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Revision of constitutive models for repairing bridge columns with fiber polymers

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Abstract

Federal Highway Network in Mexico has 7,230 bridges. More than two third parts were built in the period of 1960 to 1970 without considering in their design seismic loads or using design spectra with small amplitudes. The use of Fiber Reinforced Polymers (FRP) is a plausible alternative for retrofitting columns that have suffered some type of damage during a seismic event. In this paper, models of confinement with purpose of application in repairing damage of circular columns of bridges are analyzed. When evaluating different expressions for the confinement thickness of FRP in columns, a great dispersion exists. Each author of the revised models endorses analytically and experimentally its results, but for practical applications it is difficult to determine the appropriate model to be used.

Keywords: Repair, Bridges, Columns, Fiber Polymers.

1. Introduction

There is no doubt that the infrastructure, particularly for transportation, constitutes an essential element for economic growth, competitiveness and social integration of a country. In Mexico, the free toll Federal Highway Network has a longitude of 45,405.7 km with 7,230 bridges of which more than two third parts were built in the period of 1960 to 1970. In case of strong seismic event, it is necessary to carry out a prompt bridge repairs with the aim of guaranteeing the correct operation of the highway network. At present, it is well-known the effectiveness of the casing of concrete elements with Fiber Reinforced Polymers (FRP) as a seismic reinforcement alternative. However, its applicability for retrofitting bridge damaged columns after a seismic occurrence is currently under study in several parts of the world.

Currently bridge design recommendations suggest the use of the "non-collapse" criteria for the design earthquake. A non-linear bridge response is allowed, conducting to a great deal of damage, including cracking and detachment of the concrete, yielding of the steel reinforcement, and the possibility of rupture of the reinforcement bars. Therefore, after the occurrence of a seismic event a bridge column could be damaged but it would be theoretically repairable. The visual classification of the damage suffered by a column is the first step of the repair process; such classification provides the reference point for determining the condition of the column and the selection of the more appropriate repair technique.

2. Damage states

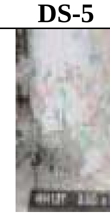
In the Concrete Repair Manual [ACI-440.2R Committee (2002)] there is a procedure to perform a visual inspection of the damage and the actual states of the damage. There are five states in such document (Damage States, DS) namely: DS-1 through DS-5. Damage state DS-1 corresponds to very slight damage with minimum cracking, characterized by the formation of cracks by bending and no visible spalling. Damage state DS-2 has slight damage and occurs when first spalling and shear cracks are visible. Damage state DS-3 is a moderate damage and is associated with extensive cracks and spalling. Damage state DS-4 is represented by a severe damage, in which the spiral and longitudinal reinforcing bars become visible and large cracks, holes and spalled areas are observed. Damage state DS-5 corresponds to very severe damage and is associated with imminent failure and cracks propagation inside the concrete core. Table 1 describes the characteristics of each aforementioned damage state as referenced in the Concrete Repair Manual [ACI, BRE (2001)]. In general, two parameters are considered to determine the possibility for a column to be repaired: (1) the deformations of the longitudinal reinforcement and (2) the deformations of the spiral hoop reinforcing bars. Strain quantification in the reinforcing bars is important for determining possible repair methods, i.e. if the longitudinal bars reach a rupture strain, repair via FRP jackets is not appropriated. Likewise, if the spiral hoops reach the yielding strain, damage to the cover core is expected. In this case, repairing with an FRP jacket is not appropriate.

Table 1. Visual Characterization of the Damage State

Damage	**Damage State**				
	1 (Very slight)	**2 (Slight)**	**3 (Moderate)**	**4 (Severe)**	**5 (Very severe)**
Crackswidth on concrete	< 0.10mm	0.10-0.30mm	0.30-1.0mm	1.0-3-0mm	>5mm
Cavities	Slightly visible	Visible	Holes with a diameter of 10mm	Holes with a diameter of 10-50mm	Holes with diameter greater than 50mm
Detachment of the concrete cover	Slightly visible	Visible	Greater than aggregate´s size	In areas greater than 150mm of cross-cut section	In areas greater than 150mm of cross-cut section

The strain levels in the reinforcement bars can be estimated based on the visual classifications associated with each damage state. It is expected that the longitudinal reinforcing bars yield once flexural cracks are visible, corresponding to DS-1. The longitudinal bars yield at this point because a plastic hinge is formed. Similarly, the spiral reinforcing bars would be expected to yield once the cracks in the concrete propagate into the column core, which corresponds to DS-5. Once the spiral reinforcement yields, FRP jacket repair in not effective. Table 2 shows a relationship between the levels of deformation in the reinforcement steel and the corresponding damage state; it also determines whether it is viable to perform repairs via FRP bands.

Table 2. Parameters of the damage response in relation to the damage state

Damage State	**Level**	**DS-1**	**DS-2**	**DS-3**	**DS-4**	**DS-5**
						
Maximum strain in longitudinal reinforcement	Upper Limit	0.00214	0.0132	0.0194	0.0231	0.0289
	Average	0.00917	0.0183	0.0263	0.0348	0.0425
	Lower Limit	0.0162	0.01235	0.0332	0.0466	0.0561
Maximum strain in spiral reinforcement (microstrains)	Upper Limit	0.000172	0.000419	0.000632	0.00113	0.00143
	Average	0.000307	0.000694	0.00108	0.00167	0.00307
	Lower Limit	0.000442	0.000970	0.00152	0.00222	0.00472
Repair work with FRP?		No	Yes			No

3. Confinement models for circular columns

It is well known that the restraint provided by an external jacket, which restricts the transverse dilation of the column, produces some confinement and increases the strength and deformation capacity of the element. Even though a reliable and accurate model of concrete confinement is not yet fully accepted by the researcher's community. Many models of confined strength of concrete and associated strain have been proposed dating back to Richart et al. (1928). Fardis and Khalili (1982) suggested that Richart et al. model could be directly used for FRP confined concrete. However, recent studies revealed that existing models for the axial compressive strength of steel confined concrete are unconservative and cannot be used for FRP confined concrete [Teng et al. (2002)]. For a circular column, the lateral confining pressure of the FRP jacket, f_l, can be evaluated by the free-body diagram shown in Figure 1.

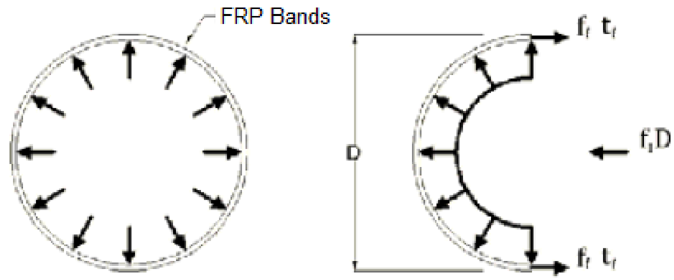


Figure 1. Free-body diagram to calculate the confinement forces

From Figure 1 the following equation is used to calculate the side confinement pressure provided by the FRP bands f_l,

$$f_l = \frac{2t_f f_f}{D} = \frac{2t_f E_f \varepsilon_f}{D}. \tag{1}$$

where t_f is the thickness of the reinforcement FRP bands, f_f is the tension stress of the FRP bands in the radial direction and D is the diameter of the concrete confined section; E_f and ε_f are the elastic module and the unitary deformation of the FRP bands in the radial direction, respectively. Because the FRP behaves elastically up to rupture, the pressure held by the reinforcement bands is proportionally increased, in contrast with steel strands that provides a constant side confinement pressure.

In existing reinforced concrete columns with insufficient transversal reinforcement and/or poor seismic detailing, three different types of failure modes can be observed under seismic excitation namely: shear failure mechanism, plastic hinge formation and insufficient development length (Fig. 2). The first and most critical failure mode is the shear failure, where inclined cracking, cover concrete spalling, and rupture or opening of the transverse reinforcement can lead to brittle or explosive column failures. The failure sequence consists of 5 steps, namely: (1) the development of inclined cracks once the tensile strength of the concrete is exceeded, (2) the opening of inclined or diagonal cracks in the column and onset of cover concrete spalling; (3) rupture or opening of the transverse or horizontal reinforcement; (4) buckling of the longitudinal column reinforcement, and (5) disintegration of the column concrete core.

Shear failure mechanism Formation of plastic hinge Insufficient development length
Figure 2. Common failures in columns that occurred in 1989 during the Loma Prieta Earthquake

The purpose of the seismic repair work is to improve the seismic performance of the columns by increasing their ductility and their resistance to flexural, shear and axial stresses. An FRP jacket provides confinement increasing the bending stress capacity, the concrete's ultimate strain and providing lateral support. The jacket with FRP bands may contribute to the shear stress capacity of the column and thus inhibit this type of failure. Jacket with FRP bands in columns inhibits the failure for lack of overlapping through the application of confinement pressure on the zone where the failure may occur. It increases the confinement side force and prevents the detachment of the concrete. The failure mode for lateral buckling of the longitudinal reinforcement is considered an uncommon mode of failure as a consequence of the close stirrup spacing recommended in many design codes. In literature there are several diverse confinement models for reinforced concrete circular columns with FRP jacket. Hernandez et al. (2009) summarized expressions of 20 different types of confinement models and described the experimental tests conducted by several authors. Such models are expressed in function of stresses (Table 3) or ultimate strains (Table 4).

Table 3. Summary of confinement models based on stresses [Hernández, *et al*. (2009)]

Author	Value of f'_{cc}	
Richart *et al*. (1928)	$f'_{cc} = f'_{co} + 4.1 f_l$	
Fardis and Khalili (1982)	$f'_{cc} = f'_{co} + 3.7\left(\frac{f_l}{f'_{co}}\right)^{0.86}$	
Mander *et al*. (1988)	$f'_{cc} = f'_{co}\left[-1.254 - 2\left(\frac{f_l}{f'_{co}}\right) + 2.254\left[1 + 7.94\left(\frac{f_l}{f'_{co}}\right)\right]^{0.5}\right]$	
Cusson and Paultre (1995)	$f'_{cc} = f'_{co}\left[1 + 2.1\left(\frac{f_l}{f'_{co}}\right)^{0.70}\right]$	
Karbhari and Gao (1997)	$f'_{cc} = f'_{co}\left[1 + 2.1\left(\frac{f_l}{f'_{co}}\right)^{0.87}\right]$	Empirical expression
	$f'_{cc} = f'_{co} + 3.1 f'_{co} v_c \left(\frac{2t_f E_f}{DE_c}\right) + 2\left(\frac{\sigma_t t_f}{D}\right)$	Simplistic analysis
Miyauchi *et al*. (1997)	$f'_{cc} = f'_{co} + 3.485 f_l$	
Pilakoutas and Mortazavi (1997)	$f'_{cc} = f'_{co}\left(1.125 + 2.5\frac{f_l}{f'_{co}}\right)$ for $2\frac{f_l}{f'_{co}} \geq 0.10$	
	$f'_{cc} = f'_{co}\left(1 + 5\frac{f_l}{f'_{co}}\right)$ for $2\frac{f_l}{f'_{co}} \langle\ 0.10$	
Kono *et al*. (1998)	$f'_{cc} = f'_{co} + 0.0572 f'_{co} f_l$	
Samaan *et al*. (1998)	$f'_{cc} = f'_{co} + 6.0\left(f_l\right)^{0.70}$	
Spoelstra and Monti (1999)	$f'_{cc} = f'_{co}\left[0.20 + 3.0\left(\frac{f_l}{f'_{co}}\right)^{0.50}\right]$	
Toutanji (1999)	$f'_{cc} = f'_{co}\left(1 + 3.5\left(\frac{f_l}{f'_{co}}\right)^{0.85}\right)$	
Saafi *et al*. (1999)	$f'_{cc} = f'_{co}\left(1 + 2.2\left(\frac{f_l}{f'_{co}}\right)^{0.84}\right)$	

Table 3. Summary of confinement models based on stresses (*Continued*).

Author	Value of f'_{cc}	
Xiao and Wu (2000)	$f'_{cc}=f'_{co}\left[1.1+\left(\frac{f_l}{f'_{co}}\right)\left(4.1-0.75\left(\frac{\left[f'_{co}\right]^2 D}{2E_f t_f}\right)\right)\right]$	
Thériaul and Neale (2000)	$f'_{cc}=f'_{co}\left(1+2\frac{f_l}{f'_{co}}\right)$	
Lam and Teng (2002)	$f'_{cc}=f'_{co}+2.0f_l$	
Wu *et al.* (2003)	$f'_{cc}=f'_{co}+2.0f_l$	When the stress of the FRP is determined experimentally
	$f'_{cc}=f'_{co}+3.0f_l$	FRP material
	$f'_{cc}=f'_{co}+2.5f_l$	FRP tube
Xiao and Wu (2003)	$f'_{cc}=f'_{co}\left[1.1+\left(\frac{f_l}{f'_{co}}\right)\left(4.1-0.45\left(\frac{\left[f'_{co}\right]^2 D}{2E_f t_f}\right)^{1.40}\right)\right]$	
Bisby *et al*. (2005)	$f'_{cc}=f'_{co}\left[1+2.425\frac{f_l}{f'_{co}}\right]$	
	$f'_{cc}=f'_{co}\left[1+2.217\left(\frac{f_l}{f'_{co}}\right)^{0.911}\right]$	
	$f'_{cc}=f'_{co}\left[1+3.587\left[f_l\right]^{0.84}\right]$	
Guralnick and Gunawan (2006)	$f'_{cc}=f'_{co}\left[0.616+\left(\frac{f_l}{f'_{co}}\right)+1.57\left(\frac{f_l}{f'_{co}}+0.06\right)^{0.50}\right]$	
Youssef *et al*. (2007)	$f'_{cc}=f'_{co}\left(1+2.25\left(\frac{f_l}{f'_{co}}\right)^{1.25}\right)$	
Girgin (2009)	$f'_{cc}=f'_{co}+2.109f'_{co}\left(\frac{f_l}{f'_{co}}\right)^{0.783}$	Based on the Mohr-Coulomb model
	$f'_{cc}=f_l+\sqrt{\left[f'_{co}\right]^2+3.5f'_{co}f_l}$	Based on the Hoek-Brown model

Where f'_{cc} is the strength of confined concrete at failure; f_l is the lateral confining pressure; f'_{co} is the strength of the unconfined concrete; ε_{cc} is the strain corresponding to the confined strength; ε_{co} is the longitudinal strain of the unconfined concrete at failure, which is typically assumed to be 0.002; E_f is the modulus of elasticity of the FRP jacket; ν is the Poisson´s ratio; ε_f is the strain in the FRP jacket; E_{eff} is the effective modulus of elasticity of the FRP in the hoop direction; E_c is the modulus of elasticity of the concrete; t_f is the thickness of the FRP band and D is the column diameter.

Table 4. Confinement models based on ultimate strain [Hernández, *et al*. (2009)]

Autor	**Valor de** ε_{cc}	
Richart *et al.* (1928)	$\varepsilon_{cc} = \varepsilon_{co}\left(1 + 20.5\dfrac{f_l}{f'_{co}}\right)$	
Fardis and Khalili (1982)	$\varepsilon_{cc} = \varepsilon_{co} + 0.0005\left(\dfrac{E_f t_f}{D f'_{co}}\right)$	
Mander *et al.* (1988)	$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\dfrac{f'_{cc}}{f'_{co}} - 1\right)\right]$	
Cusson and Paultre (1995)	$\varepsilon_{cc} = \varepsilon_{co} + 0.21\left(\dfrac{f_l}{f'_{co}}\right)^{1.70}$	
Karbhari and Gao (1997)	$\varepsilon_{cc} = \varepsilon_{co} + 0.01\left(\dfrac{f_l}{f'_{co}}\right)$	Empirical expression
Karbhari and Gao (1997)	$\varepsilon_{cc} = 1 - \left[\dfrac{1.004}{\left(1+\varepsilon_f\right)^2}\right]\left[1 - \dfrac{f'_{co}}{E_{eff}} - 8.2\dfrac{t_f E_f f'_{co} v_c}{D\, E_c E_{eff}}\right]$	Simplistic analysis
Miyauchi *et al.* (1997)	$\varepsilon_{cc} = \varepsilon_{co} + 10.6\left(\dfrac{f_l}{f'_{co}}\right)^{0.373}$	for $f'_{co} = 30MPa$
Miyauchi *et al.* (1997)	$\varepsilon_{cc} = \varepsilon_{co} + 10.5\left(\dfrac{f_l}{f'_{co}}\right)^{0.525}$	for $f'_{co} = 50MPa$
Kono *et al.* (1998)	$\varepsilon_{cc} = \varepsilon_{co} + 0.28\varepsilon_{co} f_l$	
Samaan *et al*. (1998)	$\varepsilon_{cc} = \dfrac{f'_{cc} - f_o}{E_2}$ where: $f_o = 0.872 f'_{co} + 0.371 f_l + 6.258$ $E_2 = 245.61\left[f'_{co}\right]^{0.20} + 1.3456\dfrac{E_f t_f}{D}$	
Spoelstra and Monti (1999)	$\varepsilon_{cc} = \varepsilon_{co}\left(2 + 1.25\dfrac{E_c}{f'_{co}}\varepsilon_f\left[\dfrac{f_l}{f'_{co}}\right]^{0.50}\right)$ where $\varepsilon_{co} = 0.002$	
Toutanji (1999)	$\varepsilon_{cc} = \varepsilon_{co}\left[1 + \left(1.90 + 310.57\varepsilon_f\right)\left(\dfrac{f'_{cc}}{f'_{co}} - 1\right)\right]$	
Saafi *et al.* (1999)	$\varepsilon_{cc} = \varepsilon_{co}\left[1 + \left(2.60 + 537\varepsilon_f\right)\left(\dfrac{f'_{cc}}{f'_{co}} - 1\right)\right]$	
Lam and Teng (2002)	$\varepsilon_{cc} = \varepsilon_{co}\left[2 + k_2\left(\dfrac{f_l}{f'_{co}}\right)\right]$ where $k_2 = 15$ for CFRP	

Table 4. Confinement models based on ultimate strain (*Continued*)

Autor	Valor de ε_{cc}		
De Lorenzis and Tepfers (2003)	$\varepsilon_{cc} = \varepsilon_{co}\left[1+26.2\left(\frac{f_l}{f'_{co}}\right)^{0.80}\left[\frac{t_f E_f}{D}\right]^{-0.148}\right]$		For FRP belts
	$\varepsilon_{cc} = \varepsilon_{co}\left[1+26.2\left(\frac{f_l}{f'_{co}}\right)^{0.68}\left[\frac{t_f E_f}{D}\right]^{-0.127}\right]$		For FRP tubes
Wu *et al.* (2003)	$\varepsilon_{cc} = \frac{\varepsilon_f}{v_u}$	For belts of CFRP, GFRP and AFRP with normal module:	$v_u = 0.56\left(\frac{f_l}{f'_{co}}\right)^{-0.66}$
		Por GFRP or CFRP tubes.	$v_u = 0.31\left(\frac{f_l}{f'_{co}}\right)^{-0.44}$
		For FRP belts with high module.	$v_u = 0.56 k_f\left(\frac{f_l}{f'_{co}}\right)^{-0.66}$
	Where: $k_f = 1.0$ for $E_f \le 250GPa$		
Wu *et al.* (2003)	$k_f = \sqrt{\frac{250}{E_f}}$ for $E_f \rangle\ 250GPa$		
Bisby *et al.* (2005)	$\varepsilon_{cc} = \varepsilon_{co} + k_2\left(\frac{f_l}{f'_{co}}\right)$		CFRP: $k_2 = 0.0240$
			GFRP: $k_2 = 0.0137$
			AFRP: $k_2 = 0.0536$
Youssef *et al.* (2007)	$\varepsilon_{cc} = 0.003368 + 0.259\left(\frac{f_l}{f'_{co}}\right)\left(\frac{f_f}{E_f}\right)^{0.50}$		

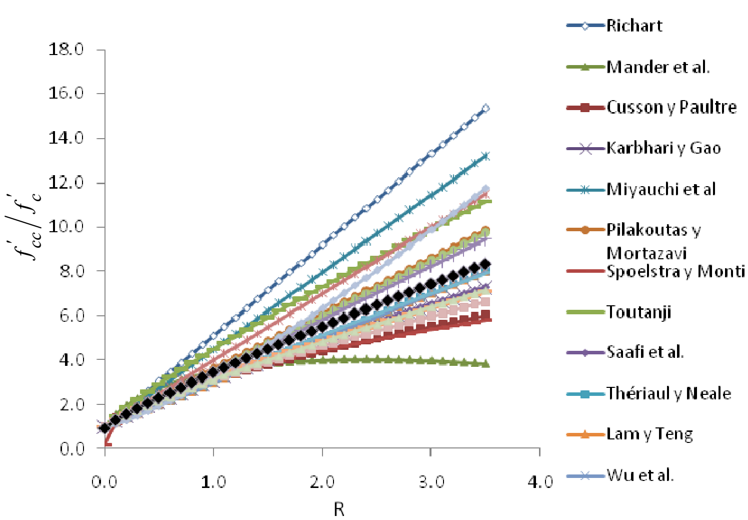


Figure 4. Volumetric relationship R for different confinement models.

With the purpose of determining the model that can be employed in a fast repair work of bridge columns after an earthquake occurrence, the existing models in the literature are displayed in figure 4. A great dispersion dispersion of the confined sgtrength were obtained for different confinement models as function of the volumetric relationship, $R = f_l / f'_{co}$. All authors justify their proposed expressions by the use of experimentally and analytically models. However, Figure 4 shows the large variations among them. The International Federation for Structural Concrete [FIB (2003)] recommends the use of confinement model proposed bySpoelstra and Monti (1999) due to its simplicity and accuracy.

4. ACI 440.2 R-02 Recommendations (2002)

The ACI 440.2R-02 (2002) guideline makes use of the confinement model originally proposed by Mander et al. (1988), derived for steel hoop reinforcement. The Mander's confinement model is slightly modified to account for the linearly elastic behavior of FRP. The confinement model predicts the confined strength of concrete, f'_{cc}, through the following relation:

$$f'_{cc} = f'_{co}\left[2.25\sqrt{1+7.9\frac{f_l}{f'_{co}}} - 2\frac{f_l}{f'_{co}} - 1.25\right]. \tag{2}$$

Where f'_{co} is the unconfined concrete strength and f_l is the confining pressure, which is defined as follows:

$$f_l = \frac{k_s\, \rho_f\, \varepsilon_f\, E_f}{2}. \tag{3}$$

k_S is the efficiency coefficient, which depends on the column geometry. For a circular column, $k_S = 1.0$; ε_f is defined as the smaller value of 0.0004 or 0.75 ε_{fu}, where ε_{fu} is the ultimate strain in the FRP. This strain limit was chosen to avoid the aggregate interlock loss in the concrete and was determined trough pure axial tests. It is, therefore, an approximation for combined axial and bending situations induced by seismic forces. The volumetric relationship of the FRP is equal to:

$$\rho_f = \frac{4\, t_f}{D}. \tag{4}$$

Equation (3) can also be written as:

$$f_l = \frac{2\, k_s\, t_f\, \varepsilon_f\, E_f}{D}. \tag{5}$$

The increased axial load carrying capacity of a strengthened column can be expressed as:

$$P_n = k_e\, \varphi\left[0.85\, \psi_f\, f'_{cc}\left(A_g - A_{st}\right) + f_y A_{st}\right]. \tag{6}$$

Where $k_e = 0.85$ for spiral reinforced columns and $k_e = 0.80$ for tie reinforced columns; P_n is the nominal axial load carrying capacity, $\varphi = 0.75$ is the strength reduction factor; $\psi_f = 0.95$ is the additional reduction coefficient for FRP wrapped columns, $A_{st} = 1.0$ is the longitudinal steel area; A_g is the concrete total area and f_y is the steel yield strength.

The Equation (6) only considers the axial increase in strength. For retrofitting in seismic areas, ACI 440 suggests using Mander et al. (1988) model for confined strain, ε_{cc},

$$\varepsilon_{cc} = \frac{1.71\left[5 f'_{cc} - 4 f'_{co}\right]}{E_c}. \tag{7}$$

Where E_C is the modulus of elasticity of concrete. ACI 440 suggests that the design should be developed to have sufficient strain capacity associated to the desired displacement demands.

For each damage state, from DS-1 to DS-5, there is a deformation range for the transverse and longitudinal reinforcements (Table 2). Each deformation value can be associated with the axial capacity of the column through a sectional analysis of the element. The axial capacity for a certain level of deformation can be compared to the theoretical original capacity of the column prior to the damage, which can be determined using the following expression proposed by the ACI 318 (2008):

$$P_n = 0.80\varphi\left[0.85\left(f_c^{'}A_g + f_{ccs}^{'}A_{cc}\right) + f_y A_s\right]. \tag{8}$$

Where P_n is the axial capacity of the column, φ is the reduction factor of the transverse reinforcement, which is equal to 0.75 for closed stirrups, A_g is the net area or total area of de column section, A_{cc} is the confined area by the transverse reinforcement, f_y is the yielding stress of the longitudinal steel, A_s is the area of the longitudinal reinforcement and f'_{ccs} is the stress of the confined concrete produced by the transverse steel stirrups and can be determined with the expression of Richart et at. (1928), which is:

$$f_{ccs}^{'} = f_c^{'} + 4.1 f_l . \tag{9}$$

where f_l is the confinement pressure that results from the transverse reinforcement,

$$f_l = \frac{2A_h f_{sy}}{d\ s} . \tag{10}$$

A_h is the stirrups cross section area, f_{sy} is the yielding stress of the transverse reinforcement, d is the diameter of the column that is confined by the stirrups, which is equal to the diameter of the thick section, D, minus the double product of the concrete cover, cc, minus the double product of the stirrups diameter, dh. The spacing of stirrups is represented by s. The loss of axial capacity for each damage state is just the calculated theoretical axial capacity with equation (8) minus the axial capacity that the column would have for the deformation level at the corresponding damage, which is obtained from a sectional analysis.

With the loss of column axial capacity for a certain damage state, the thickness of the FRP band confinement necessary to restore the initial compression capacity of the column is obtained. The confinement effect of the FRP must be added to the equation to calculate the axial capacity of the column, so that the following equation is obtained:

$$P_n = 0.80\varphi\left[0.85\left(f_c^{'}A_g + f_{ccs}^{'}A_{cc} + f_{cc}^{'}A_g\right)\right]. \tag{11}$$

The effect of the longitudinal reinforcement, $f_y\ A_s$, is not taken into consideration because of the steel yielding at the deformations of the DS-1 damage state. Assessed the loss of axial load capacity for columns, the necessary FRP thickness can be calculated. The confinement stress of the concrete due to the FRP is determined with the following equation:

$$f_{cc}^{'} = \frac{P_{ini} - P_{dam}}{(0.8\varphi)\,0.85A_g} . \tag{12}$$

where P_{ini} is the initial theoretical load capacity of the column and P_{dam} is the axial load capacity of the element for the damage state and it is obtained from a sectional analysis of the element. The thickness of the FRP band is calculated with following equation:

$$t_f = \frac{D\rho_f}{4} . \tag{13}$$

where,

$$\rho_f = \frac{2f_l}{k_s\ \varepsilon_f\ E_f} . \tag{14}$$

In order to use equation 14, it is necessary to know the value of f_l, which can be determined for comparison purposes, with any of the confinement stress models as function of f_l. Equation (13) can be written as follows:

$$t_f = \frac{D}{2} \frac{f_l}{k_s \; \varepsilon_f \; E_f} . \tag{15}$$

5. Caltrans memo 20.4 recommendations (2000)

The Caltrans Memo 20-4 (2000) is specifically aimed at preventing flexure failure, and recommends the use of target confining stress, $f_l = 2068 \; kPa$ and radial dilating strain $\varepsilon_{cc} = 0.004$ inside the plastic hinge zone. These limiting values were determined experimentally at a corresponding displacement ductility, $\mu = 6.0$. For regions outside the plastic hinge zone, it is recommended to consider $f_l = 1034 \; kPa$ and $\varepsilon_{cc} = 0.004$. The FRP thickness can be calculated using the following equation, which was derived from equilibrium:

$$2 \, t_f \; E_f \; \varepsilon_{cc} = f_l \; D . \tag{16}$$

Then,

$$t_f = \frac{f_l \; D}{2(0.90) E_f \; \varepsilon_{cc}} . \tag{17}$$

Where E_f is the elastic module of FRP and a reduction safety factor of 0.90 is added. This approach is exclusively based on target confinement pressure and does not specifically account for various possible failure modes.

It is worth to mention that previous recommendations are valid for non slender columns. Hence, second order effects were neglected.

6. Example 1

In this section the expressions proposed by the ACI 440.2R-02 and the Caltrans Memo 20-4 (2000) are applied to calculate the FRP thickness and to assess the axial capacity of a circular column with: diameter $D = 305mm$, height of $2438mm$, concrete cover of $cc = 19 \; mm$, unconfined concrete stress $f'_c = 34.45MPa$, and an elastic module of $E_c = 27.58MPa$. The column has an FRP casing with the following properties: Elastic modulus of $E_f = 59.36MPa$ and tensile strength of $f_f = 752MPa$. The ultimate deformation of the FRP is:

$$\varepsilon_{fu} = \frac{f_f}{E_f} = 0.01266\text{mm} . \tag{18}$$

The longitudinal reinforcement consists of 16 bars number 3, each with an area of $71mm^2$ for a total area of $A_s = 1135mm^2$; stirrups with a diameter of $d_h = 4.92mm$, an area of $A_h = 19mm^2$ spaced at $s = 32mm$. The yield stress of the steel is of $f_{yh} = 420MPa$.

The column of this example was experimentally tested and repaired by Vossoghi and Saiidi (2008), who used the expression of the Caltrans Memo 20-4 (2000) for repairing purposes and concluded that the design thickness of 2.03mm indeed restored the axial stress capacity of the damaged column and 87% of the column stiffness was restored.

The confinement pressure provided by the stirrups is the following:

$$f_h \approx f_l = 1.9405\text{MPa} . \tag{19}$$

The stress of confined concrete by the hoop steel f'_{ccs} can be calculated with one of the equations previously mentioned (Table 3).

The effective axial load of the column, P_{dam}, for the deformation state at the considered damage, is obtained from a sectional analysis of the column, for each damage level of the average deformation (Table 2). The deformation of the longitudinal steel for the damage state DS-4 is 0.0348 mm/m; a sectional analysis of the column [Response (2000)] showed that the resisting load for such deformation is of 173.03 kN.

Knowing the resistance of the damaged column, the necessary FRP thickness to restore its axial load capacity is determined. First, the value of the stress f'_{cc} is calculated and then, with the stress and deformation relationship, the confinement stress f_l of the concrete due to the FRP is determined. Several expressions proposed by different authors can be used (Table 3), considering that the unconfined concrete stress is $f'_{co} \approx f'_c$. The following expression [Karbhari (2005)] is used for comparison purposes.

$$\frac{t_f}{2} = \frac{Df_l}{4\,k_s\;\varepsilon_f\;E_f}\,. \tag{20}$$

where $k_s = 1.0$ (circular column); ε_f is defined as the lower value of 0.0004 or $0.75\varepsilon_{fu}$, where ε_{fu} is the ultimate FRP deformation, $\varepsilon_{fu} = 0.01266mm$; i.e. $\varepsilon_f = 0.0004$; equation (17) is applied to calculate the confinement thickness, considering $\varepsilon_{cc} = 0.004$.

Table 5 presents the required FRP thickness for a damage state DS-4, to restore the axial column capacity obtained with the constitutive models proposed by several authors (Table 3). The thickness was computed with the expressions (19) and (20), for comparison purposes only. Youssef et al. (2007) equation conducts to the largest value, while the smallest one is obtained with the Bisby et al. (2005) expression. The confinement thickness obtained with the expressions (17) and (20) are in all cases different.

The results obtained with the equations suggested by the remaining authors provide more conservative values of the FRP thickness bands. Equation (20) results are in the range of 4.9 to 10.1mm, while equation (17) results are in the range of 5.4 to 11.2mm.

Considering that the expressions suggested by Mander et al. (1988) are widely recommended by most design codes, it can be deducted that the expressions proposed by other authors can be conservative, such is the case of Thériaul and Neale (2000), Lam and Teng (2002), and Wu et al. (2003).

Table 5. Values of the FRP thickness using the equations of different authors

Author	f'_{ccs}	P_{ini}	f'_{cc}	f'_{cca}	f_l	t_f **(mm)**	
						(20)	**(17)**
Richart *et al.* (1928)	42.406	2693.246	67.555	67.555	8.074	5.2	5.8
Mander *et al.* (1988)	46.333	2797.272	70.347	70.347	7.582	4.9	5.4
Cusson and Paultre (1995)	44.109	2738.346	68.766	68.766	11.870	7.6	8.5
Karbhari and Gao (1997)	40.373	2639.390	66.110	66.110	13.325	8.6	9.5
Miyauchi *et al.* (1997)	41.213	2661.633	66.707	66.707	9.256	5.9	6.6
Pilakoutas and Mortazavi (1997)	43.608	2725.071	68.409	68.409	11.861	7.6	8.5
Kono *et al.* (1998)	38.274	2583.786	64.618	64.409	15.203	9.8	10.8
Samaan *et al.* (1998)	43.993	2735.286	68.683	68.683	12.035	7.7	8.6
Spoelstra and Monti (1999)	31.419	2402.195	59.744	59.744	9.010	5.8	6.4
Toutanji (1999)	44.906	2759.473	69.333	69.333	8.007	5.1	5.7
Saafi *et al.* (1999)	41.214	2661.676	66.708	66.708	12.461	8.0	8.9
Thériaul and Neale (2000)	38.331	2585.300	64.658	64.658	15.104	9.7	10.8
Lam and Teng (2002)	38.331	2585.300	64.658	64.658	15.104	9.7	10.8
Wu *et al.* (2003)	38.331	2585.300	64.658	64.658	15.104	9.7	10.8
	40.272	2636.703	66.038	66.038	10.529	6.8	7.5
	39.301	2611.001	65.348	65.348	12.359	7.9	8.8
Bisby *et al.* (2005)	39.156	2607.146	65.245	65.245	12.699	8.2	9.1
	40.007	2629.705	65.850	65.850	12.985	8.3	9.3
	250.109	8195.178	215.212	215.212	1.573	1.0	1.1
Guralnick and Gunawan (2006)	41.609	2672.130	66.988	66.989	11.645	7.5	8.3
Youssef *et al.* (2007)	36.577	2538.839	63.411	63.411	15.673	10.1	11.2
Girgin (2009)	42.090	2684.869	67.330	67.330	12.515	8.0	8.9
	39.634	2619.808	65.584	65.584	13.048	8.4	9.3

7. Seismic Reinforcement

The approach proposed by Seible et al. (1997) and Karbhari (2005) differs from most of the aforementioned documents because it considers each failure mode separately and calculates a design thickness associated with each of the failure modes.

(i) Required thickness to prevent shear failure is calculated as follows:

$$t_j = \frac{\frac{V_0}{\varphi_v} - \left(V_c + V_s + V_p\right)}{0.004\left(\frac{\pi}{2}\right)E_f D}. \tag{21}$$

where V_0 is the column shear demand based on full flexural over-strength at the potential plastic hinge region, $\varphi_v = 0.85$ is the shear capacity reduction factor, V_c is the shear capacity contribution from concrete, V_s is the shear capacity contribution from horizontal steel reinforcement and V_p is the shear capacity contribution from axial load.

The shear demand, V_0, is calculated from the as built moment capacity, M_{yi}; Therefore, the shear demand is expressed as:

$$V_0 = \frac{1.5M_{yi}}{H}. \tag{22}$$

where H is the height of the column. The shear concrete contribution can be calculated with:

$$V_c = k\sqrt{f'_{co}}A_e. \tag{23}$$

In this equation, $k = 0.50$ inside the plastic hinge and $k = 3.0$ outside of it, and A_e is the effective area, which can be considered as the 80% of the column gross area. However, in damaged columns the concrete contribution should be neglected, since the loose of aggregate interlock can be presented.

The shear contribution from the steel hoop reinforcement is calculated from,

$$V_s = \frac{\pi}{2}\frac{A_h f_y D' \cot\theta}{s}. \tag{24}$$

where A_h is the area of hoop steel reinforcement and θ is the angle of the shear crack, which can conservatively be assumed as 45^o, s is the spacing between steel hoop and D' is the effective diameter, which is:

$$D' = D - 2cc - 2d_h. \tag{25}$$

where, D, is the overall diameter, cc, is the concrete cover and d_h is the diameter of the steel hoop bar. The shear contribution from the axial load is determined by:

$$V_p = \frac{P\left(D-c\right)}{H}. \tag{26}$$

where P is the applied axial load and c is the neutral axis depth. The required thickness is applied over the shear reinforcement length, L_v, from either column ends, where L_v is equal to 1.5 times the diameter of the column, D.

(ii) For the flexural hinge failure mode

The thickness in the flexural hinge region is calculated as follows:

$$t_j = 0.09 \frac{D\left(\varepsilon_{cu} - 0.004\right) f'_{cc}}{\varphi_f f_f \varepsilon_f}. \tag{27}$$

where $\varphi_f = 0.90$ is the flexural capacity reduction factor, conservatively it is assumed that $f'_{cc} = 1.5\, f'_{co}$ and ε_{cu} is the ultimate concrete strain that depends on the level of confinement, and is calculated as:

$$\varepsilon_{cu} = 0.004 + \frac{2.8 \rho_j f_f \varepsilon_f}{f'_{cc}}. \tag{28}$$

where ρ_j is the volumetric jacket reinforcement ratio; and ε_{cu} can be obtained from:

$$\varepsilon_{cu} = \Phi_u c_u. \tag{29}$$

c_u is the neutral axis depth and Φ_u is the ultimate section curvature and it can be obtained from the ductility factor equation:

$$\mu_\Delta = 1 + 3\frac{L_p}{L}\left(\frac{\Phi_u}{\Phi_y} - 1\right)\left(1 - 0.5\frac{L_p}{L}\right). \tag{30}$$

where Φ_y is the section yield curvature and L_p is the plastic hinge length, determined from:

$$L_p = 0.08L + 0.022 f_y d_b. \tag{31}$$

The thickness required for the flexural reinforcement jacket is applied over the flexural plastic hinge region, where L_{c1} is the primary flexural hinge region, equal to the greater value of $0.5D$ or $L/8$; L_{c2} is the secondary flexural hinge region and is equal to the greater value of $0.5D$ or $L/8$. The secondary flexural hinge region has $t/2$ thickness.

(iii) The lap slice failure mode is prevented through applying a FRP thickness in the lap splice region of:

$$t_j = 500 \frac{D\left(f_l - f_h\right)}{E_f}. \tag{32}$$

where f_h is the horizontal stress provided by the existing hoop reinforcement at a strain of 10% and it is calculated with the equation (33),

$$f_h = \frac{0.002 A_h E_s}{s\, D}. \tag{33}$$

and f_l is the lateral clamping pressure, defined as,

$$f_l = \frac{A_s f_y}{\left[\frac{p}{2n} + 2\left(d_b + cc\right)\right] L_s}. \tag{34}$$

where p is the perimeter line in the column cross section along the lap-spliced bar locations, n is the number of spliced bars along p; A_s is the area of one main column reinforcing bar, cc is the concrete cover to the main column reinforcement and d_b is the diameter of the main column reinforcement bars. The lap splice reinforcement length, L_s, must be greater than the lap length itself.

8. Example 2

In Seible et al. (1995) a column that is deficient in bending is reported. It had a limited development length and it was reinforced using FRP bands with a modulus of elasticity, $E_f = 124.1\ GPa$ and ultimate stress of $f_f = 1.31\ GPa$; the ultimate strain is:

$$\varepsilon_f = \frac{f_f}{E_f} = \frac{1.31GPa}{124.1GPa} = 0.0106 \approx 1.10\% . \qquad (35)$$

Table 6 shows the columns loads and properties.

Table 6. Properties of the column with bending resistance deficiency [Seible *et al.* (1995)]

Column section properties	Column height, H	3.658m
	Column diameter , D	0.61m
	Concrete cover, cc	19.05mm
	Concrete strength, $f_c^{'}$	34.45MPa
Longitudinal reinforcement (grade 40)	Bars diameter, d_b (26 total)	19mm
	Bar area , A_s	284mm2
	Yield strength f_y	303.4MPa
Transverse reinforcement (grade 40)	Bar diameter, d_h	6.35mm
	Bar area, A_h	31.7mm2
	Spacing, s	127mm
Column load properties	Axial load, P	1800kN
	Moment capacity, M_{yi}	518.6kNm
	Yield curvature, φ_y	0.008196 1/m
	Neutral axis, c_u	136.4mm

The ACI 440.2R (2002) incorporates in the seismic reinforcement process the constitutive model as function of stresses, f'_{cc}, as well as the model in function of the strain, ε_{cc}. The suggested process by ACI 440.2R (2002) and the different models of confinement suggested by several authors (Tables 3 and 4) are used. It is assumed that a ductility of $\mu_\Delta = 10$ is required, as it is usual in reinforced concrete columns for bridges. The design basic values are shown in Table 7.

Table 7. Design values of example 2

Column	Lp (mm)	μ_Φ		Φ_y (1/m)	c_u (mm)	ε_{cc} (mm)	
		$\mu_\Delta = 8$	$\mu_\Delta = 10$			$\mu_\Delta = 8$	$\mu_\Delta = 10$
Example 2	273.14	17.89	22.71	0.008196	136.4	0.01999	0.0254

By applying the shear design procedure suggested by Seible et al. (1995) the design values of Table 8 were obtained. As it can be observed, the column does not present shear stress problems.

Table 8. Design values by shears stress

Column	M_{yi} (kN m)	V_0 (kN)	P (kN)	D' (mm)	V_s (kN)	V_p (kN)	t_j (mm)
Example 2	518.6	216.66	1800.0	565.55	67.276	233.04	.0001

For bending reinforcement, the FRP thickness in the zone of plastic articulation is calculated with:

$$t_j = 0.09 \frac{D\left(\varepsilon_{cu} - 0.004\right) f'_{cc}}{\varphi_f f_f \varepsilon_f}. \tag{36}$$

where $\varphi_f = 0.90$ is the resistance reduction factor. Considering $f'_{cc} = 1.5 f'_{co}$, $f_f = 1.30\ GPa$ and the ultimate strain of $\varepsilon_f = 0.0106$, the confinement thicknesses obtained are summarized in Table 9.

Table 9. Design of FRP thickness for bending

Column	ε_{cu} (mm)		t_j (mm)	$t_j/2$ (mm)	t_j (mm)	$t_j/2$ (mm)
	$\mu_\Delta = 8$	$\mu_\Delta = 10$				
Example 2	0.01999	0.0254	9.6	4.8	12.9	6.4

Determining the value of ε_{cc} (table 7), the required confinement stress f'_{cc} or f_l is calculated (Table 3), that is,

$$f'_{cc} = \frac{\frac{\varepsilon_{cc} E_c}{1.71} + 4 f'_c}{5}. \tag{37}$$

f_l depends on the stresses and the FRP thickness is calculated with the following equation,

$$t_f = \frac{D}{2} \frac{f_l}{k_s\ \varepsilon_f\ E_f}. \tag{38}$$

Table 10 shows the required thickness of the confinement obtained for a ductility capacity of 10 with the models suggested by some of the previously mentioned authors. The confinement thickness must be compared with the value defined with the ACI methodology of 12.9mm, reported on Table 9.

It can be observed that a large dispersion exists in the results obtained for the FRP thickness by using the strain equations of different authors (table 10). The FRP thickness values for a deformation of $\varepsilon = 0.0173$ mm/m, varied in the range of 4.7 to 18.7mm and for a deformation of $\varepsilon = 0.0254$ mm/m the thickness values of the FRP bands are within a range of 6 to 29mm. The third column of Table 10 shows the percentage of the relative error, considering as the correct one, the thickness obtained with the procedure suggested by Seible *et al.* (1997).

Table 10. FRP thickness of confinement for $\mu_\Delta = 10$

Author	$\varepsilon_{cc} = 0.0173$	$\varepsilon_{cc} = 0.0254$	Δ(%)
	t_f (mm)		
Richart *et al.* (1928)	7.9	12	-7
Mander *et al.* (1988)	8.4	18	47
Cusson and Paultre (1995)	4.7	6	-53
Kono *et al.* (1998)	16.8	26	102
Toutanji (1999)	13.7	23	81
Saafi *et al.* (1999)	18.7	24	86
Lam and Teng (2002)	9.4	15	16
Bisby *et al.* (2005)	13.5	21	63
	18.7	29	125
	6.0	9	-30
Youssef *et al.* (2007)	11.1	18	40

9. Conclusions

The structural retrofit and repair using Polymer Fibers (FRP, Fiber Reinforced Polymers) has increased in recent years. For the case of columns that have suffered some type of damage during an earthquake occurrence, the application of the FRP jacket technique is a rapid and efficient alternative. This study presents the possible application of different FRP jackets for repairing the seismic damage of circular columns. First, it is determined the damage state of a column according to the Repair of Concrete Structures Manual [ACI, BRE, Concrete Society and ICRI (2001)]. After the damage state has been specified, the possibility of performing its repair by means of FRP jacket is determined. When evaluating different expressions for calculation the FRP thickness of confinement, it was found that a great dispersion exists in spite of the analytical and experimental background of each author proposals. In a practical application, it is therefore difficult to determine the best model to be used.

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