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CONCRETE FILLED CFRP CYLINDRICAL SHELLS FOR BRIDGE COLUMNS IN SEISMIC ZONES

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ABSTRACT

A promising approach to effectively use of fiber-reinforced-polymer composites in new construction is as structurally-integrated formwork in hybrid concrete/FRP structural components. This paper summarizes research developments on a concept that combines filament wound Carbon Fiber Reinforced Polymer (CFRP) cylindrical shells/tubes filled on-site with concrete for composite columns. The CFRP shell provides the functions of formwork and external reinforcement, while the concrete core supplies the main compression force mechanism and allows for the anchorage of connection elements. Ductile and elastic (strength) designs were developed for applications in seismic zones. Analytical and design models were developed and the performance for both column design approaches was experimentally validated through large-scale lateral load tests.

Introduction

The use of advanced composite materials, or Fiber Reinforced Polymers (FRP), in civil infrastructure has become increasingly important as they offer unique mechanical and chemical characteristics in terms of strength, stiffness, and durability. These advantages have already become apparent in their application for the rehabilitation (retrofit and repair) of existing buildings and bridges. However, the full potential of FRPs to replace traditional structural materials, such as steel and concrete, in new design and construction remains to be realized. This is primarily due to economic reasons related to the high materials and manufacturing costs and simplistic design approaches which replace conventional structural components with FRP components on a member-by-member basis and which utilize design criteria developed for conventional structural materials and not specifically for FRPs. The advantages of FRPs can only be effectively used when structural designs and construction concepts combine these “new” materials with conventional structural materials such as concrete and steel to form new composite structural members that are designed based on the mechanical characteristics of the new composite member.

Recent research has shown that a promising way to technically and economically use FRP composites in new construction is through structurally-integrated formwork in hybrid concrete/FRP structural components. Once such concept is the use of thin prefabricated filament-wound carbon/epoxy (CFRP) cylindrical shells filled on-site with concrete. The CFRP shell serves the dual function of reinforcement and stay-in-place formwork for the concrete core. The concrete provides compression force transfer,

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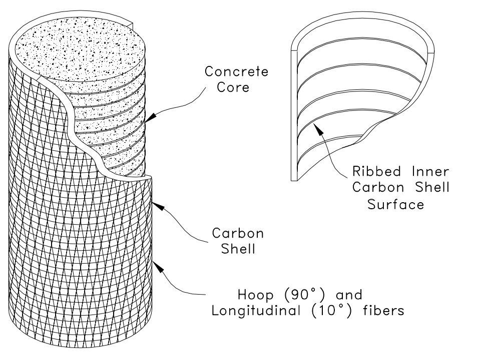
stabilizes the thin shell against buckling, and allows the anchorage of connection elements. The design of the CFRP cylindrical shells, or tubes, provides carbon fibers in the longitudinal and transverse tube directions. Transverse ribs are provided on the inside of the shell for full force transfer between the concrete infill and the shell. The concrete filled CFRP shell system thus combines the fiber dominated tensile characteristics of FRPs, with the dominant characteristics of conventional materials, such as compression in concrete and inelastic deformation capacity in steel (in connection concepts). The concept is schematically depicted in Fig. 1, together with a picture of a filament wound carbon/epoxy shell for experimental validation.

While the concrete filled Carbon Shell System is rather simple and similar to conventional Concrete Filled Steel Tube (CFST) systems, the system offers the advantages of (1) light weight, (2) tailorable mechanical properties through fiber orientation, and (3) increased durability due to the high chemical inertness of the carbon fibers. These advantages, however, can only be fully utilized when appropriate connection systems and concepts can be developed and proven. The presented work highlights research efforts on the development of hybrid CFRP/concrete columns for seismic zones, including concept development, analytical and design models, and large-scale experimental validation.



a) Schematic of CFRP/concrete Composite System

b) Filament Wound CFRP Shell for Testing

Figure 1. Concrete filled CFRP shell system.

Design Concepts for CFRP/Concrete Columns in Seismic Zones

The development of the CFRP/concrete composite system for columns in seismic zones demanded that the axial and lateral response of the new system be comparable to or better than conventionally reinforced concrete columns. Due to the innovative nature of the system the research approach was to bound the expected spectrum of structural response by developing design concepts that addressed Design concepts for seismic applications required addressing both elastic and inelastic design principles. Since the CFRP shells are inherently non-ductile research for column applications focused on bounding the spectrum of structural response through ductile and elastic design concepts (Seible, et al. 1995).

The ductile design concept relies on the inelastic deformation capacity provided by steel anchorage bars that extend from the connecting reinforced concrete element (footing, pier cap, etc.) into the column core for the development of a conventional ductile anchorage detail. A force transfer mechanism takes place from the carbon/epoxy shell to the anchorage bars in a lap-splice fashion. The presence of steel starter bars, the confinement from the CFRP shell, and the provision of a gap between the column and the connecting reinforced concrete element ensure the development of a conventional flexural plastic hinge.

The strength design concept continues the concrete filled CFRP shell into the connecting reinforced concrete element (footing, pier cap, etc.) without the use of steel bars. The anchorage mechanism consists of the reaction moment generated inside the reinforced concrete element due to the compression force couple and friction between the carbon/epoxy shell outer surface and the concrete in the concrete element. Because of the linear elastic nature of the laminated CFRP composite, and the lack of inelastic deformation capacity, the overall system response is essentially linear-elastic up to failure.

Member Behavior and Analysis Models

CFRP Shell Behavior

The laminated FRP cylindrical shell was assumed to be thin compared to the radius of the section and behave linear elastically. It was also assumed that the circular geometry of the shell constrains the coupling that may exist between in-plane and out-of-plane behavior. The FRP shell was thus modeled as a linear elastic orthotropic plate with equivalent in-plane properties. Assuming a biaxial stress state in the shell the longitudinal and transverse strains (ε_L and ε_T) are defined by Eq. 1, where E_L and E_T are the equivalent engineering modulus in the longitudinal and transverse directions and ν_{LH} and ν_{HL} are the associated Poisson's ratios.

$$\begin{Bmatrix} \varepsilon_L \\ \varepsilon_T \end{Bmatrix} = \begin{bmatrix} 1/E_L & -\nu_{TL}/E_T \\ -\nu_{LH}/E_L & 1/E_T \end{bmatrix} \begin{Bmatrix} \sigma_L \\ \sigma_T \end{Bmatrix} \tag{1}$$

Confined Concrete Behavior

The increased compressive strength and ductility of confined concrete is well recognized and many models have been proposed to capture this behavior. When steel (hoops or tube) confines the concrete, the restraining effect will reach a limit upon yielding of the steel. Passive models confinement models account for this behavior by assuming a constant radial confining pressure (Mander et al. 1998). Due to the elastic response of FRPs, the confinement pressure from the FRP shell never reaches a maximum value but rather it develops in direct proportion to the dilation of the concrete core. Although passive models are unrealistic for elastic confining systems, their use in an incremental approach based on the current concrete radial expansion has been found to be satisfactory (Mirmiran et al. 1997).

The FRP confined concrete behavior was thus modeled through the use of an incremental elastic analysis approach in combination with Mander's (Mander et al. 1988) passive concrete confinement model (Burgueño, 1999). The concrete core elastic response was defined the generalized Hooke's law for an isotropic cylinder assuming that radial and tangential stresses are equal. The constitutive relations are as given in Eq. 2 where, E_c, and ν_c refer to the concrete's elastic modulus and Poisson's ratio, respectively.

$$\begin{Bmatrix} \varepsilon_x \\ \varepsilon_r \end{Bmatrix} = \begin{bmatrix} 1/E_c & -\nu_c/E_c \\ -\nu_c/E_c & (1-\nu_c)/E_c \end{bmatrix} \begin{Bmatrix} \sigma_x \\ \sigma_r \end{Bmatrix} \tag{2}$$

Evaluation of the concrete core dilation is based on the definition of a varying equivalent Poisson's ratio of the concrete under axial compression. The equivalent Poisson's ratio is a measure of the dilation of the concrete core caused by internal microcraking, which deviates from the initial elastic material property. The equivalent Poisson's ratio for concrete was assumed to vary with the axial strain as described by Eq. 3 (Elwy and Murray 1979), where ε_x is the axial strain, ε_{co} is the critical concrete strain at maximum strength, and ν_o is the initial concrete Poisson ration. An upper limit of ν_c = 0.5 is set (Elwy and Murray 1979) to avoid unreasonable values for $\varepsilon_x/\varepsilon_{co}$ rations greater than 0.7. The longitudinal tension behavior of the concrete infill is modeled assuming it is linear elastic, with initial modulus and Poisson's ratio up to the tension splitting stress.

$$\nu_c = \nu_o \left[1.0 + 1.3763 \frac{\varepsilon_x}{\varepsilon_{co}} - 5.36 \left(\frac{\varepsilon_x}{\varepsilon_{co}} \right)^2 + 8.586 \left(\frac{\varepsilon_x}{\varepsilon_{co}} \right)^3 \right]. \tag{3}$$

Under compression stresses the concrete core will dilate, initially elastically and subsequently due to microcracking. The FRP shell will restrain this dilation only if the radial expansion of the CFRP shell itself is less than the dilation of the concrete core. The FRP shell hoop modulus and thickness, together with the relative Poisson's ratio of the shell and the concrete, determine the amount of confining pressure that will be present. Thus, for axially stiff FRP tubes it becomes important to include the effects of the FRP Poisson's ratio in the calculation of the stress-strain response of the concrete core.

CFRP/Concrete Axial/Flexural System Behavior

Modeling of the concrete-shell interaction was made by assuming full composite action between the concrete core and the CFRP shell. With this assumption, the longitudinal strains and the hoop and radial strains in the shell and the concrete cylinder are equal. Additionally, the tangential, or hoop, stresses in the thin walled shell are related to the internal expansion or the concrete core similar to the case of a thin pressure vessel with average radius *R*. The expressions for these conditions together with Eqs. 1 to 3 were combined to determine the radial strain and radial stress in the concrete core (Davol 1998, Davol et al. 2001):

$$\varepsilon_r = \varepsilon_x \left[\frac{\nu_c E_c R(1-\nu_{TL}\nu_{LT}) + \nu_{LT} E_T t(1-\nu_c - 2\nu_c^2)}{-E_T t(1-\nu_c - 2\nu_c^2) - E_c R(1-\nu_{TL}\nu_{LT})} \right] \tag{4}$$

$$\sigma_r = -\frac{(\nu_{LT}\varepsilon_x + \varepsilon_r)E_T t}{(1-\nu_{TL}\nu_{LT})R} \tag{5}$$

Assuming that the transverse shell stress can be taken as the radial stress in a pressure vessel with pressure ε_r and Eq. 1, the transverse and longitudinal stresses in the FRP shell defined by Eqs. 6 and 7. Eq. 4 can then be used incrementally to establish to solve for both ε_r and ε_x.

$$\sigma_T = -\sigma_r \frac{R}{t} \tag{6}$$

$$\sigma_L = E_L \left(\varepsilon_L - \sigma_r \frac{\nu_{TL}}{E_T} \frac{R}{t} \right). \tag{7}$$

Transverse shell strains as a function of tensile axial strains is modeled by assuming the concrete core as an elastic plug resisting the contraction of the shell in tension due to its Poisson's ratio (Davol et al. 2001):

$$\varepsilon_T = \varepsilon_L \left[\frac{E_T t \nu_{LT}}{\nu_{LT}\nu_{TL} E_{co} R - E_T t - E_{co} R \nu_{LT}} \right]. \tag{8}$$

CFRP/Concrete System Shear Response

For determining shear response a modeling approach and numerical procedure capable of evaluating the shear load-deformation response of concrete filled FRP composite cylindrical shells with composite and non-composite interaction has been recently proposed (Burgueño and Bhide 2006). The formulation extends the previously presented models to include anisotropic FRP behavior, a smeared shear modulus for cracked concrete, and conditions of equilibrium, compatibility and shear force transfer across cracks in analogy to the modified compression field theory. Interested readers are referred to the noted publication.

Member Design Models

Flexure Design

A simplified evaluation of the flexural response of circular concrete filled carbon shells can be done using equivalent engineering properties for the CFRP shell and a simplified equivalent stress block for the concrete core (Burgueño 1999). The strain compatibility equations can be carried out for a prescribed strain limit state (tension or compression) on the CFRP shell. The ultimate compressive or tensile strain can be determined from a separate failure analysis of the laminated plate or from tests.

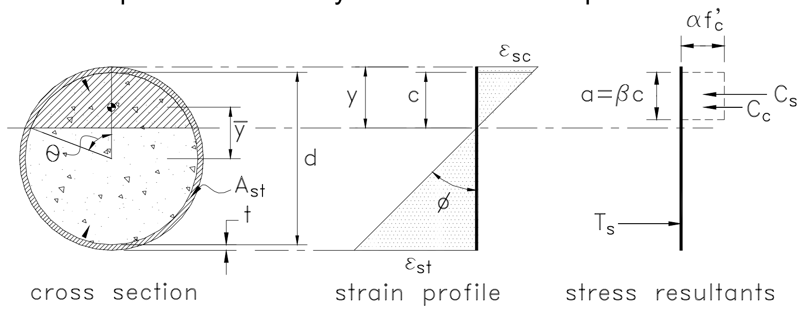


Figure 2. Equivalent section for flexural analysis of concrete filled CFRP shells.

With reference to Fig. 2, the stress resultants can be approximated by the expressions in Eqs. 9 to 11, where $\alpha = 0.85 A_{seg}$ = the area of the concrete core in compression, E_s = the CFRP shell longitudinal elastic modulus, and A_{sc} and A_{st} = the areas of the CFRP shell in compression and tension, respectively. The extreme compressive (ε_{sc}) and tensile (ε_{st}) strains on the CFRP shell are defined by the linear strain distribution and either a tensile or compressive failure limit state.

$$C_c = \alpha f_c^{'} A_{seg} \tag{9}$$

$$C_s = (5/12)\varepsilon_{sc} E_s A_{sc} \tag{10}$$

$$T_s = (5/12)\varepsilon_{st} E_s A_{st} \tag{11}$$

The area of the concrete in compression is evaluated by considering the circular segment of reduced depth by the stress block parameter β. The segmental areas for the concrete core and CFRP shell can be determined from geometry formulas for a circle segment with reference to Fig. 2. The neutral axis depth y is obtained by force equilibrium of the stress resultants and any applied axial load. Considering moment equilibrium about the neutral axis, the section moment capacity can be approximated by:

$$M_n = C_c(y - t - 0.5\beta c_c) + 0.8 C_s y + 0.8 T_s (D - y)\,. \tag{12}$$

The above expressions have been evaluated against the presented refined models. It should be noted however, that the equations, like the models, were developed for carbon/epoxy FRP shells with large longitudinal stiffness. Their accuracy for FRP shells with low longitudinal stiffness or made from other FRP materials (e.g., E-glass) has not been verified.

Shear Design

Design and assessment of shear strength for the concrete/CFRP composite member (Burgueño 1999) can be based on the UCSD additive component shear strength model (Priestley et al. 1996). Thus, shear capacity is defined by three components (Eq. 13), where V_c = a concrete component dependent on the level of member ductility, V_p = an axial load component dependent on the column aspect ratio, V_f = an FRP shell component dependent on the elastic shear stresses carried by the CFRP shell.

$$V_n = V_c + V_p + V_f \tag{13}$$

The concrete and axial load components are in strict agreement with the model presented by Priestley et al. (1996) and are defined by Eqs. 14 and 15, where A_e = 0.8A_{gross}, and k, within plastic regions, depends on the member displacement ductility μ_Δ, reducing from 0.29 (MPa units) for $\mu_\Delta \leq 2$ to 0.05 (MPa units) for $\mu_\Delta \geq 8$; P = the axial compressive load, and α is the angle formed between the column axis and the strut from the point of load application to the center of the flexural compression zone at the column plastic hinge critical section.

$$V_c = k\sqrt{f_c^{'}}A_e \tag{14}$$

$$V_p = P\tan\alpha \tag{15}$$

An estimate of the maximum shear stress for the cracked concrete filled shell can be obtained from mechanics of materials, assuming isotropic material behavior for both the concrete core and CFRP shell (Burgueño 1999). The shear stresses caused by shear force V on the CFRP shell can be obtained by considering a transformed section, where the inner concrete core is reduced by the ratio of the concrete modulus to the longitudinal modulus in the carbon shell, n_{cg}. Shear capacity of the CFRP will be reached when the maximum shear stress equals the in-plane ultimate shear strength of the laminated shell, τ_{su}. This value can be obtained from a failure analysis of the laminated shell based on ply strengths. The maximum shear strength contribution from the CFRP shell can then be estimated with Eq. 16, where A_s = the total cross-sectional area of the laminated FRP shell, and A_c = the total cross sectional area of the concrete core.

$$V_f = \tau_{su} \Big/ \left[(2/A_s) + (8/3n_{cg}A_c)\right] \tag{16}$$

Connection Design Models

Ductile Connections

Design for ductile connections was governed by the provision of adequate confinement to the concrete core to ensure proper force transfer between the reinforcement bars and the CFRP shell. An expression for the required confinement based on an approach for the confinement of lap-splices (Priestley et al. 1996) was followed (Seible et al. 1995), where a dilation strain of 0.001 was adopted as the limit to avoid bond failure. Thus, for a circular column of diameter D, and starter bar length l_s, the minimum effective required confining pressure, f_l, that needs to be provided by the CFRP shell can be expressed as:

$$f_l \geq \frac{1.5\ A_b f_y}{1.4\ \pi D\, l_s}. \tag{17}$$

Assuming a passive shell of total thickness t and equivalent transverse modulus E_T, the minimum effective required thickness within the anchorage region is defined by:

$$t \geq 500\, D\, f_l\ /\ E_T. \tag{18}$$

In addition to the confinement requirements for anchorage of the lapped bars, consideration must be given to ensure that enough confinement capacity exists within the plastic hinge region that will be developed within the ductile connection detail. For this purpose, an appropriate volumetric confinement ratio must be provided (Priestley et al. 1996). Following recommendations for the retrofit of reinforced concrete columns with FRP jackets, the required transverse thickness to supply volumetric confinement within the plastic hinge region of a ductile connection is given by Eq. 19, where ε_{cu} = the ultimate compressive strain in the concrete, f'_{cc} = the maximum confined concrete strength, f_{uj} = the ultimate transverse allowable stress in the FRP shell, and ε_{uj} = the ultimate allowable transverse strain in the FRP shell.

$$t = \frac{0.1(\varepsilon_{cu} - 0.004)Df'_{cc}}{f_{uj}\varepsilon_{uj}} \tag{19}$$

As in the case of shear design requirements, the flexural and shear reinforcements were considered to contribute to the confinement requirements. The magnitude of their contribution was based on the expressions presented above, taking care of identifying the appropriate thickness, elastic modulus, and ultimate stress and strain values associated with the direction of the dilation strain (i.e., transverse).

Strength Connections

Strength, or elastic, connections can be achieved by continuing the CFRP/concrete member into a footing block, or other prismatic reinforced concrete (RC) element. The anchorage mechanism is provided by the reaction moment from the compression force couple generated inside the RC element and the friction between the outer skin of the CFRP shell and the surrounding concrete. This connection concept is thus similar to the connection of precast columns to foundation blocks. The length of embedment of the CFRP/concrete member was guided by the need to withstand the prying shear failure in the footing element. An embedment of approximately 1.5*D*, where *D* is the outer diameter of the CFRP element, was found to be adequate a minimum realistic value. Detailed attention must be given to minimize stress concentration effects at the column-footing interface of the FRP member and the reinforced concrete element. The detail must alleviate normal and transverse shear concentration effects, localized Poisson effects, and allow localized rotations with the purpose of minimizing localized effects and achieving larger member displacements (Seible et al. 1995).

Experimental Evaluation

Design Considerations

The CSS columns were experimentally validated for strength and inelastic deformation capacity through two large-scale lateral load tests (Seible et al. 1995). The CSS test units were designed so as to match the performance of a “reference” conventionally reinforced concrete column. To match moment capacity of the reference column at the critical section, the anchorage detail for the Ductile Design Concept featured the same reinforcement detail for anchorage. The inside surface of the CFRP shell along the transition region was provided with ribs to aid in the force transfer mechanism to the starter bars. A gap was allowed between the bottom of the CFRP shell and the top of the footing to avoid crushing of the shell at the compression toe under large deformations and allow concentration of the flexural inelastic actions on the anchorage reinforcement.

For the Strength Design Concept, the carbon shell jacket was continued into the footing without the use of steel bars for anchorage. Since the overall system response was expected to be essentially linear-elastic, an equal energy approximation was used for this design concept. The target design moment was thus found by equating the area under the inelastic force-deformation curve for the reference column with the area under the elastic force-deformation curve for the CSS column with equal initial stiffness. A design limit state of f_u with a strength reduction factor of 0.85 (or a reduction of three times the standard deviation) due to material properties variation was adopted.

Test Unit Details

The test units consisted of a 40% scale circular cantilever bridge columns with an effective height of 3.66 m, and a concrete core diameter of 610 mm. The reference column (Fig. 3a) contained 20 #7 G60 bars of continuous reinforcement with 25 mm cover. Transverse reinforcement was provided by #3 G60 spirals with a pitch of 57 mm. The anchorage detail for the Ductile Design column (Fig. 3b) also featured 20 #7

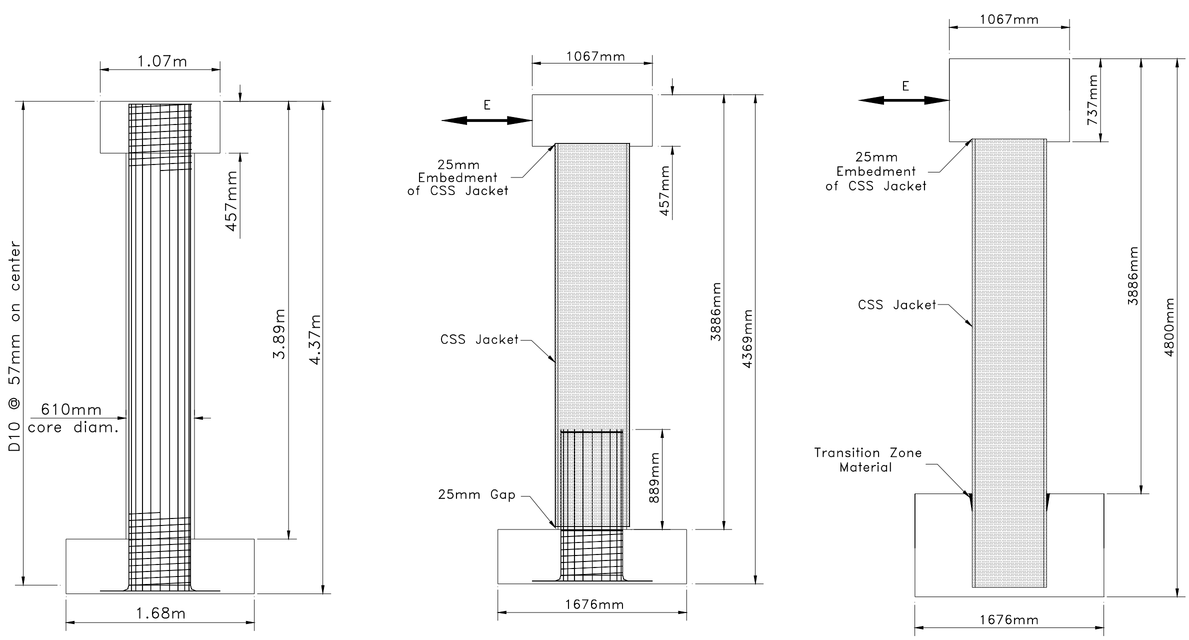
G60 steel bars and the starter bar length was assumed to be forty times the bar diameter. The CFRP shell thickness in the anchorage region (914 mm long) was 10 mm, while the thickness in the main region was 5 mm. The bottom gap for localization of inelastic flexural actions was 25 mm.

For CFRP shell for the Strength Design column (Fig. 3c) had a constant thickness of 12 mm along the member length. To minimize stress concentrations at the column-footing interface a compliant transition zone provided around the CFRP shell. The compliant zone was distributed vertically along the top 152 mm of the footing connection and taped down linearly from a 13 mm thickness at the top. The selected material for this interface region was a structural adhesive with a compressive elastic modulus of approximately one-half of concrete. All test units used normal weight concrete with a compressive strength of 34.5 MPa.
The CFRP shells were filament wound with Hercules AS4D-GP (12k) carbon fibers and an HBRF-55A epoxy resin system. The layup for the main region of the Ductile Design column was [90/±10/90/±10$_2$/90/±10/90] with an additional [90$_{20}$] in the anchorage region. The layup for the Strength Design column was [90/±10$_2$/90/±10$_2$/90/±10$_2$/ 90/±10$_2$/90/±10$_2$/90]. The thickness of the 90° plies was 0.254 mm and the thickness of the ±10° plies was 1.016 mm (for each pair). The ply properties were E_{11}=130.3 GPa, E_{22}=8.205 GPa, G_{12}=4.344 GPa, ν_{12}=0.445. The resulting equivalent engineering properties are shown in Table 1.

Table 1. Equivalent engineering properties of CFRP shells.

Design Concept	**E_L (GPa)**	**E_T (GPa)**	**G_{LT} (GPa)**	**ν_{LT}**
Ductile Design [a]	100	32.92	7.05	0.19
Ductile Design [b]	54.6	82.2	5.69	0.061
Strength Design	100	24.2	7.52	0.23

[a] General Region, [b] Anchorage Hinge Region



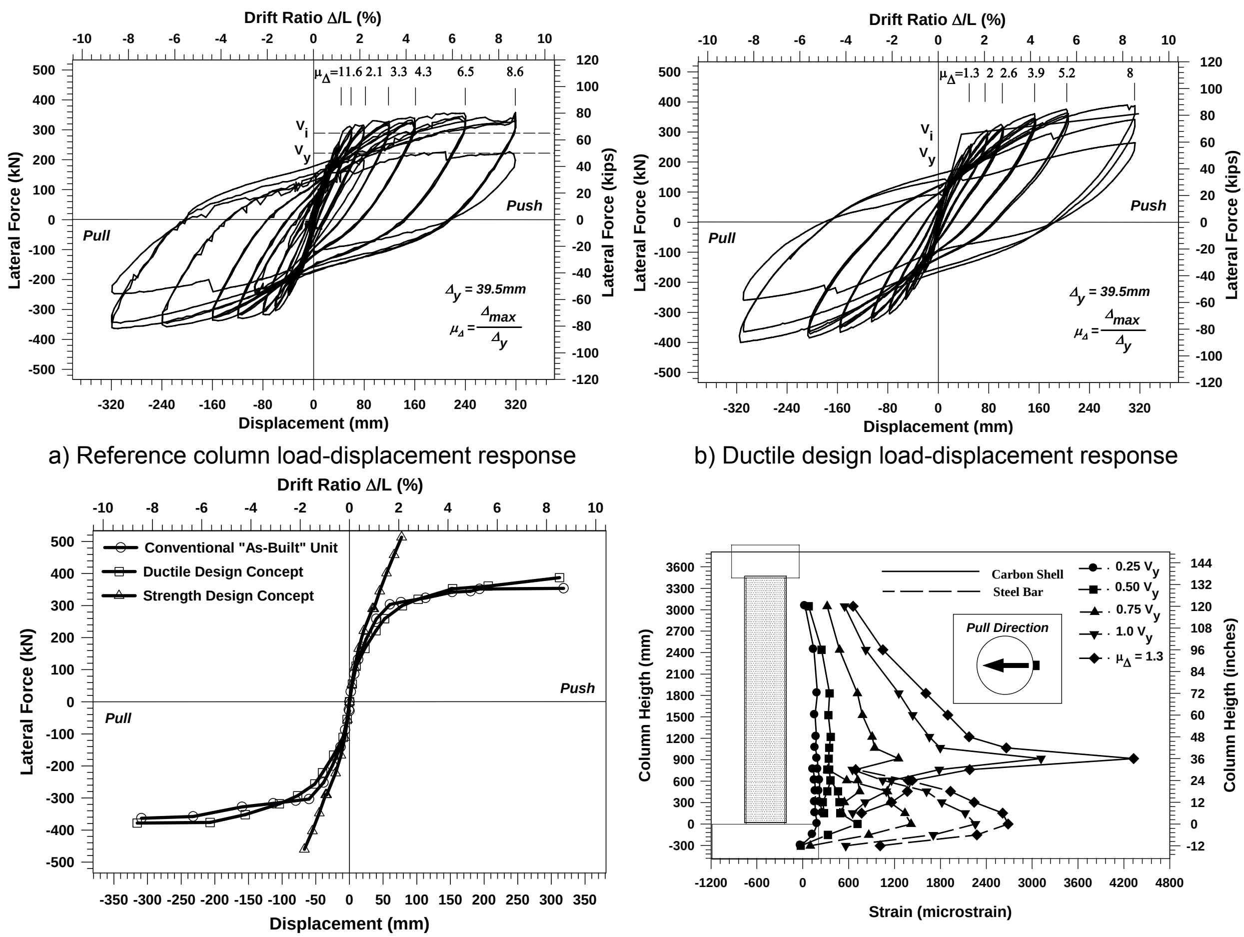
a) Reference RC column b) Ductile Design CSS column c) Strength Design CSS column

Figure 3. Test unit geometry and details.

Observed Response and Test Results

The columns were loaded cyclically according to a standard, incrementally in-creasing, fully-reversed cyclic pattern, with a constant axial load of 1779 kN, which represents a practical upper bound for single column bridge piers. Representative results of the column tests (Seible et al., 1995) are shown in Fig. 4. The ductile design concept (Fig. 4b) displayed a stable ductile behavior matching almost exactly the response of the reference column (see Fig. 4a and 4c). The CFRP shell fully confined the column concrete in the plastic hinge zone, allowing a displacement ductility of μ_Δ = 8 to be reached prior to starter bar rupture in low cycle fatigue. An effective force transfer between the CFRP shell and the anchorage bars was observed as seen in the longitudinal strain profiles of Fig. 4d. The response of the strength design concept was essentially linear elastic (Fig. 4c). The system failed in a brittle form due to in-plane compressive forces at the toe of the column and highly localized transverse shear stresses.

An overview of the experimental validation of the presented analytical tools is shown in Fig. 5. The flexural characterization of CFRP/concrete elements was achieved by studies on bridge girder systems. Fig. 5a shows the extreme fiber longitudinal strains on the CFRP shell girder in a four-point bending test. Validation of the analytical tools for sections under flexure and a constant axial load was done through the strength design column test. A comparison of the computed moment-curvature trace and the measured response is shown in Fig. 5b.



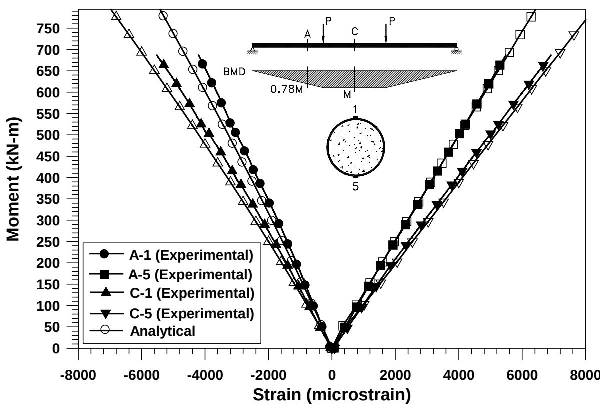
a) Reference column load-displacement response

b) Ductile design load-displacement response

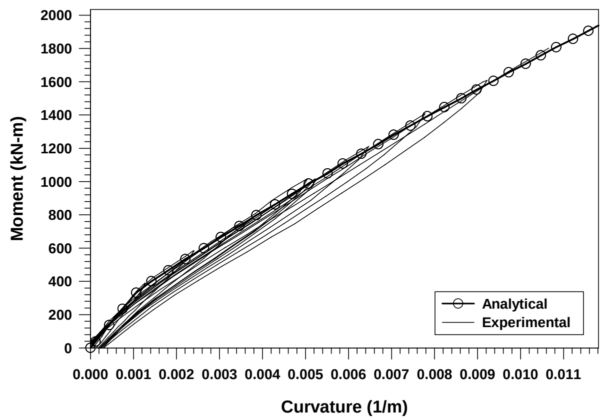
c) Force-displacement response envelopes

d) Ductile design tension strain profiles

Figure 4. Representative experimental results from cantilever cyclic tests.



a) Flexural behavior of CFRP/concrete girders



b) Moment-curvature response of strength design

Figure 5. Experimental validation of analytical tools for CFRP/concrete members.

Conclusions

CFRP/concrete systems represent an innovative approach to composite column design by taking advantage of the unique mechanical properties of FRPs for directional strength along the fibers with the dominant characteristics of concrete in compression and steel in inelastic deformation. Research on column applications has led to the development of two design concepts that address both elastic and inelastic response. An inelastic, or ductile, design concept was shown successful in developing a stable hysteretic ductile response duplicating a well-built conventional reinforced concrete column. The desired response of an elastic, or strength, design concept was limited by the failure of the carbon/epoxy shells due to its low interlaminar shear strength. Thus, the adequate response of strength design concepts rely on careful detailing of the critical section by providing additional hoop reinforcement and a more compliant transition region at the connection level.

Analytical models developed and experimentally validated in a parallel research project for composite girder applications, can adequately capture the axial and flexural behavior CFRP/concrete composite columns. Simple member design models based on standard approaches were proposed and verified against refined models. Connection design models based on retrofit measures have been proposed and experimentally validated. The presented research developments show that CFRP/concrete composite columns are feasible and that together with basic analysis tools and design details these systems can provide a viable alternative to conventional steel or concrete columns designs in seismic zones.

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