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PERFORMANCE-BASED DESIGN OF FRP JACKETS FOR PLASTIC HINGE CONFINEMENT OF CONCRETE COLUMNS

D. A. Moran[1] and C.P. Pantelides[2]

ABSTRACT

A design procedure is presented for predicting the behavior of circular or rectangular concrete columns confined with either bonded or non-bonded fiber reinforced polymer (FRP) composite jackets. In this procedure the mechanical properties of the FRP jacket required to achieve the target displacement ductility during a seismic event are determined. A series of relationships are developed between the curvature ductility of the upgraded column and the target displacement ductility, the resultant column curvature, the axial strain in the column and the neutral axis location of the FRP upgraded column in single or double curvature. In the proposed design procedure, the design thickness of the FRP jacket is determined based on the target ultimate compressive strain and resultant dilation of the confined concrete core within the potential plastic hinge region of a circular or rectangular concrete column. Unlike other design procedures, no consideration is given to the unknown increase in compressive strength due to the passive confinement provided by the FRP jacket; rather the design is based on the strain ductility increase provided by the confining FRP jacket and is thus a strain-based approach using performance-based design principles. The design procedure compares favorably with experimental results for columns in single curvature in the literature that were upgraded with FRP jackets and had demonstrated a substantial displacement ductility increase.

Introduction

The encasement of concrete in Fiber Reinforced Polymer (FRP) composite jackets can significantly increase the compressive strength and strain ductility of reinforced concrete columns, and the structural system the columns are part of, be it a building or a bridge. Analysis and design of FRP confined concrete members requires an accurate estimate of the performance enhancement due to the confinement provided by the FRP composite jackets. An analytical design procedure is presented for predicting the behavior of reinforced concrete columns confined with either bonded FRP-confined concrete (BFCC) sections, concrete filled FRP tubes (CFFT), or unbonded FRP composite jacketed sections. Rehabilitation of existing concrete

[1] Ph.D. Candidate, Dept. of Civil and Envr. Eng., Univ. of Utah, Salt Lake City, UT 84112
[2] Prof., Dept. of Civil and Envr. Eng., Univ. of Utah, Salt Lake City, UT 84112

structures using advanced FRP composite materials is gaining attention due to the need for seismic rehabilitation of the existing infrastructure. Rehabilitation is undertaken either for strengthening or upgrading the seismic performance of existing reinforced concrete buildings and bridges to significantly improve their axial, shear, and flexural behavior during a seismic event. In particular, buildings and bridges that were designed using outdated and inadequate seismic codes can benefit significantly from seismic rehabilitation using FRP composites.

The use of FRP composites for improving the shear strength and ductility capacity of reinforced concrete members, in particular the use of confinement systems utilizing FRP composite jackets, has become a popular structural rehabilitation option for the design engineer in regions of high seismicity. The presence of FRP composite jackets within the plastic hinge region of a reinforced concrete beam-column element can induce the development of ductile flexural behavior, while inhibiting premature lap splice, anchorage, or shear failure of a reinforced concrete column; this type of behavior is desirable for concrete sections subjected to cyclic lateral loads such as those that occur in a seismic event.

Strain-Based Design Procedure

Seible et al. (1997) introduced a strain energy-based design procedure and Monti et al. (2001), introduced a multivariate regression analysis-based upgrading index design procedure for the design of FRP jackets for plastic hinge confinement of reinforced concrete columns. In the analytical design procedure developed here, the performance enhancement in compressive strength and strain ductility of FRP-confined concrete is expressed in terms of an internal damage-based stress-strain model (Moran 2009). The proposed design methodology is different in that it is based on the strain ductility increase provided by the confining FRP jacket, and is thus a strain-based approach using performance-based design principles. The additional confinement and enhanced strain ductility provided by the available hoop reinforcement is ignored, because of the wide spacing and arrangement of the transverse steel, and because of possible corrosion damage of the hoop reinforcement. The design method is proposed for a concrete column in a rigid system which is modified to include the effects of system flexibility on the displacement demand imposed on the existing reinforced concrete column during a seismic event.

Consider the case of an existing reinforced concrete column with a height L_c. The displacement ductility of the existing column $\left(\mu_{\Delta_r}\right)_{ex}$ can be found by performing a moment curvature analysis of the reinforced concrete cross section. Assuming a bilinear behavior, in which linear elastic behavior occurs up to the stage of first yield and that plastic behavior (rotation) is concentrated at the center of the plastic hinge (Priestley and Park 1987), as shown in **Fig. 1**, the displacement ductility $\left(\mu_{\Delta}\right)_m$ of the concrete column can be approximated by:

$$\left(\mu_{\Delta}\right)_m = \left(\Delta_u / \Delta_y\right)_m = \bar{M}_m + \frac{1}{C_\Phi}\left[\left(\mu_\Phi\right)_m - 1\right]\ 3\lambda_p\left(1 - 0.5\lambda_p / C_\Phi\right) \qquad (1)$$

$$\left(\mu_\Phi\right)_m = \left(\Phi_u / \Phi_y\right)_m = 1 + \left(\Phi_p / \Phi_y\right)_m;\ \ \lambda_p = 0.12 C_\Phi + 0.014\alpha_s\left(f_{ye} d_{bl} / L_c\right) \qquad (2)$$

where the subscript m indicates two different conditions: $m = ex$ indicates an existing column and $m = up$ indicates the upgraded column; $C_\Phi = (L_v / L_c)$ is the column curvature coefficient: for single curvature bending $C_\Phi = 1.0$, and for double curvature bending $C_\Phi = 0.50$; $(\Delta_u)_m$ and $(\Delta_y)_m$ are the analytical ultimate and yield displacement of the column, respectively; $\bar{M}_m$ is the moment capacity ratio of the column: for an existing column $\bar{M}_{ex} = (M_u / M_y)_{ex}$, and for an upgraded column $\bar{M}_{up} = (M_u)_{up} / (M_u)_{ex}$; $(\mu_\Phi)_m$ is the curvature ductility factor of the column; $(\Phi_u)_m$ and $(\Phi_y)_m$ are the ultimate and yield curvature of the column section, respectively; $\lambda_p = (L_p / L_c)$ is the normalized plastic hinge length (Panagiotakos and Fardis 2001); L_p and L_v are the analytical plastic hinge length and column shear span, respectively. In addition, f_{ye} and d_{bl} are the expected yield strength and bar diameter of the longitudinal steel reinforcement, respectively; α_s is the reinforcing slippage coefficient: $\alpha_s = 1.0$ if slippage in the plastic hinge region is possible, and $\alpha_s = 0$ otherwise; the use of $\alpha_s = 1.0$ is recommended.

The displacement ductility $(\mu_{\Delta f})_m$ of the existing or upgraded column in a structural system with elastic flexibility, as shown in **Fig. 1(c)**, can be found in terms of the displacement ductility $(\mu_\Delta)_m$ of the rigid system of Eq. (1), as follows:

$$(\mu_{\Delta f})_m = 1 + C_s\left[(\mu_\Delta)_m - 1\right] \tag{3}$$

$$C_s = (1 + c_p c_e)/(1 + c_e) \ ; \ c_e = \Delta_{es}/\Delta_y \ ; \ c_p = \left(\bar{M}_{ex} - 1\right) \Big/ \left[(\mu_\Delta)_{ex} - 1\right] \tag{4}$$

where Δ_y is the analytical column yield displacement in a rigid system and Δ_{es} is the elastic displacement due to system flexibility; C_s is a system flexibility coefficient that accounts for the elastic flexibility of the existing structure (i.e. soil-structure interaction, beam-column connection, beam flexibility, footing-column connection, etc.), where typically $0.68 \le C_s \le 1.0$. A flexibility coefficient of $C_s = 1.0$ indicates a rigid support (i.e. $\Delta_{es} = \Delta_{ps} = 0$), the lower bound flexibility coefficient $C_s = 0.68$ corresponds to $(\mu_\Delta)_{ex} = 2.0$, $\bar{M}_{ex} = 1.05$, and $c_e = \Delta_{es}/\Delta_{yf} = 0.50$.

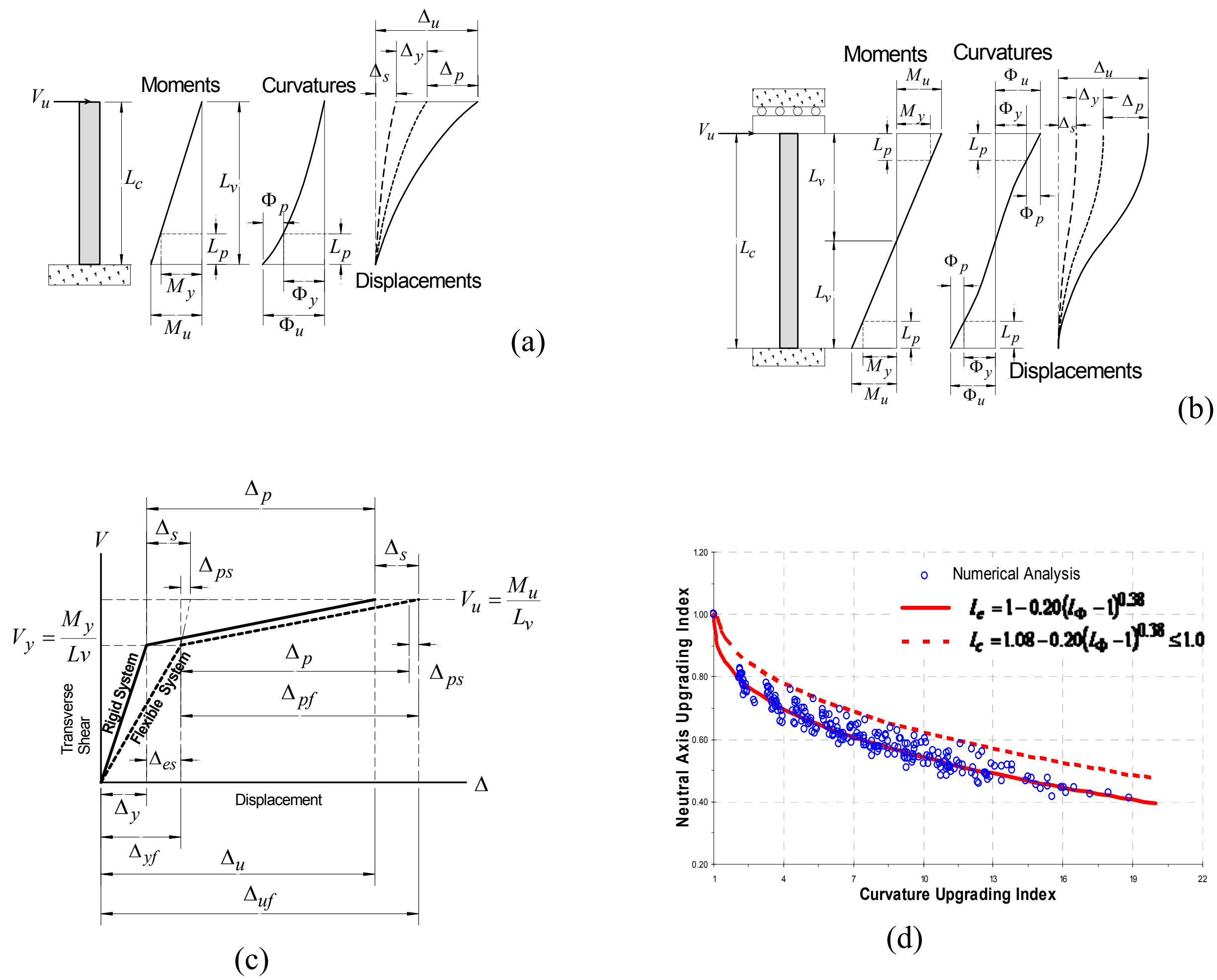


Figure 1. Moment, curvature and displacement of a column: (a) single curvature, (b) double curvature, (c) effect of elastic flexibility, (d) neutral axis upgrading index.

Upgrading of an existing column with inadequate ductile capacity $(\mu_{\Delta f})_{ex}$ may be required to achieve a desired level of performance during a seismic event. By selecting a given target ultimate displacement ductility $(\mu_{\Delta})_{up}$, the designer can establish a target displacement upgrading index $I_{\Delta f}$ of the system defined as:

$$I_{\Delta f} = (\mu_{\Delta f})_{up} / (\mu_{\Delta f})_{ex} \tag{5}$$

The curvature ductility of the existing column $(\mu_{\Phi})_{ex}$ can be found by solving for the displacement ductility $(\mu_{\Delta})_{ex}$ using $(\mu_{\Delta f})_{ex}$ of Eq. (3), and substituting it into Eq. (1); the curvature ductility of the upgraded column $(\mu_{\Phi})_{up}$ can be found by substituting the following displacement ductility $(\mu_{\Delta})_{up}$ into Eq. (1):

$$(\mu_\Delta)_{up} = 1 + I_{\Delta f}\left[(\mu_\Delta)_{ex} - 1\right] + (I_{\Delta f} - 1)/C_s \tag{6}$$

This indicates that the target curvature ductility $(\mu_\Phi)_{up}$ can be obtained in terms of the selected target $I_{\Delta f}$ of Eq. (5) and the displacement ductility of the as-built column $(\mu_\Delta)_{ex}$ of Eq. (1). Upon selecting a target displacement upgrading index $I_{\Delta f}$, using Eqs. (1)-(6) yields the following curvature upgrading index I_Φ:

$$I_\Phi = (\mu_\Phi)_{up} / (\mu_\Phi)_{ex} \tag{7}$$

Using the assumption of a plane section remaining plane, and considering that at yield $(\Phi_y)_{up} \cong (\Phi_y)_{ex}$, gives $I_\Phi = (\Phi_u)_{up} / (\Phi_u)_{ex}$, with $(\Phi_u)_m = (\varepsilon_{cu} / c_u)_m$. Also, $(\varepsilon_{cu})_{ex}$ and $(c_u)_{ex}$ are the ultimate compressive strain and neutral axis depth of the existing column, respectively, determined from sectional analysis of the existing column section; $(\varepsilon_{cu})_{up}$ and $(c_u)_{up}$ are the target ultimate compressive strain and neutral axis depth of the FRP-upgraded column, respectively, which are unknown properties of the FRP-upgraded column. The unknown target ultimate compressive strain $(\varepsilon_{cu})_{up}$ in the FRP-upgraded column can be conservatively found using:

$$(\varepsilon_{cu})_{up} = I_c I_\Phi (\varepsilon_{cu})_{ex} \;\; ; \;\; I_c = (c_u)_{up} / (c_u)_{ex} \approx 1.08 - 0.20(I_\Phi - 1)^{0.38} \leq 1.0 \tag{8}$$

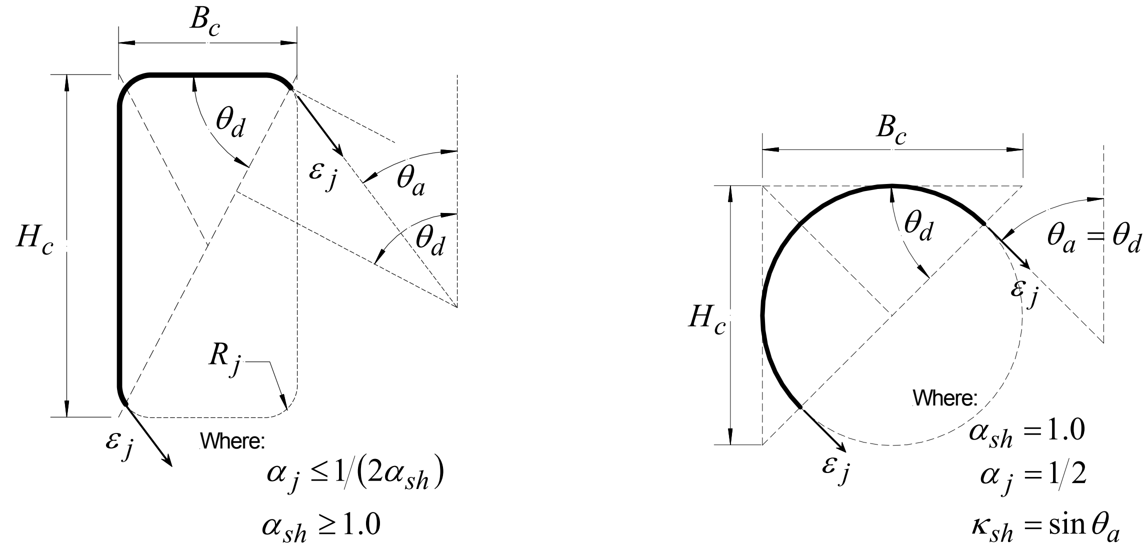
where I_c is the neutral axis upgrading index, an unknown geometric parameter of the upgraded FRP-confined concrete section; here it is determined from a parametric study (Moran 2009) of existing and FRP-upgraded circular, square and rectangular reinforced concrete sections. The above relationship represents a mean plus three standard deviations prediction of I_c as shown in **Fig. 1(d)** as a dashed line; the solid line plotted in this figure, is the best fit curve determined from regression analysis.

Upon establishing the target compressive strain $(\varepsilon_{cu})_{up}$ of Eq. (8), an FRP jacket material having an effective transverse modulus E_j and an ultimate tensile coupon failure strain ε_{fu} is selected by the designer. The actual rupture strain of the confining FRP jacket $(\varepsilon_{ju})_{up}$ can occur at much lower strains than ε_{fu} due to stress-concentrations at the jacket-to-concrete interface from axial strain-induced damage (internal cracking, aggregate sliding or crushing, void compaction or nucleation) of the confined concrete and at the rounded corners of rectangular FRP jacketed sections (Lam and Teng 2003; Eid et al. 2009). As a result, the following design jacket strain of $(\varepsilon_{ju})_{up}$ is recommended herein:

$$\left(\varepsilon_{ju}\right)_{up} = \kappa_{sh}\varepsilon_{fu} \;\; ; \;\; \kappa_{sh} = \sqrt{2}\tan\theta_a\left[1-2\alpha_j\left(1-\sin\theta_a\right)\right] \Big/ \left[4\left(1-2\alpha_j\alpha_{sh}\right)\sin\theta_a\right] \quad (8)$$

where $\kappa_{sh} = \left(\varepsilon_{ju}\right)_{up} / \varepsilon_{fu}$ is a shape dependent strain reduction coefficient which determines the influence that the FRP jacket shape has on the premature failure of the FRP jacket of circular and rectangular jackets with rounded corners, as shown in **Fig. 2.** Also, $\alpha_{sh} = H_c / B_c = \tan\theta_d$ and $\alpha_j = R_j / H_c$ are the section and jacket corner aspect ratio, respectively. The diagonal jacket strain angle θ_a, shown in **Fig. 2,** is given by $\theta_a = \theta_d - \sin^{-1}\left(\sin\theta_d - \cos\theta_d\right)$. A target plastic dilation rate $\left(\mu_{jp}\right)_{up} = \left(\varepsilon_{ju}\right)_{up} / \left(\varepsilon_{cu}\right)_{up}$ μ_{jp} is determined using $\left(\varepsilon_{cu}\right)_{up}$ of Eq. (8) and $\left(\varepsilon_{ju}\right)_{up}$ of Eq. (9). Using this target dilation rate, a target effective stiffness $\left(K_{je}\right)_{up}$ of the FRP jacket is determined using:

$$\left(K_{je}\right)_{up} = 35\left[\sqrt{\left(\sqrt{2}-\nu_{ci}\right) \Big/ \left[\left(\mu_{jp}\right)_{jp} - \nu_{ci}\right]} - 1\right] \geq 25.0 \quad (9)$$



(a) (b)

Figure 2. Typical geometry and definition of terms: (a) rectangular or square and (b) circular FRP jacketed sections.

where ν_{ci} is the initial Poisson's ratio of the concrete core where typically $0.15 \le \nu_{ci} < 0.25$; a value of $\nu_{ci} = 0.20$ is recommended. A minimum FRP jacket stiffness of $\left(K_{je}\right)_{up} = 25.0$ is recommended herein to provide adequate restraint against unrestrained expansion of the confined concrete, premature FRP jacket rupture due to bulging of the confined concrete, and premature buckling of the vertical steel reinforcement in the reinforced concrete column. Using $\left(K_{je}\right)_{up}$ of Eq. (10), the minimum thickness of the upgrading FRP jacket $\left(t_j\right)_{up}$ can be determined using:

$$\left(t_j\right)_{up} = \left(K_{je}\right)_{up} H_c f_{co} / \left(k_e C_{sh} E_j\right) \tag{10}$$

where f_{co} is the unconfined peak compressive strength of the concrete core ; k_e is the confining efficiency of the FRP jacket, where $k_e = 1.0$ for circular jackets; for rectangular and square jackets:

$$k_e = 1 - 0.667\left[\left(1 - 2\alpha_{sh}\alpha_j\right)^2\right] / \left[1 - \alpha_{sh}\left(4 - \pi\right)\left(\alpha_j\right)^2\right] \tag{11}$$

where C_{sh} is the FRP jacket shape-dependent reinforcement ratio coefficient, where $C_{sh} = 2.0$ for circular jackets; for rectangular or square jackets:

$$C_{sh} = \left[\left(1 + \alpha_{sh}\right) - \left(4 - \pi\right)\alpha_j \alpha_{sh}\right] / \left[1 - \alpha_{sh}\left(4 - \pi\right)\left(\alpha_j\right)^2\right] \tag{12}$$

Eq. (11) indicates that in order to obtain a certain level of performance in the reinforced concrete column, the minimum required thickness $\left(t_j\right)_{up}$ of the FRP jacket depends on the shape and the geometry of the FRP-confined section, the FRP jacket shape-dependant reinforcement ratio coefficient, the major dimension of the FRP-confined section, and the mechanical properties of the concrete core and FRP jacket.

FRP jacket Design Examples

Circular FRP-upgraded Concrete Column Design Example

Consider the as built and FRP upgraded reinforced circular cantilevered concrete columns tested by Seible et al. (1997), with experimental and analytical properties summarized in **Table 1**. Section analysis of the as built column indicates that the yield curvature is $\Phi_y = 7.08x10^{-6} rad/mm$ and $\bar{M}_{ex} \approx 1.0$. Estimating a system flexibility coefficient $C_s = 0.90$ and $C_\phi = 1.0$ for a column in single curvature bending, determining λ_p of Eq. (2) yields $\lambda_p = 0.142$ or $L_p = 520$ mm. Using the section properties and material properties listed in **Table 1** and the design procedure outlined in the flow chart of **Fig. 3**.

Table 1. Column specifications and details of cantilevered reinforced concrete columns confined by FRP jackets in the potential plastic hinge region performed by Seible et al. (1997)

		Circular Section	Rectangular Section
Section Properties	Column Height (L_c)	3.658 m	3.658 m
	Column Shear Span (L_v)	3.658 m	3.658 m
	Column Depth (H_c)	610 mm	730 mm
	Column width (B_c)	610 mm	489 mm
	Concrete strength (f_{co})	34.45 MPa	34.45 MPa
	Longitudinal reinforcing diameter (d_{bl})	25 mm and 22 mm	19 mm
	Axial Load (P_u)	1,780 KN	1,780 KN
	Corner radius (R_j)	305 mm	25 mm
FRP jacket properties	Jacket Modulus (E_j)	124 GPa	124 GPa
	Ultimate Strain (ε_{fu})	0.010 mm/mm	0.010 mm/mm
	FRP jacket Thickness (t_j)	5.1 mm	10.2 mm

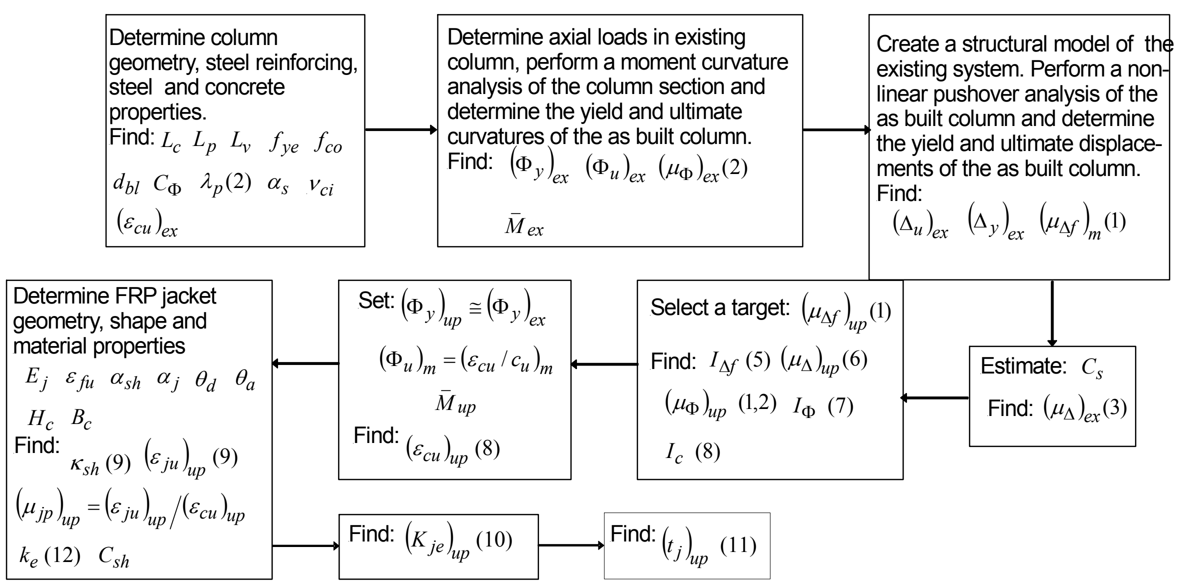


Figure 3. Flow chart of proposed strain based design procedure.

The pushover analysis and experimental tests results indicate that the as built column experiences a displacement ductility of approximately $(\mu_{\Delta f})_{ex} = (\Delta_{uf}/\Delta_{yf})_{ex} \approx 2.2$ or from Eq. (3) $(\mu_\Delta)_{ex} = (\Delta_u/\Delta_y)_{ex} \approx 2.33$, $(\mu_\Phi)_{ex} = 4.37$. From moment curvature analysis of the section, $(\Phi_u)_{ex} = 3.09x10^{-5}$ rad/mm; $(C_u)_{ex} = 208.2$ mm and $(\varepsilon_{cu})_{ex} = 0.00633$ mm/mm. Using the coupon failure strain of **Table 1** and $\kappa_{sh} = \sqrt{2}/2$ from Eq. (9) yields $(\varepsilon_{ju})_{up} = 0.00707$ mm/mm, $(\mu_{jp})_{up} = 0.339$, and $(K_{je})_{up} = 68.44$ is found. Selecting a target displacement ductility of $(\mu_{\Delta f})_{up} = 8.0$, $I_{\Delta f} \approx 3.64$, and $(\mu_{\Delta})_{up} = 8.30$. Using $\bar{M}_{up} = 1.25$ in Eq. (1), $(\mu_{\Phi})_{up}$ yields $(\mu_{\Phi})_{up} = 18.85$. Using $I_{\Phi} = 4.31$ yields $I_c = 0.765 \le 1.0$ and $(\varepsilon_{cu})_{up} = 0.0209$ mm/mm. Using $Ej = 124$ GPa of **Table 1** and $k_e = 1.0$ yields the minimum FRP jacket thickness $(t_j)_{up}$ of Eq. (11) of $(t_j)_{up} = 5.80$ mm. This thickness is approximately 13.7 % larger than the 5.1 mm jacket used in cantilevered column test performed by Seible et al. (1997), which performed to a displacement ductility of approximately $\mu_{\Delta f} \approx 10.0$ that is greater than the expected value of $(\mu_{\Delta})_{up} = 8.30$.

Rectangular FRP-upgraded Concrete Column Design Example

Consider the as built and FRP upgraded reinforced rectangular cantilevered concrete column tested by Seible et al. (1997), with the experimental and analytical properties summarized in **Table 1**. Section analysis of the as built column indicates that the yield curvature is $\Phi_y = 5.01x10^{-6}$ rad/mm and $\bar{M}_{ex} \approx 1.0$. Estimating $C_s = 0.90$ and $C_{\phi} = 1.0$ as in the previous example, and determining λ_p of Eq. (2) yields $\lambda_p = 0.149$ or $L_p = 545$ mm. The pushover analysis and experimental test results indicate that the as built column experiences a displacement ductility of approximately $(\mu_{\Delta f})_{ex} = (\Delta_{uf} / \Delta_{yf})_{ex} \approx 3.0$ or from Eq. (3) $(\mu_{\Delta})_{ex} = (\Delta_u / \Delta_y)_{ex} \approx 3.22$ and solving for $(\mu_{\Phi})_{ex}$ in Eq. (1) yields $(\mu_{\Phi})_{ex} = 6.37$. Moment curvature analysis of the section indicates that $(\Phi_u)_{ex} = 3.09x10^{-5}$ rad/mm; with $(C_u)_{ex} = 171.6$ mm and $(\varepsilon_{cu})_{ex} = 0.00633$ mm/mm. Selecting a target displacement ductility of $(\mu_{\Delta f})_{up} = 8.0$ yields $I_{\Delta f} \approx 2.67$ and $(\mu_{\Delta})_{up} = 8.19$. Using $\bar{M}_{up} = 1.25$ and solving for $(\mu_{\Phi})_{up}$ yields $(\mu_{\Phi})_{up} = 19.17$. This results in $I_{\Phi} = 3.08$, $I_c = 0.816 \le 1.0$, and $(\varepsilon_{cu})_{up} = 0.0137$ mm/mm. Using the coupon failure strain of **Table 1** and $\kappa_{sh} = 0.476$ yields $(\varepsilon_{ju})_{up} = 0.00477$ mm/mm, $(\mu_{jp})_{up} = 0.348$, and $(K_{je})_{up} = 65.25$. Using $Ej = 124$ GPa of **Table 1,** $k_e = 0.464$ of Eq. (12), $k_e = 1.0$ yields the minimum FRP jacket thickness $(t_j)_{up}$ of Eq. (11) of $(t_j)_{up} = 11.63$ mm. This thickness is approximately 14.0 % larger than the 10.2 mm jacket used in cantilever column test performed by Seible et al. (1997), which performed to a displacement ductility of approximately $\mu_{\Delta f} \approx 8.0$, which is close to the expected value of

$(\mu_{\Delta})_{up} = 8.19$.

Conclusions

In the analytical design procedure presented herein, no consideration is given to the unknown increase in axial compressive strength in the FRP confined concrete, since this increase in strength even though it is beneficial, is a secondary effect that results from the axial strain induced dilation of the FRP-confined concrete core and resultant transverse confining stresses provided by the elastic FRP jacket as transverse dilation progresses. The proposed design approach is in contrast with those provided in both the strain energy based design procedure, and the multivariate regression analysis upgrading index based design procedure. The method presented here is based on the strain ductility increase that results from the constant kinematic restraint provided by the confining elastic FRP jacket, and is thus a strain-based approach using performance-based design principles.

As was demonstrated herein, the information required to determine the minimum FRP jacket thickness within the plastic hinge region of a reinforced concrete section is: (1) the geometry of the concrete section or new FRP jacket; (2) the unconfined concrete core strength, (3) the material properties of the selected FRP jacket; and (4) the ultimate design FRP jacket strain determined based on the FRP material selected by the design engineer and the geometry of the jacket. The design procedure compares favorably with experimental results for columns in single curvature from the literature that were upgraded with FRP jackets and had demonstrated a substantial displacement ductility increase.

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