if the strain was less than the yield strain. A fragility curve was presented to define the probability of exceeding a target residual crack width as a function of maximum prior steel strain..
3. Concrete spalling strains ranged from 0.0039 to 0.011 in compression. Lower values were associated with larger column aspect ratios. Axial load ratio, reinforcement ratio, and confinement ratio did not have a significant influence on spalling strain.
4. Height of spalling increased with column aspect ratio and axial load ratio.
5. Crushing of the concrete core was observed for compressive core concrete strains as low as 0.010 and as high as 0.029.
6. Column failure occurred rapidly following onset of longitudinal reinforcement fracture.

ACKNOWLEDGMENTS

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Table 1 - Specimen Properties

Column	Series LR	Series AR	Series AL	Series SR	Series CL	Length (mm)	Reinforcement					Axial Load, $\gamma = P/A_g f_c$
							Longit. bars	ρ **	Spiral Spacing*** (mm)	ρ_{sp} **	L_c **** (mm)	
415	x	x	x	x		2438	22 No. 5	1.5	32	0.68	n/a	0.10
407	x					2438	11 No. 5	0.75	32	0.68	n/a	0.10
430	x					2438	44 No. 5	3.0	32	0.68	n/a	0.10
815		x				4877	22 No. 5	1.5	32	0.68	n/a	0.10
1015		x				6096	22 No. 5	1.5	32	0.68	n/a	0.10
415P			x			2438	22 No. 5	1.5	32	0.68	n/a	0.20
415S				x		2438	22 No. 5	1.5	64	0.34	n/a	0.10
328T					x	1829	28 No. 6	2.7	25 / 50	0.85	610	0.15
828T					x	4877	28 No. 6	2.7	25 / 50	0.85	915	0.15
1028T					x	6096	28 No. 6	2.7	25 / 50	0.85	1220	0.15

* An entry "x" indicates that the column was part of the test series indicated.

** percent.

*** spacing inside well-confined region/spacing outside well-confined region.

**** L_c is the length of the well-confined region. n/a indicates full-length confinement.

Table 2 - Required and Provided Spiral Reinforcement Ratios

Column	AASHTO/ACI	ATC 32	Caltrans	Provided
415, 407, 430, 815, 1015	0.60%	0.66%	0.38%	0.68%
415P	0.60%	0.78%	0.45%	0.68%
415S	0.60%	0.66%	0.38%	0.34%
328T, 828T, 1028T	0.60%	0.89%	0.41%	0.85%

Table 3 - Materials Properties

Column Designation	Concrete	Longitudinal Reinforcement					Spiral Reinf.
	f_{cm} *	f_{ym} *	f_{um} *	ϵ_{yh} **	ϵ_u **	f_{yhm} *	
415, 407	30	497	662	0.02	0.13	607	
430	32	497	662	0.02	0.13	607	
815, 1015	34	497	662	0.02	0.13	607	
415S, 415P	37	462	n/a	n/a	n/a	607	
328T, 828T, 1028T	34	448	634	0.02	0.14	607	

* f_{cm} is concrete compressive strength. f_{ym} and f_{um} are yield and ultimate strengths of longitudinal steel, respectively. f_{yhm} is yield strength of spiral steel. All MPa.

** ϵ_{yh} and ϵ_u are strains at onset of strain-hardening and ultimate strength, respectively.

Table 4 - Concrete Damage Parameters

Column	Δ_p (mm)	Initial Spalling			Initial Core Crushing			L_{spall} ³	L_c/L_{spalls} Eq. (5)
		Δ^1	h_{spall} ²	ϵ_{spall} ⁴	Δ^1	$\%L$ ⁵	ϵ_{core} ⁶		
415	16.1	38	102	-0.0068	127	13	-0.022	305	2.00
407	15.2	38	114	-0.0064	127	10	-0.0207	254	2.40
430	16.8	51	102	-0.011	178	15	-0.017	356	1.71
415S	17.4	38	76	-0.0073	127	17	-0.0086	406	1.50
415P	16.0	38	76	n/r	127	21	N/R	508	1.20
815	56.6	133	127	-0.0074	445	12	-0.0201	584	1.04
1015	76.7	191	229	-0.0039	635	11	-0.0287	660	0.92
328T	13.0	30	190	-0.0057	71	21	-0.0098	381	1.60
828T	59.4	178	127	n/r	445	20	N/R	965	0.76
1028T	97.5	254	559	-0.0043	889	22	-0.0175	1350	0.68
		Mean		-0.0066			-0.018		1.38
		Coefficient of Variation		0.33			0.36		0.41
		Minimum		-0.0039			-0.0098		0.68

¹ Δ = peak displacement for that cycle (mm); ² h_{spall} = height at which initial spalling was observed (mm); ³ L_{spall} = length of spalled region (mm); ⁴ ϵ_{spall} = compressive strain at circumference corresponding to h_{spall} ; ⁵ $\%L = L_{spall}/L$; ⁶ ϵ_{core} = compressive strain corresponding to $L_{spall}/2$; ⁷n/r = data were deemed not reliable.

Table 5 - Data Measured at Bar Buckling and 20% Loss of Load

Column	$h_{buckled}$	Bar buckling ¹ displacement (mm) /cycle	20 % loss displacement (mm) /cycle	ϵ_{comp} ⁴	ϵ_{ten} ⁵
407	102	127 / 1	127 / 3	-0.0279	0.044
415	102	178 / 1	178 / 1	-0.0784	0.0496
430	76	178 / 1	178 / 2	-0.0531	0.0436
415p	102	127 / 2	178 / 1	-0.0476	0.0307
415s	102	127 / 2	127 / 3	-0.0559	0.0482
815	152	445 / 1	445 / 2	-0.0609	0.0512
1015	254	635 / 1	635 / 2	-0.0495	0.0491
328T	76	132 / 1	132 / 2	-0.0631	0.0442
1028T	305	889 / 2	889 / 3	-0.0447	0.0468
Mean				-0.0535	0.045
Standard Deviation				0.014	0.006

¹Bar buckling based on visual observations; ² $h_{buckled}$ = mid-height of buckled bar estimated from measurements made during testing; ³Damage state corresponding to loss of lateral load calculated at peak displacements; ⁴Compression strains from external vertical displacement potentiometers, averaged over length 0D to 0.5D; ⁵Tensile strains from external vertical displacement potentiometers, averaged over length 0.5D to D.

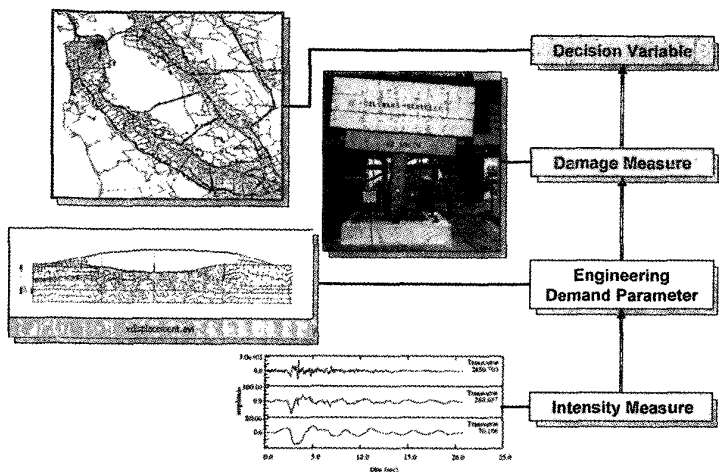


Figure 1 – Performance-based earthquake engineering framework – damage to individual bridges determines the functionality of the bridge network

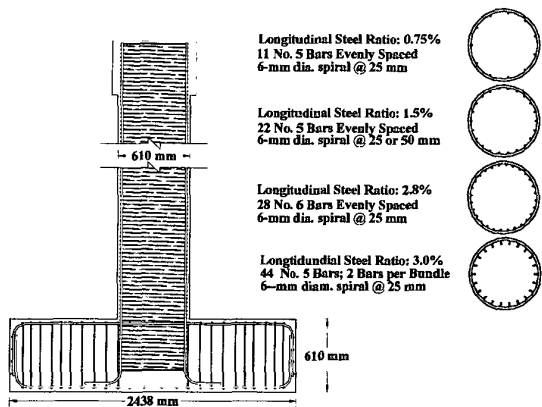


Figure 2 – Column geometry and reinforcement

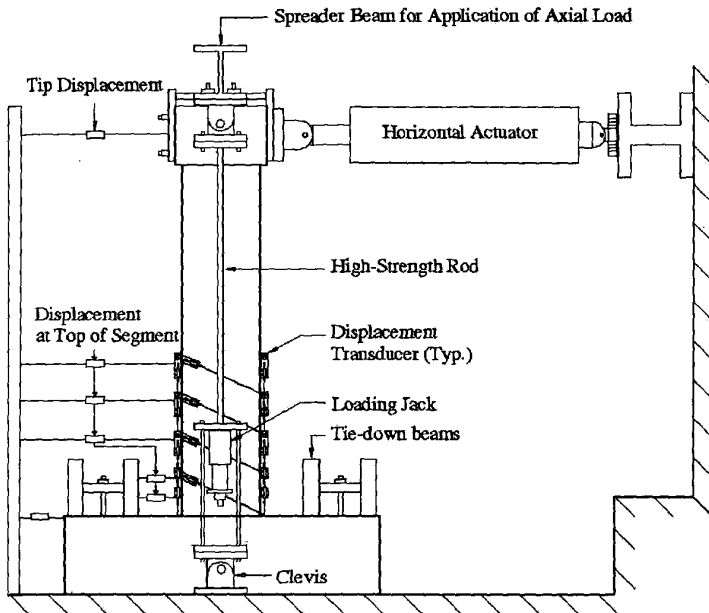


Figure 3 – Test configuration and instrumentation

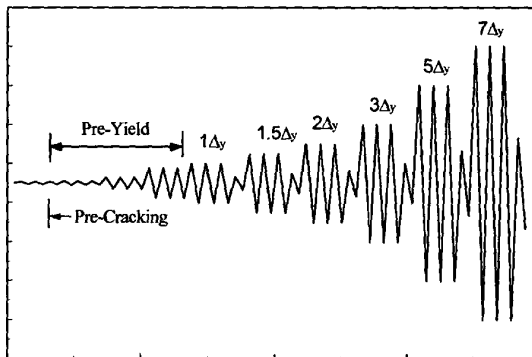


Figure 4 – Target displacement history

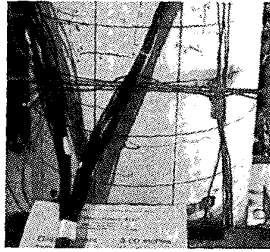


Figure 5 – Crack patterns for 76-mm cycle of Column 407

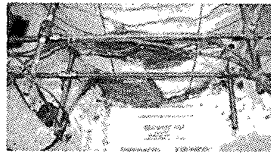


Figure 6 – Spalling at 76-mm cycle for Column 407

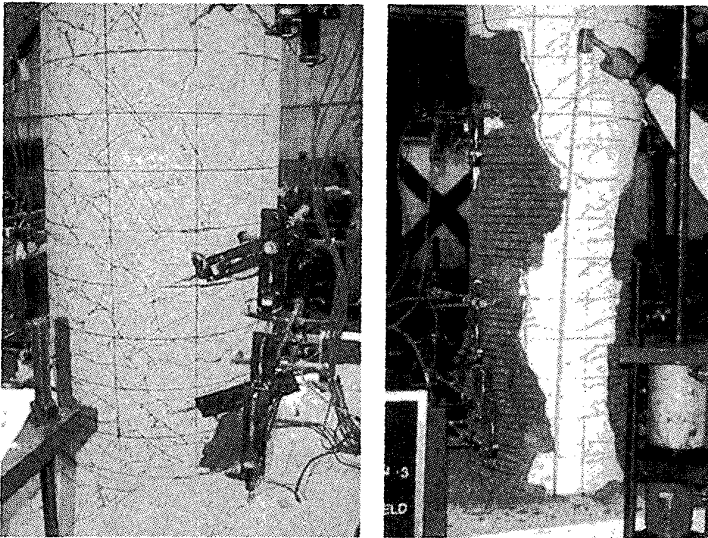


Figure 7 – Spalled regions for Columns 430 (left) and 1028T (right)

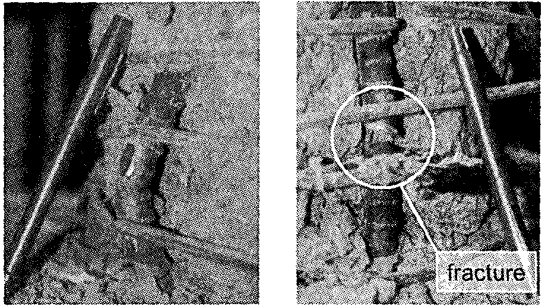


Figure 8 – Rebar buckling (left) and fracture (right) for Column 407

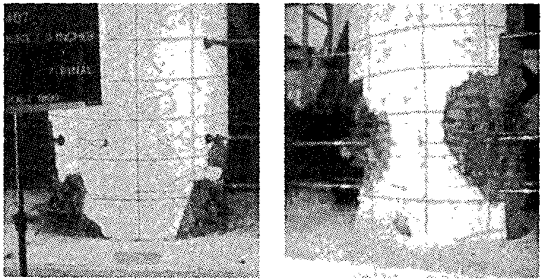


Figure 9 – Final damage states of Columns 407 (left) and 1015 (right)

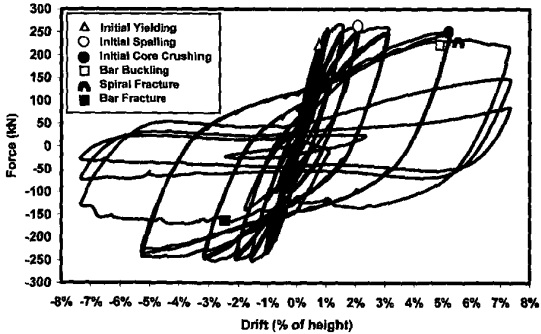


Figure 10 – Force-drift relation, Column 415

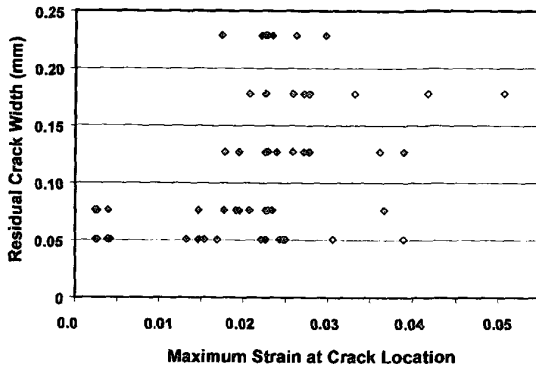


Figure 11 – Residual crack width as function of maximum prior longitudinal reinforcement tension strain at crack location

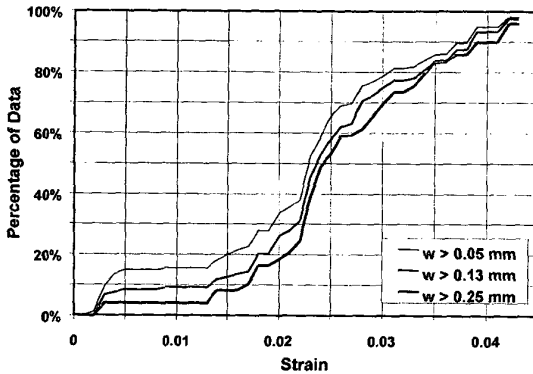


Figure 12 – Cumulative probability relation for residual crack as function of maximum prior longitudinal reinforcement tension strain at crack location

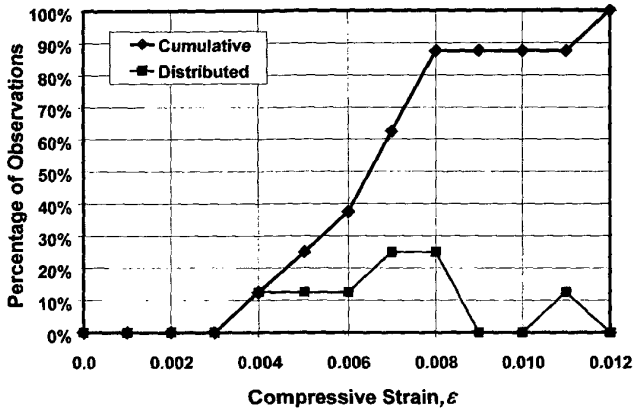


Figure 13 – Distribution and cumulative distribution of compression strain at onset of spalling

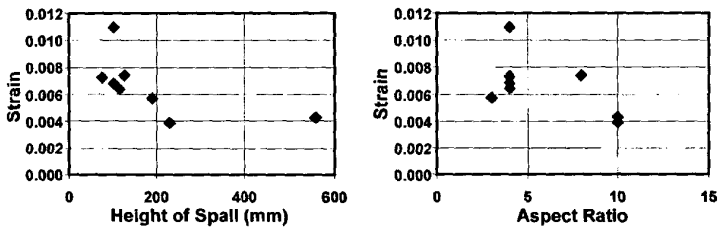


Figure 14 – Dependence of initial spalling strain on height of spall and aspect ratio

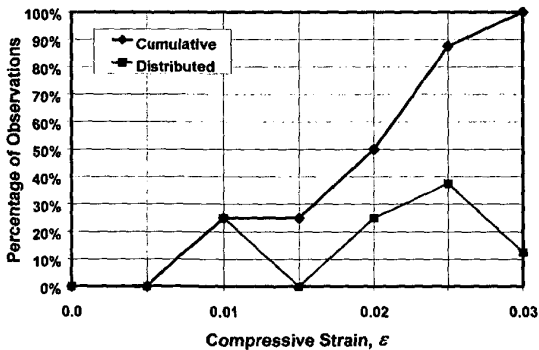


Figure 15 – Distribution and cumulative probability relations for compression strain corresponding to core crushing