Title no. 102-S35

Seismic Design Criteria for Circular Lap-Spliced Reinforced Concrete Bridge Columns Retrofitted with Fiber-Reinforced Polymer Jackets

by Hussein M. Elsanadedy and Medhat A. Haroun

In this study, an object-oriented computer code was developed for predicting the behavior of circular lap-spliced bridge columns retrofitted with advanced composite-material jackets. The numerical model is based on a moment-curvature analysis of the column section with the inclusion of bond-slip mechanism and fiber-reinforced polymer-confined concrete models. The developed software was calibrated through a parametric study comparing the experimental and predicted results for different test data available in the literature. Accordingly, an optimum computational tool was developed to accurately predict the performance of all columns. The emphasis of this paper is on the establishment of a seismic design procedure for circular lap-splice reinforced concrete bridge columns upgraded with fiber-reinforced polymer jackets.

Keywords: bridges; columns; lap splice; reinforced concrete; seismic design.

INTRODUCTION

In many of the tall bridges built in California and designed using pre-1971 guidelines, the longitudinal reinforcement of the bridge columns was spliced with starter bars extending from the column footing with a lap length of 20 bar diameters. In addition, the transverse column reinforcement was typically Grade 40 No. 4 (12.7 mm-diameter) bars spaced at 305 mm (12 in.) on center, independent of column size, strength, or deformation demands. As a result, the transverse reinforcement provides inadequate confinement for the core concrete under compression and insufficient clamping action to the lap splices to prevent debonding. The cover concrete may therefore start to spall prematurely and anchorage of the lapped bars may degrade rapidly due to the splitting action under fully reversed cyclic loads.

To upgrade and retrofit circular bridge columns with poor lap splice details (short lap splices with insufficient hoop reinforcement), various retrofitting means have been developed by researchers and practicing engineers. Steel jacketing has been proven by Chai, Priestley, and Seible[1] to be an effective method to retrofit columns with insufficient lap-splice lengths. Even though steel jacketing has been widely used in practice in California and elsewhere, relatively higher costs are expected due to the complexity during installation. Moreover, the corrosion of the steel jacket could be a potential problem in the future.

Advanced composite materials have been recently recognized and applied to bridge retrofit. The advantages of composite retrofit systems include: light weight, high strength or stiffness-to-weight ratios, corrosion resistance, and, in particular, the ease of installation. All of these advantages make these materials more suitable for retrofitting bridge columns. Moreover, contrary to other retrofit techniques, fiber-reinforced polymer (FRP) jackets will not affect the lateral stiffness of the columns and hence will not alter the bridge dynamic characteristics.

The cyclic performance of circular composite-jacketed bridge columns with poor lap splice details was studied through an experimental program conducted at the University of Southern California.[2] The retrofit system used a series of prefabricated E-glass fiber-reinforced composite cylindrical shells with slits. This system was employed to retrofit circular half-scale bridge columns with insufficient lap splice length at the base, and it proved to be very efficient in terms of clamping on the lap splice region and enhancing the column ductility. Most recently, a comprehensive testing program was carried out at the University of California, Irvine[3,4] to study the seismic performance of circular lap-splice bridge columns retrofitted with advanced composite-material jackets. Eight half-scale circular columns were constructed with a 381 mm (15 in.) lap splice at their base and tested in flexure: two as-built columns and six samples retrofitted by six different composite-material jackets. A brittle failure was observed in the as-built columns due to bond deterioration of the lap-spliced longitudinal reinforcement. The FRP-jacketed circular columns demonstrated a significant improvement in their cyclic performance by reaching ductility greater than 6.0.

For engineers to use FRP for the purpose of seismic upgrade of substandard structural elements, design procedures should be developed. One of the fundamental objectives of this research is to establish practical design criteria for column retrofit by composite-material jackets. The retrofit design methodology of circular bridge columns with insufficient lap-splice length is addressed in this paper.

RESEARCH SIGNIFICANCE

Poor performance of reinforced concrete (RC) columns, primarily due to inadequate lateral reinforcement and insufficient lap-splice length of the starter bars, caused many bridge failures during recent earthquakes. As an effective and economical means for seismic upgrade of lap-splice-deficient columns, this research addresses the use of advanced composite-material jackets. This will provide an insight into promising materials such as fiber composites for various applications in civil engineering.

ACI Structural Journal, V. 102, No. 3, May-June 2005.
MS No. 03-109 received March 12, 2003, and reviewed under Institute publication policies. Pertinent discussion including author's closure, if any, will be published in the March-April 2006 *ACI Structural Journal* if the discussion is received by November 1, 2005.

ACI member **Hussein M. Elsanadedy** *is an assistant professor of civil engineering at Helwan University, Cairo, Egypt. He received his BSc and MSc from Helwan University and his PhD in structural engineering from the University of California, Irvine, Calif., in 2002. He is a member of ACI Committees 370, Short Duration Dynamic and Vibratory Load Effects; 408, Bond and Development of Reinforcement; and 440, Fiber Reinforced Polymer Reinforcement. His research interests include seismic retrofit and rehabilitation of concrete structures using fiber composites, and failure analysis and damage assessment of buildings and bridges.*

ACI member **Medhat A. Haroun** *is AGIP Chair Professor, Department of Construction Engineering, American University, Cairo, Egypt, and Professor Emeritus, Department of Civil and Environmental Engineering, University of California, Irvine. His research interests include theoretical and experimental modeling of the seismic behavior of structural systems such as liquid storage tanks, bridge structures, and buildings.*

ANALYTICAL MODELING

To predict the performance of RC lap-splice columns, an object-oriented computer code (LAP) was developed. The code was written using FORTRAN90 language to run on any personal computer. The program is based on moment-curvature analysis of the column section with the inclusion of a bond-slip mechanism and different concrete confinement models. The ultimate purpose of this program is to provide bridge engineers with a simplified tool that can be used to assess the capacity of circular lap-splice bridge columns with the allowance to try different types of composite-jacket retrofit. The program reads the column data (dimensions, reinforcement details, lap-splice length, concrete strength, yield strength of steel, axial load, and mechanical properties of FRP jacket) via an input text file. Upon running the program, it creates an output text file that has all the analysis results such as moment-curvature calculations, history of bar slip strain, and load-displacement envelopes.

In defining the constitutive properties of the materials, the concrete is of important concern. For as-built columns, the concrete was considered unconfined and Mander's equations for unconfined concrete[5] were used. For composite-jacketed columns, however, confined concrete models were used. Six different confinement models were employed in the analysis. The first was the model developed by Mander, Priestley, and Park,[5] which has been successfully applied to both circular and rectangular steel-jacketed columns. The second model was developed by Samaan, Mirmiran, and Shahawy[6] specifically for circular composite-jacketed columns. The third model, recently developed by Hosotani, Kawashima, and Hoshikuma,[7] is applicable to both circular and rectangular composite-jacketed columns. The fourth model was suggested by Hoppel et al.[8] for circular columns only. Toutanji[9] and Spoelstra and Monti,[10] respectively, developed two other models limited to circular composite-jacketed columns. In addition to defining the stress-strain characteristics of concrete, a model of the stress-strain properties of steel reinforcement was employed. This model was divided into three major zones: a linear portion up to the yield point; a yield plateau region; and a parabolic strain-hardening curve. In the analytical model, bond slip of the lap-spliced longitudinal bars was taken into consideration. Three different bond-slip models were used in the analysis. The first model was developed by Xiao and Rui.[11] The second model is the same as Xiao and Rui's model except that it was modified for columns with Grade 40 steel.[3] The third model was proposed by the authors.

The column was analyzed using a laminar approach. The column cross section was divided into 100 slices, five each in the top and bottom cover and 90 slices in the column core region. The moment-curvature relationship was determined by increasing the strain in the extreme compression fiber of the column section in increments up to the ultimate concrete compressive strain. For each increment, the strain at the extreme tensile steel bar was assumed. The assumption that the plane section remains plane was made so that a linear strain distribution across the column section was assumed. The strain in each concrete layer was computed and, henceforth, the stresses were calculated using the stress-strain relationship of concrete. The force resultant in each concrete layer was determined. For steel bars in compression, the strain at each bar location was computed, and by using the stress-strain model for steel, the stresses were calculated, and hence the force in each bar was determined. Based on Xiao and Rui,[11] it was assumed that for steel bars in tension, the stresses in the starter bars are transferred to the longitudinal bars through a series of shear springs, which are distributed throughout a length of L_b given by

$$L_b = \begin{cases} L_s - 0.022 d_{bl} f_s & (f_s \text{ in MPa; } L_s \text{ and } d_{bl} \text{ in mm}) \\ L_s - 0.15 d_{bl} f_s & (f_s \text{ in ksi; } L_s \text{ and } d_{bl} \text{ in inches}) \end{cases} \quad (1)$$

where L_s is the length of the lap splice; d_{bl} is the diameter of the lap-spliced longitudinal bar; and f_s is the stress of the longitudinal bar in tension. It should be noted that contrary to Xiao and Rui,[11] the length of the bond-links zone defined by Eq. (1) was assumed different for bars at different positions in the column section. It was further assumed that bond stress was uniformly distributed along length of bond links and the nonlinear effects could be ignored. The total geometric strain for each bar in tension consisted of two components: slippage and elongation. By using a bond-slip relationship and the stress-strain model for steel, an iterative procedure was performed for each bar to satisfy equilibrium by

$$4\tau_b L_b - d_{bl} f_s = 0.0 \quad (2)$$

where τ_b is the bond stress between the bar and the surrounding concrete. After satisfying Eq. (2), slippage of each bar in tension can be determined and the stresses can be computed. Hence, the tensile forces for all bars can be estimated. The equilibrium of internal forces for the entire section is assured through use of convergence criteria. Axial force convergence allows both the moment resultants on the column and the section curvature to be determined.

In estimating column lateral displacement, it was assumed to have two components due to flexural and shear deformations. Flexural deformation was computed by integrating the curvature distribution along the column height. For ease, two elastic segments represented curvature distribution up to the first yield of the longitudinal steel: one corresponded to the uncracked concrete section and the other to the cracked section. After yielding, curvature is assumed constant along the plastic hinge length. In single bending of columns with a high aspect ratio, shear deformation is too small, and hence could be ignored.

Details of all models in addition to the numerical procedure can be found elsewhere.[3]

MODEL CALIBRATION

As-built columns

To calibrate the developed numerical models for the prediction of performance of circular as-built lap-splice columns, the experimental results of five columns, tested in a single curvature configuration, were collected from the literature. Data included columns tested by Haroun et al.,[4] Ma,[2] and Chai, Priestley, and Seible.[1] Details of test columns are illustrated in Table 1. It should be noted that in the designation of test samples, the letter "C" denotes circular columns, the letter "F" symbolizes flexural testing, and the letter "A" stands for as-built columns. The computer code developed in this study was calibrated through a parametric analysis to compare between the experimental and predicted results using the three different bond-slip models that were mentioned previously. For all tested columns, the ratio of the maximum experimental lateral load V_{u-exp} to the maximum theoretical lateral load V_{u-th}, the ratio of the ultimate experimental displacement Δ_{u-exp} to the ultimate theoretical displacement Δ_{u-th}, and the ratio of the ultimate experimental ductility $\mu_{\Delta u-exp}$ to the ultimate theoretical ductility $\mu_{\Delta u-th}$ were calculated for the different bond-slip models. It is noted that all bond-slip models give approximately the same good fit to the experimental results of as-built columns. This is shown in Fig. 1 for Sample CF-A2. The load-displacement envelopes, based on modified Xiao's bond-slip model, illustrated the good agreement between the

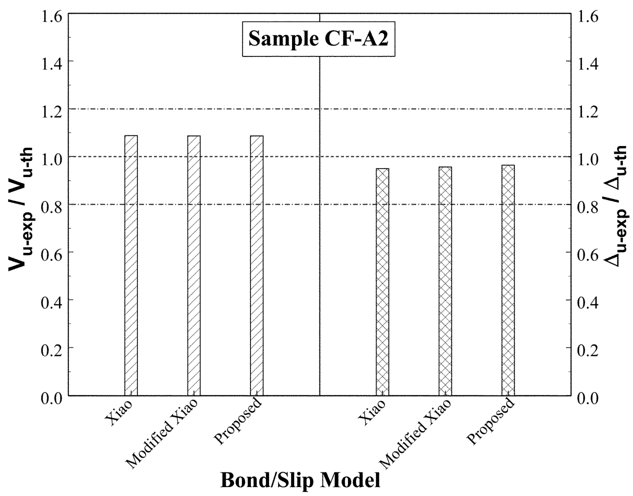

Fig. 1—Experimental and theoretical comparison of flexural response of Sample CF-A2.

Table 1—Details and dimensions of circular as-built columns

Test sample	Column height, m	Column diameter, mm	Lap-splice length, mm	Axial load, KN	Concrete cover,* mm	Longitudinal steel
CF-A1[4]	3.66	610	381	645	25	20 No. 6 (d_{bl} = 19 mm) G40
CF-A2[4]	3.66	610	381	645	25	20 No. 6 (d_{bl} = 19 mm) G40
CF-A3[2]	2.44	610	381	712	38	20 No. 6 (d_{bl} = 19 mm) G60
CF-A4[2]	2.44	610	381	645	38	20 No. 6 (d_{bl} = 19 mm) G40
CF-A5[1]	3.66	610	381	1779	20	26 No. 6 (d_{bl} = 19 mm) G40

*Measured to main steel.
Note: d_{bl} = diameter of longitudinal bar; G40 = Grade 40 steel (nominal yield strength = 276 MPa); and G60 = Grade 60 steel (nominal yield strength = 414 MPa).

experimental and predicted results as evidenced in Fig. 2(a) and (b) for Samples CF-A2 and CF-A3, respectively.

In addition, a statistical study was carried out on tested-to-predicted ratios. For the different bond-slip models, statistical parameters were computed. Based on modified Xiao's bond-slip model, summaries of statistical analysis of the (V_{u-exp}/V_{u-th}), ($\Delta_{u-exp}/\Delta_{u-th}$), and ($\mu_{\Delta u-exp}/\mu_{\Delta u-th}$) ratios are presented in Table 2 for Columns CF-A1 to CF-A5. As demonstrated in the table, the predicted values have good correlation with the experimental results.

As will be discussed later, the first step in seismic retrofit design of lap-splice bridge columns is to analyze force-displacement response of the existing column and compute its ultimate ductility factor. Thereafter, the calculated ductility factor should be compared with the ductility demand estimated from finite element analysis or ductility requirement of current design guidelines. To incorporate safety factors in the design approach, the predicted ultimate ductility for existing bridge columns should be reduced and then compared to the demand value. The situation in seismic retrofit design of bridge columns is very different from gravity load design. In the latter, it is essential that an adequate margin be maintained between strength and applied loads to avoid failure, whereas in the former, keeping a considerable margin between demand and ductility capacity is not necessary. Accordingly, the design safety factor for ultimate ductility is based on statistical lower bounds for ($\mu_{\Delta u-exp}/\mu_{\Delta u-th}$) ratio in terms of minimum value and ($m - \sigma$),

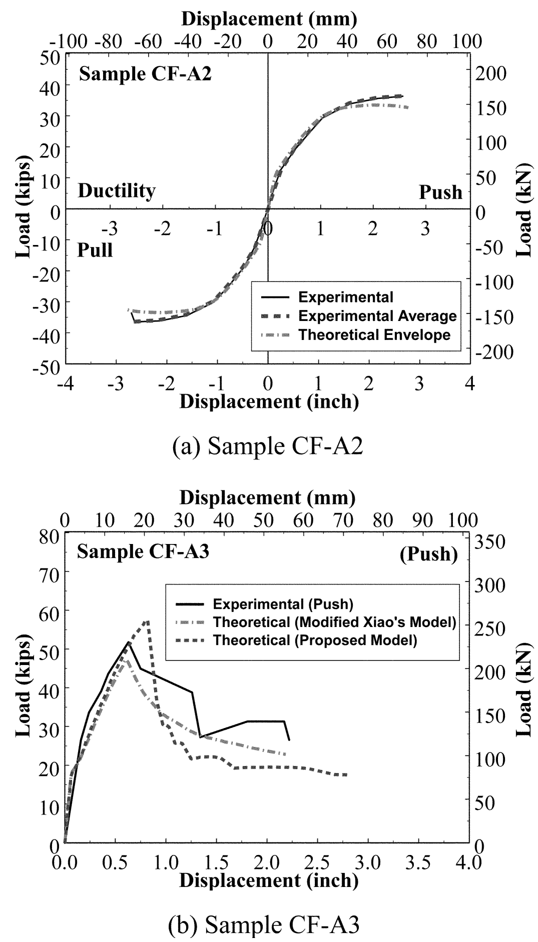

Fig. 2—Comparison between load-displacement envelopes for circular as-built columns.

rather than ($m - 2\sigma$) or ($m - 3\sigma$). The ($m - \sigma$) provides 84.13% probability that the ductility ratio is exceeded.

For circular as-built columns, the design knockdown factor is estimated from statistical analysis of the five samples (CF-A1 to CF-A5). Based on modified Xiao's bond-slip model, statistical lower bounds and recommended design factor of ($\mu_{\Delta u-exp}/\mu_{\Delta u-th}$) ratio are calculated and listed in Table 2.

Retrofitted columns

In addition to the as-built samples, the developed computer code was used to predict the behavior of composite-jacketed columns. For this purpose, the experimental database for 10 retrofitted columns was collected from the literature. Details and dimensions of all 10 columns are presented in Table 3 and 4. It should be noted that in the designation of test samples, "C" denotes circular columns, "F" symbolizes flexural testing, and "R" stands for retrofitted columns. Tests on circular FRP-jacketed columns were conducted by Haroun et al.[4] and Ma.[2] The developed computer code was calibrated through a parametric study to compare between the experimental and predicted results using the three bond-slip models and the six concrete confinement models that were discussed previously. In conclusion, it is noted that modified Xiao's bond-slip model along with Hosotani's model for FRP-confined concrete provided the best fit to the experimental results. Comparing the experimental and predicted results as shown in Fig. 3 for Sample CF-R9 supports this conclusion. Based on these best-fit models, theoretical load-displacement curves were generated and then compared with the experimental results. Examples of comparison between the experimental and predicted load-displacement envelopes are demonstrated in Fig. 4(a) and (b) for samples CF-R4 and CF-R9, respectively.

Table 2—Statistical analysis for circular as-built columns

	(V_{u-exp}/V_{u-th})	($\Delta_{u-exp}/\Delta_{u-th}$)	($\mu_{\Delta u-exp}/\mu_{\Delta u-th}$)
m	1.033	0.979	0.949
σ	0.071	0.181	0.189
$m - \sigma$	0.962	0.798	0.760
Minimum value	0.955	0.835	0.781
Maximum value	1.096	1.289	1.264
Design value	—	—	0.75

Note: m = mean value; σ = standard deviation; and $m - \sigma$ = statistical lower bound.

Table 3—Details and dimensions of circular composite-jacketed columns

Test sample	Column height, m	Column diameter, mm	Lap-splice length, mm	Axial load, KN	Concrete cover,* mm	Longitudinal steel
CF-R1[4]	3.66	610	381	645	25	20 No. 6 (d_{bl} = 19 mm) G40
CF-R2[4]	3.66	610	381	645	25	20 No. 6 (d_{bl} = 19 mm) G40
CF-R3[4]	3.66	610	381	645	25	20 No. 6 (d_{bl} = 19 mm) G40
CF-R4[4]	3.66	610	381	645	25	20 No. 6 (d_{bl} = 19 mm) G40
CF-R5[4]	3.66	610	381	645	25	20 No. 6 (d_{bl} = 19 mm) G40
CF-R6[4]	3.66	610	381	645	25	20 No. 6 (d_{bl} = 19 mm) G40
CF-R7[2]	2.44	610	381	712	38	20 No. 6 (d_{bl} = 19 mm) G60
CF-R8[2]	2.44	610	381	712	38	20 No. 6 (d_{bl} = 19 mm) G60
CF-R9[2]	2.44	610	381	645	38	20 No. 6 (d_{bl} = 19 mm) G40
CF-R10[2]	2.44	610	381	645	38	20 No. 6 (d_{bl} = 19 mm) G40

*Measured to main steel.
Note: d_{bl} = diameter of longitudinal bar; G40 = Grade 40 steel (nominal yield strength = 276 MPa); G60 = Grade 60 steel (nominal yield strength = 414 MPa).

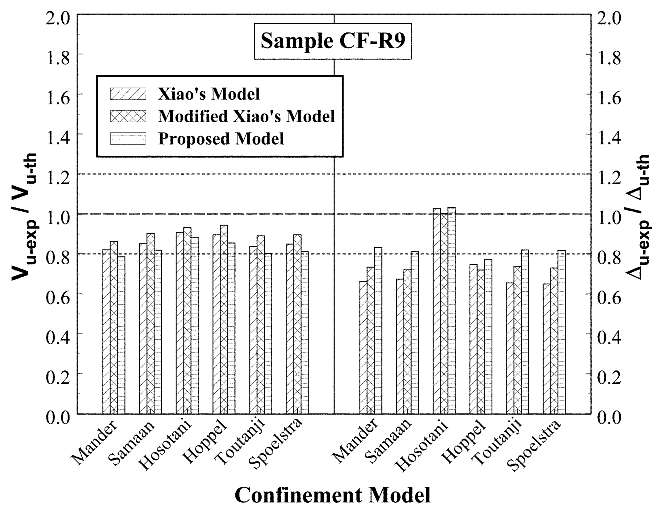

Fig. 3—Experimental and theoretical comparison of flexural response of Sample CF-R9.

Table 4—Material properties of circular composite-jacketed columns

Test sample	Concrete strength, MPa	Yield stress of main steel, MPa	Composite jacket properties: Type	Thickness within lap-splice zone, mm	Tensile strength, MPa	Tensile modulus, GPa
CF-R1	36.0	299.1	Carbon/epoxy	0.7	4168.5	231.5
CF-R2	36.9	299.1	Carbon/epoxy	0.7	4430.3	230.1
CF-R3	32.8	299.1	E-glass/ vinyl ester	11.4	744.1	36.5
CF-R4	37.7	299.1	Carbon/epoxy	1.7	4382.0	226.0
CF-R5	39.7	299.1	E-glass/ polyester	12.7	640.8	36.4
CF-R6	33.1	299.1	Carbon/epoxy	8.3	937.0	63.0
CF-R7	44.8	461.6	E-glass/ polyester	12.7	551.2	48.2
CF-R8	44.8	461.6	E-glass/ polyester	15.9	551.2	48.2
CF-R9	31.0	303.2	E-glass/ polyester	12.7	689.0	37.9
CF-R10*	31.0	303.2	E-glass/ polyester	12.7	689.0	37.9

*This sample is same as CF-R9 except it had continuous composite shell instead of individual shells.

A statistical study was carried out on (V_{u-exp}/V_{u-th}), $(\Delta_{u-exp}/\Delta_{u-th})$, and $(\mu_{\Delta u-exp}/\mu_{\Delta u-th})$ ratios. For the different confinement and bond-slip models, statistical parameters were computed for the tested-to-calculated ratios. It should be noted that Samples CF-R1 and CF-R2 were excluded from the statistical study because they were designed incorrectly by the manufacturer according to a jacket strain of 0.004 instead of 0.001 as required by Caltrans.[12] Based on the best-fit models, summaries of statistical analysis of the (V_{u-exp}/V_{u-th}), $(\Delta_{u-exp}/\Delta_{u-th})$, and $(\mu_{\Delta u-exp}/\mu_{\Delta u-th})$ ratios are listed in Table 5 for Samples CF-R3 to CF-R10.

Seismic retrofit design of lap-splice columns will be based on a demand ductility that is set by current seismic design codes. The capacity-to-demand ratio should be always greater than 1.0. To implement safety factors in the retrofit design methodology, the predicted ultimate displacement and ductility for the retrofitted bridge column should be reduced and then compared to the demand values. Accordingly, retrofit design safety factors for both ultimate displacement and ultimate ductility should be knockdown coefficients, which, of course, have to be based on statistical lower bounds for $(\Delta_{u-exp}/\Delta_{u-th})$ and $(\mu_{\Delta u-exp}/\mu_{\Delta u-th})$ ratios. Retrofit design factors for both ultimate displacement and ultimate ductility are based on statistical lower bounds for $(\Delta_{u-exp}/\Delta_{u-th})$, and $(\mu_{\Delta u-exp}/\mu_{\Delta u-th})$ ratios in terms of minimum value and $(m - \sigma)$.

Following its design for ductility capacity-to-demand ratio greater than 1.0, the retrofitted lap-splice bridge column should be protected against unfavorable brittle modes such as shear friction failure at the column base. Accordingly, the demand lateral load should correspond to maximum feasible extreme estimates of flexural strength developing at the plastic hinge location after column retrofit. Therefore, the retrofit design factor for maximum lateral load should be based on statistical upper bounds for (V_{u-exp}/V_{u-th}) ratio in terms of maximum value and $(m + \sigma)$.

Based on modified Xiao's bond-slip model along with Hosotani's confinement model, statistical lower and upper bounds of (V_{u-exp}/V_{u-th}), $(\Delta_{u-exp}/\Delta_{u-th})$, and $(\mu_{\Delta u-exp}/\mu_{\Delta u-th})$ ratios were computed and presented in Table 5 for circular composite-jacketed columns. In addition, recommended retrofit design factors are listed in Table 5.

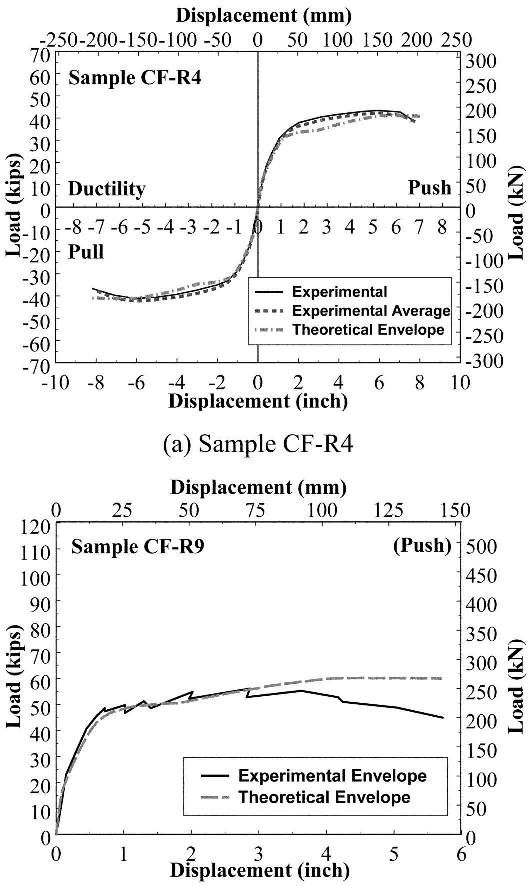

Fig. 4—Comparison between load-displacement envelopes for circular composite-jacketed columns.

Table 5—Statistical analysis for circular composite-jacketed columns

	(V_{u-exp}/V_{u-th})	$(\Delta_{u-exp}/\Delta_{u-th})$	$(\mu_{\Delta u-exp}/\mu_{\Delta u-th})$
m	0.98	0.96	0.99
σ	0.05	0.17	0.13
$m - \sigma$	0.93	0.79	0.86
Minimum value	0.90	0.74	0.75
$m + \sigma$	1.03	1.13	1.12
Maximum value	1.05	1.22	1.17
Design value	1.05	0.75	0.75

Note: m = mean value; σ = standard deviation; $m - \sigma$ = statistical lower bound.

RETROFIT DESIGN METHODOLOGY

The following methodology is proposed for seismic retrofit design of circular reinforced concrete columns with poor lap-splice details using FRP jackets. The retrofit design methodology is divided into five major steps: the first is to attain the design properties of the materials as discussed in a companion paper by the authors;[13] the second is to seismically assess the existing column. The third step involves the design of the FRP jacket for flexural ductility enhancement as the most stringent of requirement for confinement of the compression concrete within the plastic hinge zone, anti-buckling constraint, and requirement for clamping on the lap-splice region. Step 4 involves checking jacket design to mitigate high diagonal compression stress levels in the jacketed column and to preclude shear-friction failure at the column base. The last step is to design the extent of the FRP jacket. These steps are detailed as follows.

Seismic assessment of existing columns

The developed computer code (LAP) is used to seismically assess existing lap-splice bridge columns. Using modified Xiao's bond-slip model, the column is analyzed to come up with its ultimate ductility capacity $(\mu^{calc}_{\Delta(capacity)})$. Thereafter, this ductility is reduced to get the dependable ductility capacity of the existing column from

$$\mu^{Red}_{\Delta(capacity)} = \varphi_\mu \mu^{calc}_{\Delta(capacity)} \tag{3}$$

where φ_μ = ductility knockdown factor for existing columns, derived previously in Table 2 as 0.75. Subsequently, the reduced ductility capacity of the existing column is compared with the ductility demand estimated from finite element method or ductility requirement of current design guidelines. If the reduced ductility capacity exceeds the demand, then no retrofit is needed. Otherwise, the column should be seismically retrofitted according to the following procedure. Full details of the seismic assessment procedure are reported elsewhere.[3]

Jacket design for flexural ductility enhancement

Concrete confinement—The first step in the retrofit design for concrete confinement is to compute the required ductility capacity ($\mu^R_{\Delta(capacity)}$) from

$$\mu^R_{\Delta(capacity)} = \frac{\mu_{\Delta(demand)}}{\varphi_\mu} \tag{4}$$

where $\mu_{\Delta(demand)}$ is the demand ductility, and φ_μ is the ductility knockdown factor obtained previously in Table 5 as 0.75. The maximum required displacement is then determined from

$$\Delta_u = \mu^R_{\Delta(capacity)}\Delta_y \tag{5}$$

where Δ_y = idealized yield displacement for existing column (obtained from analysis of existing column using the program LAP). Thereafter, the required plastic displacement is computed from

$$\Delta_p = \Delta_u - \Delta_y \tag{6}$$

If the plastic hinge is assumed to be centered at the bottom of the column to account for strain penetration into the footing, the required plastic displacement can be expressed by

$$\Delta_p = \theta_p L_c = \Phi_p L_p L_c = (\Phi_u - \Phi_y) L_p L_c \tag{7}$$

where Φ_y = idealized yield curvature for existing column (obtained from analysis of existing column using the program LAP); L_c = column height; and L_p = plastic hinge length of jacketed column, given by Priestley, Seible, and Calvi[14] as

$$L_p = \begin{cases} g + 0.044 f_{ye} d_{bl} & (f_{ye} \text{ in MPa; } L_s \text{ and } d_{bl} \text{ in mm}) \\ g + 0.3 f_{ye} d_{bl} & (f_{ye} \text{ in ksi; } L_s \text{ and } d_{bl} \text{ in inches}) \end{cases} \tag{8}$$

where g = gap between the jacket and the footing; f_{ye} = expected yield stress of reinforcing steel; and d_{bl} = diameter of longitudinal bar. Then, the required ultimate curvature is obtained from

$$\Phi_u = \frac{\Delta_p}{L_p L_c} + \Phi_y \tag{9}$$

The neutral axis depth at ultimate response of the retrofitted column is calculated from

$$c_{u(ret)} = k_r c_{u(existing)} \tag{10}$$

where $c_{u(existing)}$ is the neutral axis depth at ultimate response of the existing column (obtained from analysis of existing column using the program LAP), and k_r is a reduction factor estimated from Eq. (11). It should be noted that the following equation was proposed after analyzing circular lap-splice bridge columns with different axial load ratios using the program LAP.

$$k_r = \begin{cases} 0.90 \text{ for } 0 \le \dfrac{P}{A_g f'_{ce}} < 0.15 \\ 0.85 \text{ for } 0.15 \le \dfrac{P}{A_g f'_{ce}} < 0.30 \\ 0.80 \text{ for } 0.30 \le \dfrac{P}{A_g f'_{ce}} \end{cases} \tag{11}$$

where P = axial force; A_g = gross area of column section; and f'_{ce} = expected concrete strength. The maximum required compression strain is then given by

$$\varepsilon_{cu} = \Phi_u c_{u(ret)} \tag{12}$$

The required volumetric ratio of FRP jacket for concrete confinement ρ_{j1} is computed using Hosotani's confinement model for circular columns. It is given by

$$\rho_{j1} = \frac{21.15 f'_{ce} (\varepsilon_{cu} - 0.00383)^{4/3}}{f_{ju(design)} \varepsilon^{2/3}_{ju(design)}} \tag{13}$$

where $f_{ju(design)}$ and $\varepsilon_{ju(design)}$ are design tensile strength and design ultimate tensile strain of FRP jacket, respectively.

Anti-buckling requirements—The volumetric ratio of FRP jacket required for restraining buckling of longitudinal bars in tall bridge columns may be given by the following proposed equation. All mathematical derivations are detailed elsewhere.[3]

$$\rho_{j2} = \begin{cases} \dfrac{30.7 n_b}{E_{j(design)}} \text{ for Grade 40 steel} \\ \dfrac{58.8 n_b}{E_{j(design)}} \text{ for Grade 60 steel} \end{cases} \quad (E_{j(design)} \text{ in MPa}) \tag{14a}$$

$$\rho_{j2} = \begin{cases} \dfrac{4.45 n_b}{E_{j(design)}} \text{ for Grade 40 steel} \\ \dfrac{8.54 n_b}{E_{j(design)}} \text{ for Grade 60 steel} \end{cases} \quad (E_{j(design)} \text{ in ksi}) \tag{14b}$$

where n_b = number of longitudinal reinforcing bars in the column, and $E_{j(design)}$ = design tensile modulus of FRP jacket. It should be noted that according to Priestley, Seible, and Calvi,[14] the anti-buckling requirement need not be applied to columns with an aspect ratio less than 4.0.

Lap-splice clamping—Along with the design for concrete confinement within the plastic hinge zone and anti-buckling requirements, design for lap-splice clamping should be carried out. The lap-splice region has to be confined in such a way that yielding occurs in the tensile longitudinal steel. The first step is to get bond strength of unconfined concrete from Eq. (15).[11]

$$\tau'_{bo} = \begin{cases} 20\sqrt{f'_{ce}}/d_{bl} \text{ (MPa; } d_{bl} \text{ in mm)} \\ 9.5\sqrt{f'_{ce}}/d_{bl} \text{ (psi; } d_{bl} \text{ in inches)} \end{cases} \tag{15}$$

where d_{bl} = diameter of longitudinal bar. Bond stress that corresponds to the first-yield point is then computed from

$$\tau_{yield} = \begin{cases} \dfrac{A_b f_{ye}}{\pi d_{bl}(L_s - 0.022 f_{ye} d_{bl})} = \dfrac{f_{ye} d_{bl}}{4(L_s - 0.022 f_{ye} d_{bl})} \text{ (MPa)} \\ \dfrac{A_b f_{ye}}{\pi d_{bl}(L_s - 0.15 f_{ye} d_{bl})} = \dfrac{f_{ye} d_{bl}}{4(L_s - 0.15 f_{ye} d_{bl})} \text{ (psi)} \end{cases} \tag{16}$$

where L_s = lap-splice length. The required bond strength of the retrofitted column (τ'_{bc}) should be greater than or equal to the bond stress at the first-yield state (τ_{yield}). According to Xiao's bond-slip model,[11] the peak bond stress of a jacketed column is expressed as

$$\tau'_{bc} = \tau'_{bo} + 1.4 f_l \tag{17}$$

where f_l is the transverse confining stress based on maximum dilation or hoop strain of 0.0015 as outlined by Priestley, Seible, and Calvi.[14] The minimum confining stress required for lap-splice clamping is then computed from

$$f_l = \frac{\tau_{yield} - \tau'_{bo}}{1.4} \tag{18}$$

Thereafter, the minimum confinement ratio for lap-splice clamping ρ_{j3} is calculated from

$$\rho_{j3} = \frac{2 f_l}{0.0015 E_{j(design)}} \tag{19}$$

The jacket confinement ratio required for flexural ductility enhancement (ρ_j) shall be the greater of ρ_{j1}, ρ_{j2}, and ρ_{j3}

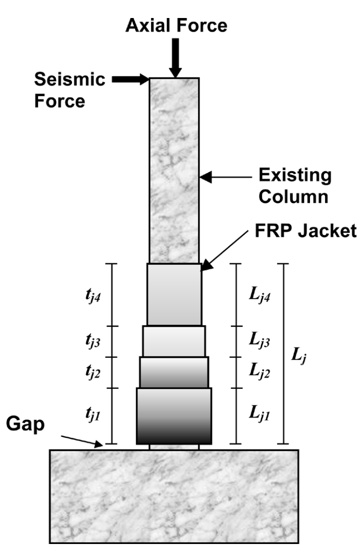

Fig. 5—Fiber-reinforced polymer jacket configuration for lap-spliced columns.

given by Eq. (13), (14), and (19), respectively. Subsequently, the required jacket thickness is obtained from

$$t_{j(req)} = \frac{D\rho_j}{4} \tag{20}$$

where D is the column diameter. The required number of layers is thereafter computed from

$$\text{No. of layers} = \frac{t_{j(req)}}{\text{thickness per layer}} \tag{21}$$

The number of layers, given by the previous equation, should be rounded up; the revised jacket thickness then has to be adjusted to determine the final design value required for flexural ductility enhancement ($t_{j(conf)}$). The developed code is then employed to analyze the retrofitted column and get its calculated ductility capacity, $\mu^{calc}_{\Delta(capacity)}$. It should be noted that in the analysis of composite-jacketed columns, the best-fit models that were determined previously have to be employed. Subsequently, the dependable (or reduced) ductility capacity is computed from

$$\mu^{Red}_{\Delta(capacity)} = \mu^{calc}_{\Delta(capacity)}\varphi_\mu \tag{22}$$

The dependable ductility capacity of the retrofitted column is then compared with the ductility demand. If the design needs to be revised, the following equation may be used.

$$t_{j(req)} = \frac{\mu_{\Delta(demand)}}{\mu^{Red}_{\Delta(capacity)}} t_{j(conf)} \tag{23}$$

Check of jacket design

Generally, lap-splice columns are tall with high aspect ratios. Accordingly, the shear demand on such columns is not high, and it is usually found out that most of existing lap-splice bridge columns are not shear dominant. Yet, FRP jacketing not only enhances the flexural ductility but increases the flexural strength as well and, as a result, shear demand on lap-splice columns increases upon jacketing. Subsequently, retrofitted columns should be protected against diagonal shear failure within and outside the hinge region. The same procedure that is detailed in a companion paper, written by the authors, for squat columns[13] will be followed exactly. In addition, the FRP-jacketed column has to be secured against both diagonal compression and shear-friction failures at its base following the same procedure detailed in a companion paper for squat columns.[13]

Extent of jacket

To guarantee the plastic hinge formation just above the column footing, the moment demand immediately above the jacket should be less than the original moment capacity of unretrofitted column section. In the design practice, a safety factor α is typically introduced. Accordingly, the jacket height should satisfy the following inequality.

$$L_j > \left(1.0 - \alpha\frac{M_{u(existing)}}{M_{u(retrofit)}}\right) L_c \tag{24}$$

Table 6—Configuration of fiber-reinforced polymer jacket for lap-splice columns

Zone	Jacket height	Jacket thickness

1	If $t^{i}_{j(sh)} < t_{j(conf)}$, then
	• $L_{j1} \ge 0.5\eta D$
	• $L_{j1} \ge 0.125\eta L_c$
	• $L_{j1} \ge L_s$
	
	If $t^{i}_{j(sh)} \ge t_{j(conf)}$, then
	• $L_{j1} \ge 0.125\eta L_c$
	• $L_{j1} \ge L_s$
	• $L_{j1} \ge 2D$
		If $t^{i}_{j(sh)} < t_{j(conf)}$, then
		• $t_{j1} = t_{j(conf)}$
		
		If $t^{i}_{j(sh)} \ge t_{j(conf)}$, then
		• $t_{j1} = t^{i}_{j(sh)}$

2	If $t^{i}_{j(sh)} < t^{sec}_{j(conf)}$, then
	• $L_{j2} \ge \eta D - L_{j1}$
	• $L_{j2} \ge 0.25\eta L_c - L_{j1}$
	
	If $t^{i}_{j(sh)} \ge t^{sec}_{j(conf)}$, then
	• $L_{j2} \ge 0.25\eta L_c - L_{j1}$
	• $L_{j2} \ge 2D - L_{j1}$
		If $t^{i}_{j(sh)} < t^{sec}_{j(conf)}$, then
		• $t_{j2} = t^{sec}_{j(conf)}$
		
		If $t^{i}_{j(sh)} \ge t^{sec}_{j(conf)}$, then
		• $t_{j2} = t^{i}_{j(sh)}$

3	If $0 < t^{i}_{j(sh)} < t^{sec}_{j(conf)}$, then
	• $L_{j3} \ge 2D - L_{j1} - L_{j2}$
	
	If $t^{i}_{j(sh)} \ge t^{sec}_{j(conf)}$, then
	• $L_{j3} = 0.0$
	
	If $t^{i}_{j(sh)} = 0.0$, then
	• $L_{j3} = 0.0$
		If $0 < t^{i}_{j(sh)} < t^{sec}_{j(conf)}$, then
		• $t_{j3} = t^{i}_{j(sh)}$
		
		If $t^{i}_{j(sh)} \ge t^{sec}_{j(conf)}$, then
		• $t_{j3} = 0.0$
		
		If $t^{i}_{j(sh)} = 0.0$, then
		• $t_{j3} = 0.0$

4	If $L_j > (L_{j1} + L_{j2} + L_{j3})$, then
	• $L_{j4} = L_j - (L_{j1} + L_{j2} + L_{j3})$
	
	If $L_j \le (L_{j1} + L_{j2} + L_{j3})$, then
	• $L_{j4} = 0.0$
		If $L_j > (L_{j1} + L_{j2} + L_{j3})$, then
		• $t_{j4} = 0.5t^{i}_{j(sh)}$ if $t^{i}_{j(sh)} > 0.0$
		• $t_{j4} = 0.25t_{j(conf)}$ if $t^{i}_{j(sh)} = 0.0$
		
		If $L_j \le (L_{j1} + L_{j2} + L_{j3})$, then
		• $t_{j4} = 0.0$

where L_j = jacket height; L_c = column height; $M_{u(existing)}$ = moment capacity of unretrofitted column section; $M_{u(retrofit)}$ = moment capacity of retrofitted column section; and α = safety factor less than 1.0, and is taken 0.85. The final jacket configuration is illustrated in Table 6 and Fig. 5. It should be noted that jacket zones in Table 6 are listed in order from bottom to top. The parameters in Table 6 are identified as: $t_{j(conf)}$ = jacket thickness for primary confinement zone; $t^{sec}_{j(conf)}$ = jacket thickness for secondary confinement zone = $0.5t_{j(conf)}$; $t^{i}_{j(sh)}$ = jacket thickness for shear enhancement within the plastic end region; and η = factor depends on axial load ratio according to the following equation, which was proposed by Seible, Priestley, and Innamorato.[15]

$$\eta = \begin{cases} 1.0: \dfrac{P}{f'_{ce}\,A_g} \le 0.3 \\ 1.5: \dfrac{P}{f'_{ce}\,A_g} > 0.3 \end{cases} \tag{25}$$

CONCLUSIONS

The following conclusions are derived from this research:

1. Developed in this study is an optimum computational tool for prediction of seismic performance of circular RC columns with lap-spliced reinforcement. It is based on modified Xiao's bond-slip model along with Hosotani's model for FRP-confined concrete;

2. In addition to performance prediction, statistical analyses were carried out on tested-to-calculated ratios. Based on statistical lower and upper bounds, safety factors were derived for both seismic assessment and retrofit design of lap-splice-deficient columns. It should be noted that due to limitations of available experimental database for circular lap-splice columns tested under lateral cyclic loading, all proposed design factors may be subject to refinement with the availability of more experimental database information; and

3. The optimum computational tool in conjunction with the proposed safety factors were incorporated in a rigorous design methodology for circular lap-splice RC bridge columns upgraded with FRP jackets. In the proposed design approach, jacket thickness within the lap-splice zone will be the greater of: requirement for confinement of the compression concrete, anti-buckling constraint, and requirement for clamping on the lap-splice region. It is imperative to note that in addition to bridge columns, the proposed retrofit design criteria may be also applied to circular RC building columns in seismic regions.

NOTATION

A_b = area of longitudinal steel bar
A_g = gross area of column section
$c_{u(existing)}$ = neutral axis depth at ultimate response of existing column
$c_{u(ret)}$ = neutral axis depth at ultimate response of retrofitted column
D = diameter of circular column
d_{bl} = diameter of longitudinal steel bar
$E_{j(design)}$ = design tensile modulus of FRP jacket
f'_{ce} = expected (most probable) concrete strength
$f_{ju(design)}$ = design tensile strength of FRP jacket
f_{ye} = expected yield stress of reinforcing steel
L_c = column height
L_j = height of FRP jacket
L_p = length of plastic hinge
L_s = lap-splice length
$M_{u(existing)}$ = moment capacity of existing column section
$M_{u(retrofit)}$ = moment capacity of retrofitted column section
n_b = number of longitudinal steel bars within column section
P = axial force
$t_{j(conf)}$ = thickness of FRP jacket within primary confinement zone
$t_{j(req)}$ = required thickness of FRP jacket
$t^{i}_{j(sh)}$ = thickness of FRP jacket for shear enhancement within plastic end region
Δ_p = plastic displacement
Δ_{u-exp} = ultimate experimental displacement
Δ_{u-th} = ultimate theoretical displacement
Δ_y = idealized yield displacement
ε_{cu} = ultimate concrete compressive strain
$\varepsilon_{ju(design)}$ = design ultimate tensile strain of FRP jacket
Φ_p = plastic curvature of column section
Φ_u = ultimate curvature of column section
φ_v = retrofit design factor for maximum lateral load
φ_μ = ductility knockdown factor
$\mu_{\Delta(demand)}$ = demand ductility
$\mu_{\Delta u-exp}$ = ultimate experimental displacement ductility
$\mu_{\Delta u-th}$ = ultimate theoretical displacement ductility
$\mu^{calc}_{\Delta(capacity)}$ = calculated ductility capacity
$\mu^{R}_{\Delta(capacity)}$ = required ductility capacity
$\mu^{Red}_{\Delta(capacity)}$ = dependable (or reduced) ductility capacity
θ_p = plastic rotation at center of plastic hinge
ρ_j = volumetric ratio of FRP jacket
σ = standard deviation
τ'_{bc} = bond strength of confined concrete
τ'_{bo} = bond strength of unconfined concrete
τ_{yield} = bond stress at first-yield state

REFERENCES

1. Chai, Y. H.; Priestley, M. J. N.; and Seible, F., "Flexural Retrofit of Circular Reinforced Concrete Bridge Columns by Steel Jacketing—Experimental Studies," *Report* No. SSRP-91/06, UCSD, Oct. 1991.

2. Ma, R., "Seismic Retrofit and Repair of Reinforced Concrete Columns Using Advanced Composite Materials," PhD dissertation, University of Southern California, Los Angeles, Calif., Aug. 1999.

3. Elsanadedy, H. M., "Seismic Performance and Analysis of Ductile Composite-Jacketed Reinforced Concrete Bridge Columns," PhD dissertation, University of California, Irvine, Calif., 2002.

4. Haroun, M. A.; Feng, M. Q.; Bhatia, H.; Baird, K.; and Elsanadedy, H. M., "Structural Qualification Testing of Composite-Jacketed Circular and Rectangular Bridge Columns," *Final Report to the California Department of Transportation*, University of California, Irvine, Calif., 1999.

5. Mander, J. B.; Priestley, M. J. N.; and Park, R., "Theoretical Stress-Strain Model for Confined Concrete," *Journal of the Structural Division*, ASCE, V. 114, No. 8, 1988, pp. 1804-1826.

6. Samaan, M.; Mirmiran, A.; and Shahawy, M., "Model of Concrete Confined by Fiber Composites," *Journal of Structural Engineering*, ASCE, V. 124, No. 9, Sept. 1998, pp. 1025-1031.

7. Hosotani, M.; Kawashima, K.; and Hoshikuma, J., "A Stress-Strain Model for Concrete Cylinders Confined by Carbon Fiber Sheets," *Report* No. TIT/EERG 98-2, Tokyo Institute of Technology, Tokyo, Japan, 1998, 55 pp. (in Japanese)

8. Hoppel, C. R.; Bogetti, T. A.; Gillespie, J. W., Jr.; Howie, I.; and Karbhari, V. M., "Analysis of a Concrete Cylinder with a Composite Hoop Wrap," *Proceedings*, 1994 ASCE Materials Engineering Conference, ASCE, New York, pp. 191-198.

9. Toutanji, H., "Stress-Strain Characteristics of Concrete Columns Externally Confined with Advanced Fiber Composite Sheets," *ACI Materials Journal*, V. 96, No. 3, May-June 1999, pp. 397-404.

10. Spoelstra, M. R., and Monti, G., "FRP-Confined Concrete Model," *Journal of Composites for Construction*, ASCE, V. 3, No. 3, Aug. 1999, pp. 143-150.

11. Xiao, Y., and Rui, M., "Seismic Retrofit of RC Circular Columns Using Prefabricated Composite Jacketing," *Journal of Structural Engineering*, ASCE, V. 123, No. 10, 1997, pp. 1357-1364.

12. Chapman, W. et al., *Pre-Qualification Requirements for Alternative Column Casings for Seismic Retrofit*, California Department of Transportation, Sacramento, Calif., 1997.

13. Elsanadedy, H. M., and Haroun, M.A., "Seismic Design Guidelines for Squat Composite-Jacketed Circular and Rectangular RC Bridge Columns," *ACI Structural Journal*, V. 102, No. 4, July-Aug. 2005.

14. Priestley, M. J. N.; Seible, F.; and Calvi, G. M., *Seismic Design and Retrofit of Bridges*, John Wiley & Sons, Inc., New York, 1996, 704 pp.

15. Seible, F.; Priestley, M. J. N.; and Innamorato, D., "Earthquake Retrofit of Bridge Columns with Continuous Carbon Fiber Jackets—Volume II, Design Guidelines," Advanced Composites Technology Transfer Consortium, *Report* No. ACTT-95/08, University of California, San Diego, Calif., Aug. 1995.