

Modeling of RC column with lap splices strengthened by Fiber Reinforced Polymer (FRP)

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Abstract

This paper presents a two-dimensional (2D) nonlinear analytical model for analyzing reinforced concrete (RC) column with short lap splice strengthened by fiber reinforced polymer (FPR) under seismic force. The model was developed based on previous proposed nonlinear model, and applied to simulate experiment studies on RC column with lap splice strengthened by FRP sheets or prefabricated plates for verification. The model is applicable to both rectangular and circular section, and to both full and partial wrapping. The model is divided into an elastic frame and an assembly of nonlinear springs to represent the elastic portion and the plastic hinge, respectively. The effectiveness of FRP wrapping for strengthening lap splice is taken into account by modifying properties of nonlinear springs. Finally, the model was incorporated into Ruaumoko2D software for analyzing RC column strengthened by FRP wrapping around lap splice zone.

Keyword: substandard column, nonlinear model, Fiber-reinforced polymer, lap splice, bond strength.

1 Introduction

The critical non-seismic reinforcement details of sub-standard columns which violate modern seismic design theory were identified. These include short lap splices of column

reinforcement located immediately above floor levels or lap splice with starter bars from footing, widely spaced column ties, inadequate hoop reinforcement in potential plastic hinge regions and joint. Lap splice length designed for gravity only was normally shorter than that required in current seismic design standards, thereby possibly aggravating seismic performance of structure. The transfer of forces across the interface between steel and concrete through bond plays a major role on the behavior of reinforced concrete members. Cracking, deflections, structural strength, energy absorption and dissipation capacities under seismic loading are all related directly or indirectly to bond.

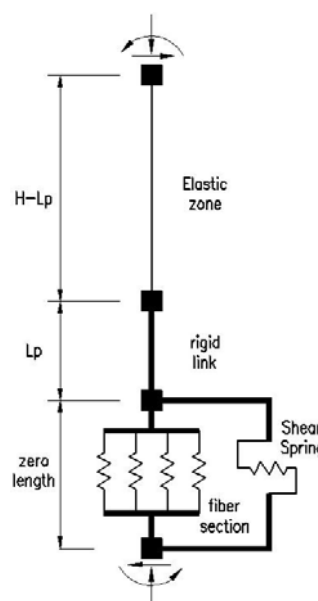


Figure 1 RC column model, Matrin [1]

Several methods, such as external jacketing by steel plate, composite materials, or steel fiber concrete, were used to strengthen lap splice capacity. Some experiments and analytical models were conducted to assess the effect of strengthening lap splice on seismic resistance of column, and also to assess the effectiveness of each strengthening method on lap splice capacity improvement.

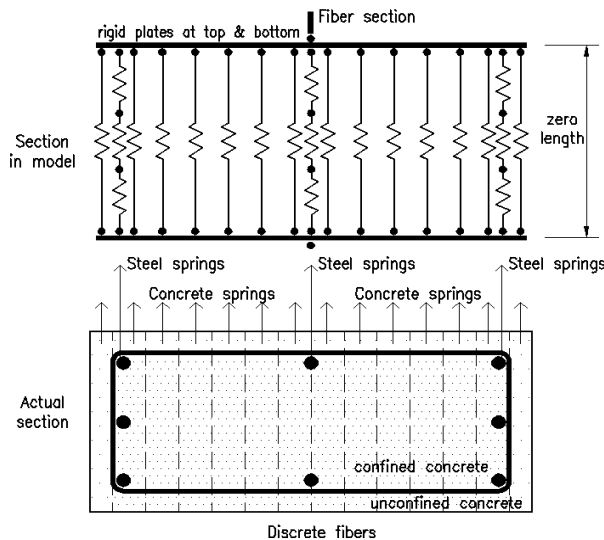


Figure 2 Discrete fiber section, Matrin [1]

Fiber reinforced polymer (FRP) composites have found increasingly wide applications in civil engineering due to their high strength-to-weight ratio and high corrosion resistance. One important application of FRP-composites is as a confining material for concrete in the retrofit of existing reinforced concrete (RC) columns. As a result of FRP confinement, the compressive strength and the ultimate strain of concrete can be greatly enhanced. The bond stress between reinforced bar and concrete is also increased.

Lowes, Mitra et al. [4] formulated a versatile two-dimensional macro-model that represents a reinforced concrete beam-column joint. In their model, the bond slip and shear deformation responses of joints were considered as separate springs. The zero-length bond-slip springs were representatives of the force transferring ability between

flexural reinforcement and the surrounding concrete while the zero-length shear springs represented the ability of shear transferring at the interface between frame members and the joint.

Matrin [1] developed a reinforced concrete column two-dimension model (Figure 1) to simulate behavior of reinforced concrete column without jacketing, subjected to seismic excitation. Basically, a single-curvature column model consists of two major parts; the first part is an elastic frame of $H - L_p$ length, which represents the elastic behavior of column. The second part consists of a zero-length fiber section which is an assembly of nonlinear springs to represent the concrete and reinforcement nonlinear behavior. The second part is connected to the first part by a rigid link of L_p length.

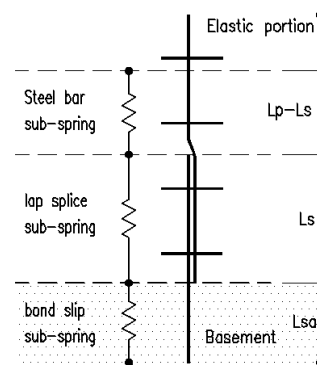


Figure 3 Model a reinforced bar by 3 sub-springs

The fiber section was divided into a number of discrete fibers (Figure 2). Each fiber will be modeled as concrete spring or steel spring, whose characteristics can be calculated from the section of fiber and material properties. The concrete springs are classified into two types, confined concrete and unconfined concrete. The steel springs represent the reinforcement in each fiber (Figure 3). The steel spring normally consisting of three sub springs, i.e. steel bar sub-spring, lap splice spring, and bond-slip spring connected in series from top of fiber. Shear behavior of column can be simulated by a separate nonlinear shear spring.

Ruaumoko2D is a well known platform for modeling reinforced concrete structure, especially with capacity of degradation simulation and a huge database of hysteresis model embedded in the program. Matrin [1] used Ruaumoko2D to simulate cyclic behavior of several reinforced concrete members, and found that the comparison between analytical model and experimental result were closed. In this paper, the authors employed Ruaumoko2D to simulate reinforced concrete column externally confined by FRP sheet around the lap splice region. The nonlinear model of lap splice spring model with FRP confinement is incorporated into Ruaumoko2D. The suitability of the nonlinear model is extensively verified via several experiments conducted in the past.

2 Numerical modeling parameters

Various nonlinear parameters for modeling RC column strengthened by FRP confinement are described here. Particular attention is paid to the nonlinear properties of lap splice spring with and without FRP wrapping.

2.1 Length of plastic hinge region

The length of plastic hinge (L_p) is an essential parameter, which separates a column into 2 parts, i.e., elastic (frame) and plastic (fiber section) part. Priestley and Seible [5] suggested the following plastic hinge length formulas for analyzing inelastic flexural behaviors of reinforced concrete columns with and without FRP jacketing:

$$\text{Without FRP jacketing } L_p = 0.08H + 0.022d_b f_y \quad (1)$$

$$\text{With FRP jacketing } L_p = g + 0.044d_b f_y \quad (2)$$

Where g is a gap at the bottom of the jacket; d_b and f_y are diameter and yield strength of reinforced bar, respectively.

Xiao and Ma [2] proposed an approach, in which the plastic hinge length was assumed variable corresponding to the steel stress of the extreme tensile bar f_s . The expressions (3) and (4) take the same format of (1) and (2) but use variable steel stress f_s , instead of the yield strength f_y .

$$\text{Without FRP jacketing } L_p = 0.08H + 0.022d_b f_s \quad (3)$$

$$\text{With FRP jacketing } L_p = g + 0.044d_b f_s \quad (4)$$

2.2 Elastic frame element

Elastic-frame element is used to simulate flexural and axial stiffness in the elastic part (top portion of the cantilever column). As recommended by ATC-40 [6], the effective flexural rigidity is set to $0.7E_c I_g$; the effective axial rigidity is set to $1.0E_c A_g$; the effective shear stiffness is set to $0.4E_c A_g / (H - L_p)$. Elastic modulus of concrete E_c is set to $15200\sqrt{f'_c}$ (ksc) as recommended by ACI318 [7]. In which, f'_c is compressive strength of unconfined concrete; A_g and H are gross area of section and the height of column, respectively.

2.3 Nonlinear Shear spring

Nonlinear shear spring is used to capture shear mechanism (deformation and stress) of column. Effective shear stiffness is set to $0.4E_c A_g / L_p$ as adopted from ATC-40 [6]

Shear strength of column with and without FRP wrapping can be calculated by equation (5) [8].

$$V_u = V_n + V_f \quad (5)$$

In which V_n is shear strength of column without FRP wrapping obtained by equation (6) [9]. V_f is additional shear strength provided by FRP sheets given by (7)-(9) [8].

$$V_n = k \left[\frac{6\sqrt{f'_c}}{a/d} \sqrt{1 + \frac{P}{6\sqrt{f'_c} A_g}} 0.8A_g + \frac{A_{st} f_{yt} d}{s} \right] \quad (6)$$

$$V_f = \nu \lambda_f f_t b d \quad (7)$$

$$\nu = \frac{1.639 \left(0.403 - 1.053 \frac{P}{A_g f'_c} + 0.176 \frac{a}{h} \right)}{\sqrt{\lambda_f} + 1.207} \quad (8)$$

$$\lambda_f = \frac{2 t_f b_f f_f}{b s_f f_t} \quad (9)$$

$$\frac{\Delta_r}{H} = \frac{4}{100} \frac{1 + (\tan \theta)^2}{\tan \theta + P \left(\frac{s}{A_{st} f_{yt} d \tan \theta} \right)} \quad (10)$$

Lateral displacement (10) of gravity load collapse Δ_r is proposed by Moehle, Elwood et al. [10]. A monotonic curve that governs the behavior of shear spring is shown in Figure 4, in which V_r is lateral force corresponding to state of gravity load collapse and assumed equal to $0.1V_u$.

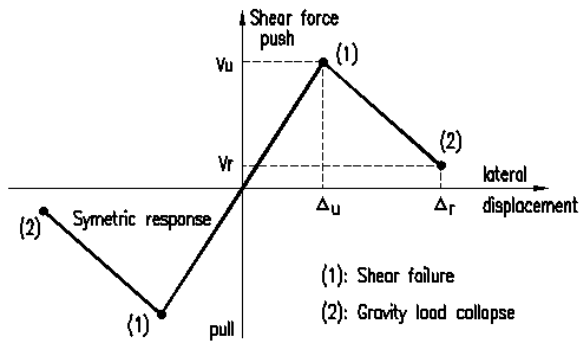


Figure 4 Monotonic curve for nonlinear shear spring

2.4 Concrete spring

The peak compressive force of concrete spring F_c (11) depends on area of concrete in a fiber A_c and confined compressive strength of concrete f'_{cc} which can be obtained from unconfined compressive strength f'_c by multiplying with confinement factor K shown in Eq. (12).

$$F_c = f'_{cc} A_c \quad (11)$$

$$f'_{cc} = K f'_c \quad (12)$$

Calculation of confinement factor K follows a uniaxial stress-strain model (13); (14) for confined concrete with either FRP or transverse reinforcement as proposed by Harajli, Hantouche et al. [11]. The model is applicable to various conditions of cross section (rectangular and circular), wrapping method (partial and full), thickness and number of FRP sheets, types of steel stirrups (spiral and tie).

$$f'_{cc} = f'_c + k_1 \left(f_{lf} + f_{ls} \frac{A_{cc}}{A_g} \right) \quad (13)$$

$$\varepsilon_{cc} = \varepsilon_o \left(1 + k_2 \left(\frac{f'_{cc}}{f'_c} - 1 \right) \right) \quad (14)$$

$$\varepsilon_{50} = \frac{3 + 0.002 f'_c}{f'_c - 1000} \quad (psi) \quad (15)$$

The degradation of concrete spring is determined by a point of remaining 50% peak stress with corresponding strain (15) [12]. A monotonic curve that governs the behavior of concrete spring (confined or unconfined) is shown in Figure 5.

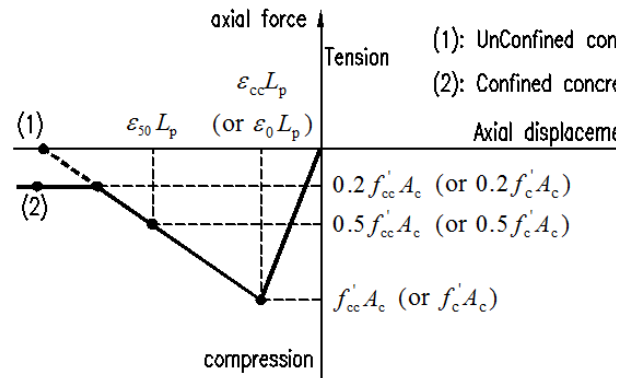


Figure 5 Monotonic curve for concrete spring

2.5 Bond slip sub-spring

A bi-linear bond-stress model proposed by Sezen and Moehle [13] is adopted. The anchored bar length was generally divided into two parts, elastic L_e and plastic L_y zone with constant bond stresses u_e and u_y , respectively. The length of elastic zone is given as $L_e = f_e d_b / (4u_e)$, while the length of plastic zone L_y is set equal to $(f_s - f_y) d_b / (4u_y)$ (if $f_s >$

f_y). When the stress acting on steel bar f_s is less than yield strength f_y , the plastic zone disappears, the anchorage behavior is purely elastic.

Plastic bond stress u_y depends on the state of stress in the anchorage zone. For tension and compression zone, $u_y = 0.4\sqrt{f'_c}$ (Shima, Chou et al. [14]) and $u_y = 3.6\sqrt{f'_c}$ (Lowes, Mitra et al. [4]), respectively. Elastic bond stress can be approximately estimated by $u_e = f_y d_b / (4L_e)$, in which the required length L_e is recommended by ACI318 [7] for tension (16) and compression (17) zone.

$$L_e = \frac{f_y}{1.1\sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s \lambda}{\left(\frac{c_b + K_{tr}}{d_b}\right)} d_b \quad (16)$$

$$L_e = \max \left(\frac{0.24 f_y}{\sqrt{f'_c}} d_b; 0.043 f_y d_b \right) \quad (17)$$

The critical force at bond failure can be obtained by trial procedure, in which the stress of bar will be increased until the satisfaction of $L_e + L_y = L_{sa}$. Figure 6 shows two types of bond failures, before and after bar yielding depending on the anchorage length L_{sa} . The bond-slip can be computed as $\Delta_s = 0.5 (L_e \varepsilon_e + L_y (\varepsilon_e + \varepsilon_s))$.

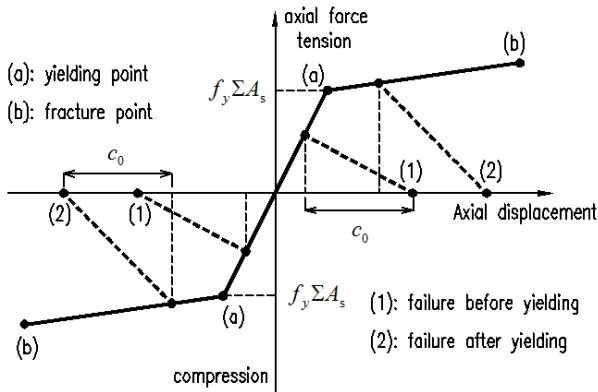


Figure 6 Monotonic curve governs bond and lap splice spring

2.6 Steel bar sub-spring

This spring represents the behavior of steel reinforcement in zone of plastic hinge L_p . The yield force of the spring

is $f_y \Sigma A_s$. The elastic and plastic stiffness of spring are equal to $E_s (\Sigma A_s) / L_p$ and $E_p (\Sigma A_s) / L_p$, respectively.

2.7 Lap splice sub-spring

In case of RC column without confinement, the behavior of lap splice spring follows the same concept as bond-slip spring. The calculation follows section 2.5, in which the lap splice length L_s is used instead of anchorage length L_{sa} .

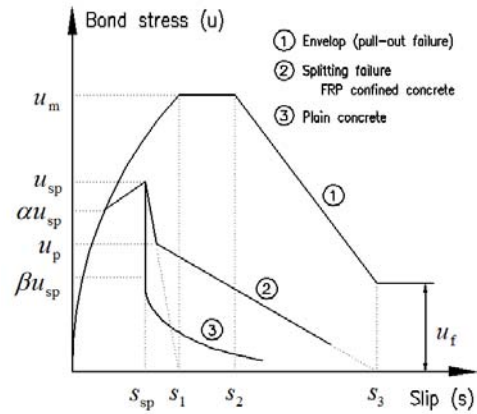


Figure 7 bond stress-slip model by Harajli [3]

Since FRP wrapping around lap splice zone provides additional lateral stress, thereby increasing the bond stress. Harajli [3] proposed a bond-slip model (Figure 7) for lap splice strengthened by FRP, the peak value of bond stress and corresponding slip s_{sp} at bond splitting failure are given in equations (18) and (19), respectively.

$$u_{sp} = 0.75 \sqrt{f'_c} \left(\frac{c + 56 \frac{E_f \alpha_f n_f t_f}{E_s n_s}}{d_b} \right)^{2/3} \leq u_m \quad (18)$$

$$s_{sp} = s_1 e^{3.3 \ln \left(\frac{u_{sp}}{u_m} \right)} + s_0 \ln \left(\frac{u_m}{u_{sp}} \right) \quad (19)$$

In which u_m is the maximum bond stress at pullout mode of bond failure given as $u_m = 2.57 \sqrt{f'_c}$. Factor α_f represents the method of wrapping, with fully wrap $\alpha_f = 1$ and partially wrap α_f taken as expression (20), in which N_f is number of FRP strips with equal width b_f .

$$\alpha_f = \frac{N_f b_f}{L_s} \quad (20)$$

As observed from eq.(18), bond strength is dependent on compressive strength of unconfined concrete f'_c , and a ratio $\frac{c}{d_b}$ (ratio of concrete cover depth to diameter of bar). Bond strength is also enhanced by increasing the number of FRP sheets or thickness of sheet. The effect of wrapping method on bond strength is accounted for by factor α_f , in which full wrapping provides greater bond strength than partial wrapping.

2.8 Built-in Hysteretic Rules in Ruaumoko2D

All nonlinear springs described above are incorporated as back bone curve in Ruaumoko 2D. The model parameters are calculated from the actual geometrical sections and material properties. The built-in degradation model and hysteresis rules were combined with the above-mentioned envelope to provide the complete cyclic spring models. In the present study, the modified Takeda Degrading Stiffness hysteresis is used for steel spring, the Bi-linear with Slackness hysteresis is used for concrete spring and the SINA Degrading Stiffness hysteresis is applied to shear spring.

3 Selected experimental program

Xiao and Ma [2] conducted an experimental study on seismic retrofit of reinforced concrete circular columns with poor lap-splice detail using prefabricated composite jacketing. In the study, three half scale model circular columns were tested. One column C1-A was tested under condition of “as built” and the other two columns C2-RT4 and C3-RT5 were tested after being strengthened with 4 and 5 FRP layers. All columns were 2.44m tall and 0.61m diameter. Longitudinal reinforcements were 20 deformed #6 bars ($d_b = 19.1$ mm, 2% reinforcement ratio, grade 60) with lap splice length of 0.38m ($20d_b$) at the base. Round #2 hoops ($\phi 6.4$ mm) spaced

at 127mm intervals were used as transverse reinforcement. Concrete strength was 44.8 MPa (28 days cylinder specimen). Composite jacketing layer was 3.2mm thick prefabricated unidirectional glass fiber sheets. The elastic modulus and ultimate strength in the circumferential direction are 48300 MPa and 552 MPa, respectively. The sheets were attached to the column via two part epoxy.

The axial load applied to the column is 712 kN ($P/A_g f'_c = 5\%$). An analytical model was built to simulate these columns. Core concrete section (inside stirrup perimeter) was symmetrically divided into 42 discrete fibers of 1.30cm width. Thus, 42 springs with various properties were used to model confined concrete fiber and 11 steel springs were used to model all reinforcing bars. Cover unconfined section (outside stirrup perimeter) was symmetrically divided into 42 discrete fibers of 1.30cm width and 6 fibers of 1.50cm width. Thus a total of 48 springs with various properties were used to represent unconfined concrete. One spring was used to model column shear behavior and one elastic element to represent elastic behavior zone of column.

Harajli and Dagher [15] conducted an experimental investigation on the use of external fiber reinforced polymer (FRP) confinement for bond strengthening of spliced reinforcement in rectangular reinforced concrete columns and the consequent effect on the seismic response. Nine 0.4m deep x 0.2 m wide x 1.5m high columns were divided into 3 series based on steel diameter used; 14mm diameter series named C14FP1, C14FP2 and C14E; 16mm diameter series named C16FP1, C16FP2 and C16E; and 20mm diameter series named C20FP1, C20FP2 and C20E. In each specimen label, suffixes “FP1” and “FP2” meant the number of FRP sheets, and suffix “E” means column with no lap splice of reinforcing bars. Each specimen consists of 8 reinforcing bars with lap splice length of $30d_b$. The ratio of concrete cover to bar

diameter c/d_b is 1.4, 2.1 and 1.0 for series C14, C16 and C20 respectively. Transverse steels were 8mm diameter spaced at 200mm throughout the height of the strengthened column, while in unconfined “E” column, stirrup was 8 mm diameter spaced at 50mm within 500mm from the base and spaced at 100mm within the remaining height. Yield strength of bars f_y in series C14, C16 and C20 were 550MPa, 528MPa and 617MPa respectively. Compressive strength of concrete f'_c varied from 32MPa to 40MPa. The FRP sheets were 0.13mm thick; with elastic modulus of 230000MPa and tensile fracture strain of 1.5%. An analytical model was built to simulate these specimens. Core section (inside stirrup perimeter) was symmetrically divided into 42 discrete fibers of 0.857cm width, thus 42 springs were used to model confined concrete fiber and 3 (or 4 in series C16) steel spring combinations were used to model all reinforced bars. Cover unconfined concrete part (outside stirrup perimeter) was symmetrically divided into 42 discrete fibers of 0.857cm width and 6 fibers of 0.667cm width, thus 48 springs were used to represent unconfined concrete. One spring was used to model column shear behavior and one elastic element to represent elastic behavior zone of column. The behavior of analytical model was compared with corresponding experimental result shown in the next section.

4 Verification of the nonlinear modeling

4.1 Series C14 tested by Harajli and Dagher [15]

Figure 8 presents the backbone relation between bond slip and ratio of bond stress (u) to unconfined concrete compressive strength (f'_c) as explained in section 2.7. This backbone relation is combined with the built-in hysteresis rules to obtain the complete cyclic bond-slip model. According to the model, ratios $u/\sqrt{f'_c}$ reach maximum values of 0.976, 1.068 and 1.157 in specimens C14, C14FP1 with one FRP layer and C14FP2 with two FRP layers, respectively.

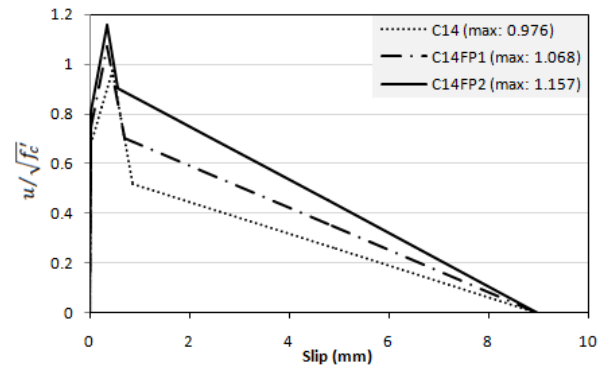


Figure 8 Bond stress-slip in C14 series

Table 1 shows the comparison between experimental and analytical results of series C14. The maximum experimental lateral force of columns C14, C14FP1 and C14FP2 was 81.2, 87.2 and 92.2 kN respectively while the analysis was 82.0, 92.3 and 97.2 kN respectively. The differences are only 1%, 6% and 5% respectively. It is found that in both analytical and experimental results, the lateral force increased with the increasing number of FRP sheets.

Table 1 Series C14 results

Specimen	Lateral force – kN			Type of lap splice failure
	Exper.	Analysis	Differ.	
C14	81.2	82.0	1.01	Before yielding of bar
C14FP1	87.2	92.3	1.06	Before yielding of bar
C14FP2	92.2	97.2	1.05	After yielding of bar
Average			1.04	
S.D			0.03	

Figure 9b and Figure 10b show analytical response of series C14, which demonstrates the performance of FRP wrapping to confine lap splice of RC column. Figure 9a and Figure 10a show corresponding experimental results. It is noted that the analytical results agreed well with experimental ones.

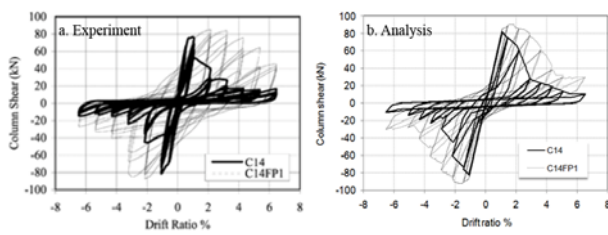


Figure 9 Response of columns C14 and C14FP1

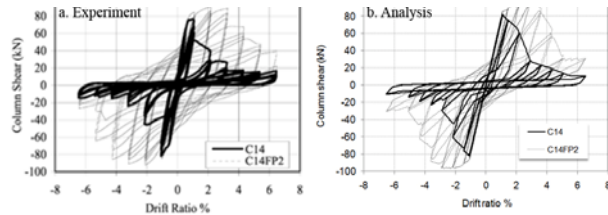


Figure 10 Response of columns C14 and C14FP2

4.2 Series C16 tested by Harajli and Dagher [15]

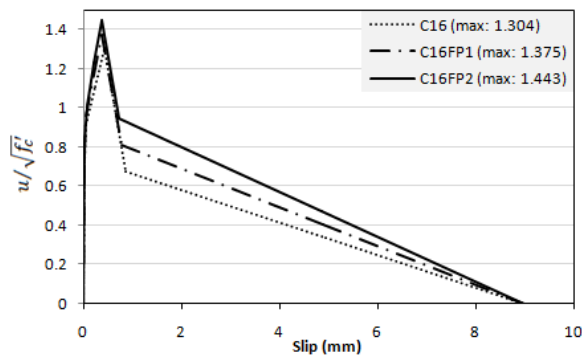


Figure 11 Bond stress-slip in C16 series

Figure 11 presents the backbone relation between normalized bond stress versus slip obtained from the model. The ratios $u/\sqrt{f_c'}$ reach maximum values of 1.304, 1.375 and 1.443 in specimens C16, C16FP1 with one FRP layer and C16FP2 with two FRP layers, respectively.

Table 2 shows the comparison between experimental and analytical results of series C16. The maximum experimental lateral force of columns C16, C16FP1 and C16FP2 was 107.4, 112.5 and 107.4 kN while the analysis gave the values of 97.9, 108.0 and 109.0 kN respectively. The differences between results are 9%, 4% and 1% respectively. It is found that in the analysis results, the lateral force increased with increasing

number of FRP sheets, while the experiment gave strange results with column C16FP2 wrapped by two FRP layers has the lateral strength equal to un-strengthened column C16 and less than the strength of column C16FP1 with one FRP layer.

Table 2 Series C16 results

Specimen	Lateral force			Type of lap splice failure
	Exper.	Analysis	Differ.	
C16	107.4	97.9	0.91	After yielding of bar
C16FP1	112.5	108.0	0.96	After yielding of bar
C16FP2	107.4	109.0	1.01	After yielding of bar
Average			0.96	
S.D			0.05	

Figure 12b and Figure 13b show analytical response of series C16, which represents the enhancement of the lateral capacity and the ductility provided by FRP confinement around lap splice zone. Figure 12a and Figure 13a show corresponding experimental results. It is found that the analytical results agreed closely with experimental ones, except the stiffness and strength degradation which seem to be lower than the experimental results. This is probably because there may be other nonlinear effects such as spalling and reinforcement buckling that are not taken into account in the present analytical model.

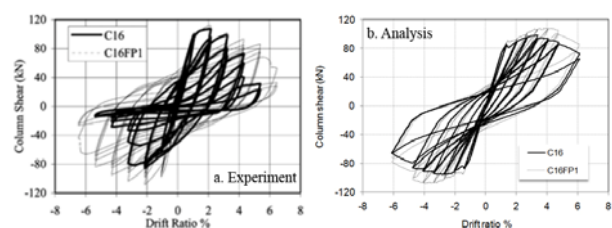


Figure 12 Response of columns C16 and C16FP1

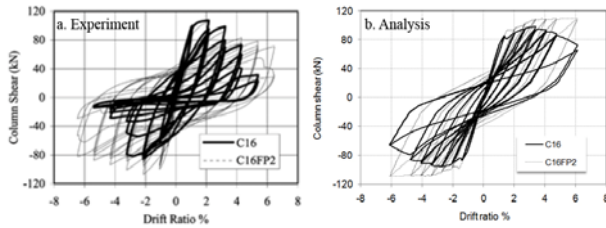


Figure 13 Response of columns C16 and C16FP2

4.3 Series C20 tested by Harajli and Dagher [15]

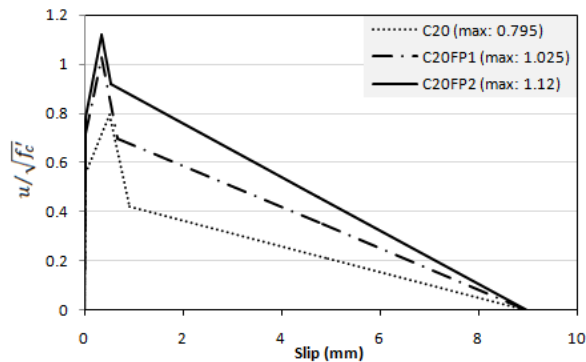


Figure 14 Bond stress-slip in C20 series

Figure 14 presents the backbone relation between normalized bond stress versus slip as obtained from the model. The ratios $u/\sqrt{f'_c}$ reach maximum values of 0.795, 1.025 and 1.120 in specimens C20, C20FP1 with one FRP layer and C20FP2 with two FRP layers, respectively.

Table 3 Series C20 results

Specimen	Lateral force			Type of lap splice failure
	Exper.	Analysis	Differ.	
C20	91	83.9	0.92	Before yielding of bar
C20FP1	126	130.0	1.03	Before yielding of bar
C20FP2	141.8	135.0	0.95	Before yielding of bar
Average				0.97
S.D				0.06

Table 3 shows the comparison between experimental and analytical results of series C20. The maximum experimental lateral force of columns C20, C20FP1 and C20FP2 was 91,

126 and 141.8 kN while the analysis gave the values of 83.9, 130 and 135 kN respectively. The differences are 8%, 3% and 5% respectively. It is found that in both analytical and experimental results, the lateral force increased with increasing number of FRP sheets.

Figure 15b and Figure 16b show analytical response of series C20, which demonstrate the increase of the lateral capacity and the ductility provided by FRP wrapping around lap splice zone. Figure 15a and Figure 16a show corresponding experimental results. As shown, the analytical results agreed well with experimental ones.

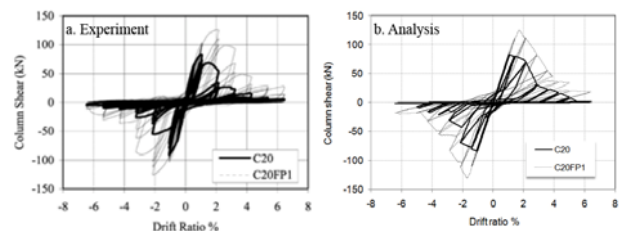


Figure 15 Response of columns C20 and C20FP1

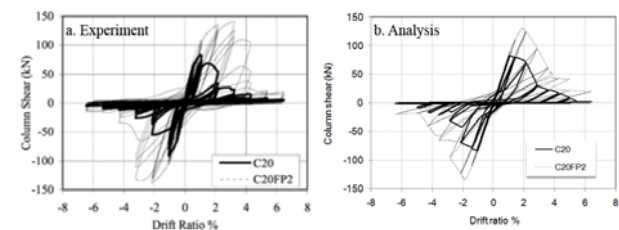


Figure 16 Response of columns C20 and C20FP2

4.4 Circular RC columns tested by Xiao and Ma [2]

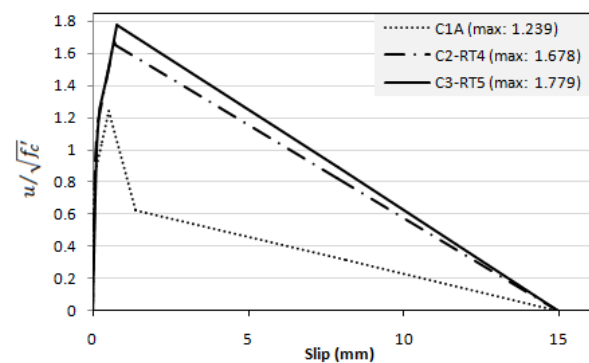


Figure 17 Bond stress-slip in Xiao and Ma [2] columns

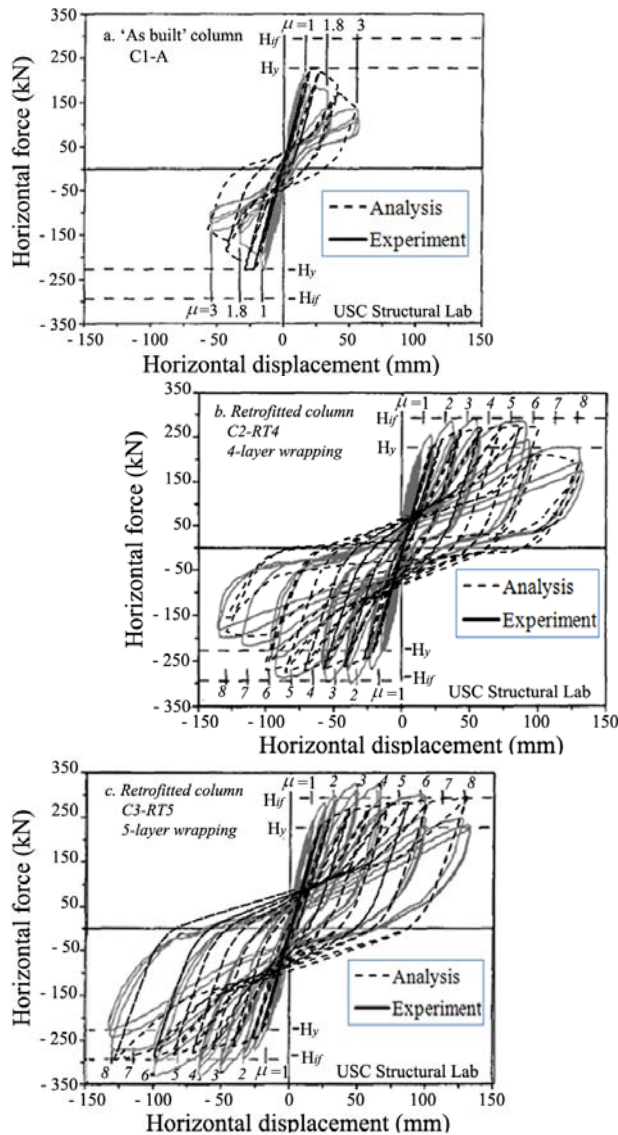


Figure 18 Response of columns C1-A; C2-RT4 and C3-RT5

Figure 17 presents the backbone relation between normalized bond stress versus slip as obtained from the model. The ratios $u/\sqrt{f'_c}$ reach maximum values of 1.239, 1.678 and 1.779 in specimens C1A, C2-RT4 with four FRP prefabricated plates and C3-RT5 with five FRP prefabricated plates, respectively. Table 4 shows the comparison between experimental and analytical results of Xiao's columns. The maximum experimental lateral force of columns C1-A, C2-RT4 and C3-RT5 was 231, 290 and 330 kN while analysis gave the values of 228, 276 and 289 kN respectively. The differences between results are 1%, 5% and 10% respectively. It is found that in

both analytical and experimental result, the lateral force increased with increasing number of FRP prefabricated plates.

Table 4 Xiao's columns results

Specimen	Lateral force			Type of lap splice failure
	Exper.	Analysis	Differ.	
C1-A	231	228.0	0.99	Before yielding of bar
C2-RT4	290	276.0	0.95	After yielding of bar
C3-RT5	330	298.0	0.90	After yielding of bar
Average				0.95
S.D				0.04

Figure 18a, b, c show both analytical and experimental responses of columns C1-A, C2-RT4 and C3-RT5. As seen, the lateral capacity and the ductility of column strengthened by FRP wrapping around lap splice zone of RC column were increased. It is noted that the actual columns tested in Xiao and Ma [2] experiment was wrapped by FRP prefabricated plates not only around the lap splice zone, but also around the middle and top parts of column.

5 Conclusion

The nonlinear analytical model was constructed to simulate the cyclic response of RC columns strengthened by FRP confinement around lap splice zone. The model consisted of one elastic frame element and an assembly of nonlinear springs, to simulate the elastic zone of RC column as well as the plastic hinge region. The effect of FRP confinement around lap splice zone is taken into account by applying confined bond-slip model of spliced bars. Through the increase of FRP layers, the splice strength and ductility is enhanced.

The comparison between model predictions and experimental results shows a good agreement. The discrepancy is within the range of 10%. The analysis results can be used to predict the

responses of column with lap splice confined by FRP jacketing. The model is found to be widely applicable. It can be used to predict both rectangular and circular cross section.

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Nomenclature

The following symbols are used in this paper:

a	Shear span of column
A_g	Gross area of section
A_c	Area of concrete in a fiber
A_{cc}	Area of concrete core
A_s (ΣA_s)	Area of reinforcement (total in fiber)
A_{st}	Area of transverse reinforcement
b	Section width
b_f	Width of a FRP strip
c_b	Smaller of (1) distance from center of bar to the nearest concrete surface and (2) one-half center-to-center spacing of bars.
d	Effective depth of column section
d_b	Diameter of reinforced bar
E_c	Modulus of elasticity of concrete
E_f	Modulus of elasticity of transverse FRP
E_p	Modulus of plasticity of reinforcement
E_s	Modulus of elasticity of reinforcement
F_c	Yield force of concrete spring element
f'_c	Compressive strength of unconfined concrete
f'_{cc}	Compressive strength of confined concrete
f_f	Tensile strength of FRP
$f_{lf}; f_{ls}$	The lateral passive confining pressure

f_s	Stress of longitudinal reinforcement
f_t	Tensile strength of concrete
f_y	Yield stress of longitudinal reinforcement
F_y	Yield force of spring element
f_{yt}	Yield stress of transverse reinforcement
g	Gap provided at the bottom of the jacket
h	Section depth
H	Height of column
I_g	Moment of inertia of the gross section
k	Shear degrading coefficient
K	confined strength ratio
$k_1; k_2$	Confinement effectiveness coefficient
k_{tr}	Transverse reinforcement index
L_e	Length of elastic zone in bond model
L_p	Length of plastic hinge zone
L_s	Length lap spliced zone
L_{sa}	Length anchorage bar zone
L_y	Length of plastic zone in bond model
n_f	Number of transverse FRP layers
N_f	Number of partial FRP strips
n_s	Number of lap splices in tension
P	Axial load applied on column
s_0	Local slip factor
s_1	Local slip at peak pullout mode failure
s	Spacing between transverse hoops or spirals
s_f	Distance of FRP strips
s_{sp}	Local slip at peak bond strength
t_f	Thickness of one FRP layer
u_e	Average elastic bond stress
u_m	Maximum bond stress at pullout mode
u_{sp}	Local splitting bond stress
u_y	Average plastic bond stress
v	Shear strengthening coefficient of FRP
V_n	Shear strength of column without FRP
V_r	Remain shear strength at collapse

- V_u Shear strength of column with FRP
 Δ_r Lateral displacement at V_r
 Δ_s Bond slip of reinforcement
 Δ_u Lateral displacement at shear strength V_u
 α_f Factor of partial wrapping
 ε_{50} Concrete strain at stress of 50% of peak stress
 ε_{cc} Concrete strain for confined concrete
 ε_{co} Concrete strain at the intersection point between the 1st and 2nd stage of the stress-strain curve
 ε_e Elastic strain
 ε_o Strain at max stress of unconfined concrete
 ε_s Strain of reinforcement
 θ Critical crack angle at shear failure
 ψ_e Coating factor
 ψ_s Reinforcement size factor
 ψ_t Reinforcement location factor
 λ Lightweight aggregate concrete factor
 λ_f Parameter of ratio of FRP amount to concrete

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