A SIMPLIFIED MODEL FOR FRP-CONFINED HOLLOW CIRCULAR CONCRETE COLUMNS UNDER AXIAL COMPRESSION

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ABSTRACT

Column jacketing with Fiber Reinforced Polymer (FRP) composite materials has been extensively studied in recent years but it is uncertain how FRP jackets may perform retrofitting hollow concrete columns since the topic has been scantily researched. Hollow reinforced concrete (RC) bridge piers are used in tall bridges to maximize the structural efficiency of the strength-mass and stiffness-mass ratios.

A unified theory for confinement of circular hollow sections is proposed herein, that can be extended to the case of solid sections. The main aim of the model is to trace step-by-step the evolution of the three dimensional stresses in confined concrete and confining devices (i.e. FRP externally bonded jackets). The model is able to estimate confinement effectiveness and to plot stress-strain relationships, which are different in the case of solid and hollow sections. In the case of a hollow core section, it is imperative to introduce the relative confinement stiffness. Through the proposed model, direct, closed form solutions have been derived to determine ultimate confined concrete properties and stress-strain curves. At present, theoretical results based on the proposed concrete circular hollow sections confinement model, in satisfactory agreement with the experimental data available in scientific literature, show that FRP jacketing can enhance the ultimate load and ductility significantly, also in the case of hollow concrete cross sections.

KEYWORDS

confinement, FRP, hollow Section, RC columns, strengthening.

INTRODUCTION

Many notable reinforced concrete (RC) bridges have been constructed throughout the world, particularly in Europe, the United States and Japan, incorporating hollow piers, where high seismic activity and natural boundaries require high elevation infrastructures. Hollow columns resist the high moment and shear demands by reducing the self weight and the high bearing demand on foundations, maximizing structural efficiency of the strength-mass and stiffness-mass ratios and reducing the mass contribution of the column to the seismic response. In particular, bridge piers designed in accordance with old design codes may suffer severe damage during seismic events, caused by low ductility. There is nowadays the need to assess the capacity of existing bridge structures and especially hollow columns.

Few studies to date have covered circular and rectangular hollow columns: a review can be found in Lignola et al. (2007a) regarding columns with applied low levels of axial load, investigating the performance of the cross sections subjected to combined shear and flexure stresses. Extensive research has shown that strengthening systems, like confinement jackets, can enhance the performance of compressed columns subjected to cyclic flexural forces or dynamic actions due to seismic events. The investigations on the behavior of hollow concrete columns wrapped with Fiber Reinforced Polymer (FRP) can be used to analyze, for example, the moment-curvature (ductility) relationships and the effect of confinement on seismic response.

Recently, Modarelli et al. (2005) investigated the stress-strain experimental behavior of hollow FRP-confined concrete columns under compressive loads. The results of this research gave useful information regarding the behavior of hollow concrete columns confined with FRP under monotonic compression, with respect to concrete strength, aspect ratio, shape of the cross section and type of FRP material.
A unified theory for confinement of circular hollow sections is proposed herein, that can be extended to the case of solid sections. The main aim of the model is to trace step-by-step the evolution of the three dimensional stresses in confined concrete and confining devices (i.e. FRP jackets externally bonded). The model is able to estimate confinement effectiveness and to plot stress-strain relationships, which are different in the case of solid and hollow sections.

BACKGROUND

An analytical model to predict the response of circular concrete members confined with FRP has been proposed by Fam and Rizkalla (2001b); the model recently was simplified in a closed form and into design charts for solid sections through linearization and error minimization of nonlinear expressions, also accounting for buckling of slender confined columns (Albanesi et al. 2007). The above model is based on equilibrium and radial displacement compatibility. Through the equations proposed by Mander et al. (1988) it adopts a step-by-step strain increment technique to trace the lateral dilation of concrete. The failure of the confined concrete member is due to the rupture of the FRP confinement which is controlled by a multiaxial criterion (i.e. Tsai-Wu failure criterion).

The passive confinement on axially loaded concrete members is due to the transverse dilation of concrete and the presence of a confining device which opposes this expansion and puts the concrete in a triaxial state of stress. Concrete cylinders under different confining levels (uniform and constant in the transverse direction) have been tested (Gardner 1969) and dilation ratios of concrete under different transverse confinement pressures have been provided.

A model based on the assumption that the increment of stress in the concrete due to confinement is achieved without any out-of-plane strain has also been proposed by Braga et al. (2006). In this model it is proposed to adopt plain strain conditions to simulate the confinement effect.

CONFINEMENT MODEL

Assuming axial symmetry, the radial displacement is the only displacement component and stress components \(\sigma_r\) and \(\sigma_\theta\) (where \(r\) calls for the radial component and \(\theta\) for the circumferential component) can be evaluated according to boundary conditions, i.e. applied external (at \(r = R_e\)) inward pressure \(q\).

In this case of external inward pressure (where \(R_i\) is the internal radius), the stress equations become:

\[
\sigma_r = \frac{q R_i^2}{R_e^2 - R_i^2} \left(1 - \frac{R_i^2}{r^2}\right) \quad (1a)
\]
\[
\sigma_\theta = \frac{q R_i^2}{R_i^2 - R_e^2} \left(1 + \frac{R_i^2}{r^2}\right) \quad (1b)
\]

In the case of an FRP jacket where the thickness \(t\ll R_e\), loaded by an outward pressure \(q\), the stress and displacement equations can be simplified as:

\[
\sigma_\theta \approx -\frac{q r_i}{t} \quad (2a)
\]
\[
s_r(r) \approx -\frac{q R_i^2}{E' t} (1-\nu) \quad (2b)
\]

A key aspect of the proposed model is that plain strain condition is considered to evaluate the radial displacement of the elements confined by FRP. In the case of hollow core sections, the different contributions of radial and circumferential stresses are explicitly considered through an equivalent average confining pressure (actually the confining stress field is not equal in the two perpendicular directions and the effect of confinement should be evaluated in each point of the section with the effective confining pressures different in two orthogonal directions, however this approach is currently neglected due to the massive computational efforts). Numerical tests showed that this assumption gives rather accurate results. For the sake of simplicity, an equivalent average confining pressure can be evaluated as:

\[
f'_i = \frac{\sigma_r + \sigma_\theta}{2} = \frac{q R_i^2}{R_e^2 - R_i^2} \quad (3)
\]

\(f'_i\) is a constant and uniform confining pressure inside the volume of the concrete tube (and is equal to \(q\) in the case of solid sections with \(R_i=0\)).

At this point, a simplified approach is taken in the present paper; this is derived from a more refined model discussed in Lignola et al. (2007b). The compatibility equation of the lateral strains \(\varepsilon_\theta\) along with the equilibrium of the confining device with the concrete cylinder (the inward pressure \(q\) on concrete cylinder is equal to the outward pressure \(q\) on the confining jacket) allows to associate at each axial strain \(\varepsilon_c\) the confining pressure \(q\) exerted on concrete by the FRP jacket.
\[
e_{\theta,\text{concrete}} = v_c \cdot e_c = \frac{q R_c}{E_f} \cdot (1 - v_f) = e_{\theta,\text{FRP}} \tag{4}
\]
and after mathematical manipulations it leads to the form \(f'_c = f'_c (e_c)\):
\[
f'_c = \left\{ \frac{R_c}{R_c^2 - R_f^2} \cdot \frac{E_f \cdot t}{(1 - v_f)} \right\} e_c = k_{\text{FRP}} \cdot e_o = k_{\text{FRP}} \cdot v_c \cdot e_c \tag{5}
\]
In the previous equation, the coefficient \(k_{\text{FRP}}\) can be also considered as a confinement stiffness ratio of FRP jacket, half of the FRP confinement ratio multiplied by the FRP Young Modulus (as in other models like Spoelstra and Monti 1999). Previous equations are based on linear elasticity theory for FRP Young Modulus and Poisson’s ratio, \(E_f\) and \(v_f\) respectively, while the Poisson’s ratio \(v_c\) accounts for the nonlinear behavior of concrete. Before peak strength the concrete dilation in the transverse direction is very low (due to an almost constant, elastic Poisson’s ratio). As the deformation increases at peak and after, confined concrete exhibits post-peak behavior characterized by the appearance of significant cracking and it shows an increase in the Poisson’s ratio.

To account for this nonlinear behavior, a secant approach can be considered where the Poisson’s ratio is set as a function of the axial strain and of the confining pressure. The secant Poisson’s ratio is used to obtain the lateral stress at a given axial strain in the incremental approach to evaluate, at the same axial strain level, the lateral stress. Before peak strength the concrete dilation in the transverse direction is very low (due to an almost constant, elastic Poisson’s ratio). As the deformation increases at peak and after, confined concrete exhibits post-peak behavior characterized by the appearance of significant cracking and it shows an increase in the Poisson’s ratio.

At any given axial strain \(\varepsilon_c\), the Poisson’s ratio is reduced by the confinement; therefore, the Poisson’s ratio at a given axial strain level is lower in the presence of the actual confining pressure. Gardner (1969) tested concrete cylinders under different confining pressures (uniform in the transverse direction) thus providing dilation ratios of concrete under different transverse confinement pressures (triaxial tests). Fitting the results curve with a second-order polynomial a simplified linear relationship for \(v_c\) under constant confining pressure is provided, and from regression analysis (Fam and Rizkalla 2001b):
\[
\frac{v_c}{v_{cc}} = 1 + \frac{\varepsilon_c}{\varepsilon_{cc}} \left( 0.719 + 1.914 \cdot \frac{f_c'}{f_{cc}} - 1 \right) \tag{6}
\]
where \(v_c\) is the actual Poisson’s ratio at a given axial strain \(\varepsilon_c\) and actual confining pressure \(f'_c\). The actual peak compressive stress and strain (evaluated for the actual confining pressure \(f'_c\)) are \(f_{cc}\) and \(\varepsilon_{cc}\). The initial values are the unconfined peak concrete strength \(f_{cc}^{'}\), and the Poisson’s ratio \(v_{cc}\) usually ranging between 0.1 and 0.3.

At any given axial strain \(\varepsilon_c\), the Poisson’s ratio is reduced with the increase of \(f'_c\), because the ratio \(\varepsilon_c / \varepsilon_{cc}\) decreases at growing of \(\varepsilon_c\). This expression (obtained by linear regression on a definite range of confining pressures) is not suitable for concrete subjected to very high confinement pressures at low axial strains and for concrete subjected to very low confinement pressures at high axial strains.

The stress-strain model of confined concrete proposed by Mander et al. (1988) has been adopted:
\[
f_c = \frac{f_{cc} - x \cdot r}{r - 1 + x} \tag{7a}
\]
\[
x = \varepsilon_c / \varepsilon_{cc} \tag{7b}
\]
\[
r = \frac{E_c}{E_c - f_{cc}/\varepsilon_{cc}} \tag{7c}
\]
where \(f_{cc}\) and \(\varepsilon_{cc}\) are the confined concrete strength, and corresponding strain respectively; in the proposed model, based on the “five parameter” multiaxial failure surface and calibrated with data from triaxial tests, it is:
\[
f_{cc} = f_{cc}^{'} \left\{ -1.254 + 2.254 \sqrt{1 + 7.94 \frac{f_{cc}}{f_{cc}^{'} - 2 \frac{f_{cc}}{f_{cc}^{'} - 4}} \right\} \tag{8a}
\]
\[
\varepsilon_{cc} = \varepsilon_{cc}^{'} \left( 5 \frac{f_{cc}^{'}}{f_{cc}^{'} - 4} \right) \tag{8b}
\]
In previous equations, the initial tangent stiffness can be evaluated as \(E_{cc} = 5700 \cdot \sqrt[3]{f_{cc}^{'}}\) and \(\varepsilon_{cc}\) is the compressive strain at the unconfined peak concrete strength \(f_{cc}^{'}\), usually setting \(\varepsilon_{cc} = 0.002\).

Usually the Poisson’s ratio of a confining device made by uniaxial FRP fibers applied by wet lay-up technique can be neglected in this procedure. But in the case of laminates with multiaxial fibers or FRP tubes, the Poisson’s ratio should be considered because generally it is greater even than the initial concrete Poisson’s ratio.

**Incremental Approach**

The iterative procedure is described in figure 1. Imposing an axial strain \(\varepsilon_c\), a first trial value of the Poisson’s ratio is determined (i.e. the initial value at the beginning of the procedure or the previously evaluated value at iteration i-1) and an equivalent average confining pressure \(f'_i\) is evaluated according to Eq. 5.
At this point, the peak compressive stress and strain, \( f_{cc} \) and \( \varepsilon_{cc} \), can be evaluated according to Eqs. 8a-b and in turn an output Poisson’s ratio is determined through Eq. 6. This is the new value of the Poisson’s ratio to repeat the procedure that converges when the output value of Poisson’s ratio is close enough to the Poisson’s ratio in input. Once the correct Poisson’s ratio is known at actual axial strain, a Mander curve at given confining pressure \( f' \) can be drawn and the “true” stress point \( f_c \) can be determined corresponding to the actual strain \( \varepsilon_c \). At each level of load/deformation (namely \( \varepsilon_c \)), the complete stress and strain regime in both the concrete cylinder and confining device is known, i.e. the circumferential stress in the confining device is given by Eqs. 1a-b.

The procedure is repeated up to a value of axial strain that induces failure of the confining device, i.e. accounting for the triaxial state of stress in the jacket or, in the case of thin concrete walls, failure of the concrete close to the inner wall can also be checked.

**Figure 1. Flow Chart of the proposed iterative procedure to evaluate \( f_c = f_c(\varepsilon_c) \)**

**DIRECT EVALUATION OF CONFINED CONCRETE ULTIMATE PROPERTIES**

In previous sections, it has been shown an iterative procedure to model the confinement of hollow and solid circular columns. In some cases it can be appropriate to evaluate directly the ultimate properties of confined concrete without performing the entire iterative procedure.

An useful expression of the confinement stiffness ratio of FRP, \( k_{FRP} \), has been defined in Eq. 5; many other confinement models - i.e. Spoelstra and Monti (1999) model – take into account the transverse volumetric confinement ratio, \( \rho_{FRP} \), as a key parameter to evaluate the effect of FRP confinement on confined concrete constitutive law. This volumetric ratio, \( \rho_{FRP} \), is computed as twice the confinement stiffness ratio \( k_{FRP} \), divided by the Young Modulus of FRP, leading to:

\[
\rho_{FRP} = \frac{2 \cdot k_{FRP}}{E_f} = \frac{R_s}{R_s^2 - R_i^2} \cdot \frac{2 \cdot t}{(1-\nu_f)}
\]
Considering the case of a solid section ($R_i=0$) and the case of a confining device made by uniaxial FRP fibers applied by wet lay-up technique ($\nu_f=0$, negligible), the volumetric ratio simplifies to the well-known expression $\rho_{FRP} = 2t/R$.

**Closed form solution**

Following the proposed model, the ultimate properties of confined concrete can be easily evaluated also looking at Eqs. 5,8a-b: parameters $f'_{l}, f_{cc}$ and $\varepsilon_{cc}$ are expressed as a function of the lateral strain $\varepsilon_\theta$ if the mechanical properties of unconfined concrete and FRP, and the geometrical dimensions of the section are known.

Given a value of $\varepsilon_\theta$ smaller than the ultimate strain of FRP jacket (to avoid FRP failure), the three parameters, $f'_{l}(\varepsilon_\theta), f_{cc}(\varepsilon_\theta)$ and $\varepsilon_{cc}(\varepsilon_\theta)$ are known (where the dependence on $\varepsilon_\theta$ has been clearly delineated).

The axial strain is related to the lateral strain through the Poisson’s ratio of concrete, $\nu_c = \varepsilon_\theta / \varepsilon_c$. To evaluate the axial strain of concrete $\varepsilon_c$ it is useful to solve Eq. 6. After some mathematical manipulations, the following expression is obtained:

\[
\frac{1}{\varepsilon_c} \left( 0.719 + 1.914 \frac{f'_{l}}{f'_{co}} \right) \varepsilon_c^2 + 1 \cdot \varepsilon_c - \frac{\varepsilon_\theta}{\varepsilon_c} = c_2 \cdot \varepsilon_c + c_1 \cdot \varepsilon_c + c_0 = 0
\]  

(10)

which is a parabolic equation in $\varepsilon_c$. The three coefficients are given by:

\[
c_2 = \frac{1}{\varepsilon_c} \left( 0.719 + 1.914 \frac{f'_{l}}{f'_{co}} \right) > 0
\]

(11a)

\[
c_1 = 1 > 0
\]

(11b)

\[
c_0 = -\frac{\varepsilon_\theta}{\varepsilon_c} < 0
\]

(11c)

Since $c_1^2 - 4c_2c_0$ is positive and $c_2, c_1, c_0$ has the observed signs, the only root with a physical meaning - $\varepsilon_c (\varepsilon_\theta)$ positive - is:

\[
\varepsilon_c (\varepsilon_\theta) = -\frac{-1 + \sqrt{c_1^2 - 4c_2c_0}}{2c_2}
\]

(12)

Once the axial strain $\varepsilon_c$ function is known, it is possible to evaluate the corresponding stress $f_c$ function through Eqs. 7. The lateral strain $\varepsilon_\theta$ is the parameter of these functions.

Confined concrete exhibits softening behavior (usually when the confining pressure $f'_{l}$ is small compared to the unconfined concrete strength $f'_{co}$) if the function $f_c(\varepsilon_\theta)$ has a maximum for an $\varepsilon_\theta$ smaller than the ultimate FRP strain $\varepsilon_{FRP,u}$; that is

\[
f_c(\varepsilon_{FRP,u}) = 0
\]

(13)

otherwise the ultimate concrete axial stress, $f_c(\varepsilon_{FRP,u})$, is equal to the confined concrete strength, because confined concrete exhibits a hardening behavior up to failure.

The ultimate state (and at this stage, the parameters pertaining to the ultimate Mander’s curve) can be defined once the maximum confinement stress, $f'_{l}(\varepsilon_{FRP,u})$ (and corresponding ultimate FRP strain, $\varepsilon_{FRP,u}$) is given.

Assuming $\varepsilon_\theta$ equal to the ultimate strain of FRP jacket at failure, to evaluate the ultimate axial strain of concrete $\varepsilon_{cc,u}$, Eq. 12 is solved. The corresponding stress is given by Eq. 7 and, if the derivative of the function $f_c(\varepsilon_\theta)=0$ has no root in the range $(0, \varepsilon_{FRP,u})$ then the evaluated stress is also the maximum value of the function, that is the confined concrete strength.

**THEORETICAL-EXPERIMENTAL COMPARISON**

Experimental tests on FRP-confined concrete specimens with circular cross sections, available in the scientific literature, have been simulated according to the proposed confinement model. Specimens are wrapped with either carbon or glass fiber FRP composites. Some experimental campaigns include both solid and hollow cross-section specimens; these tests are considered to validate the proposed unified model for both types of section. Some other test campaigns include only solid sections (made by either plain concrete or reinforced concrete) and have been considered by other authors as benchmarks for their solid concrete section confinement models (Fam and Rizkalla 2001b, Spoelstra and Monti 1999); these experimental data were also considered to validate the proposed model. Each campaign is briefly described below; it is underlined that, for all presented comparisons,
the Young Modulus of unconfined concrete has been derived from experimental curves, other materials data are provided in the following reports.

**Tests by Modarelli et al. (2005)**

A set of 124 tests including 85 specimens wrapped with FRP and 39 plain concrete specimens was conducted. Two different kinds of concrete mixes were made to investigate the effect of the concrete: the average 28-day compressive strength was 28 MPa and 38 MPa respectively.

The specimens were 150 mm in diameter and 300 mm in height, and had different internal diameters (CC1 and CC5: 0 mm, which is solid section; CC2 and CC3: 50 mm).

The concrete specimens were wrapped with unidirectional CFRP composites made by one layer with 0.165 mm thickness. The CC3 hollow specimen was wrapped with two layers (0.330 mm thickness) of CFRP. All the unidirectional CFRP composite jackets had fibers aligned at 90° to the principal axis of the specimen. Hoop strength was 3068 MPa and the Young Modulus was 221 GPa.

The response of these specimens was predicted and compared to the experimental outcomes in figures 2a-2b. Satisfactory agreement was found between the measured values and the predicted response. Poisson’s ratio ranging between 0.07 and 0.36 and experimental peak concrete compressive strain, unconfined, ranging between 0.38% and 0.63% were considered.

**Tests by Fam and Rizkalla (2001a)**

A set of three experiments on cylindrical hollow and solid plain concrete specimens confined with glass fiber tubes with different wall thicknesses were carried out. The specimens were 219 mm in external diameter and had different internal diameters (Stub 1: 0 mm that is solid section; Stub 2: 95 mm; Stub 3: 133 mm) with cylindrical compressive strength of 58 MPa.

The specimens were wrapped with a 33.4 GPa, Young Modulus glass fiber-reinforced polymer (CFRP) shell (2.21 mm thickness) with stacking sequence of nine layers [-88/-88/+4/-88/-88/+4/-88/+4/-88] and tensile strength of 548 MPa. The stress versus axial strain response was predicted and compared to the measured values in figure 3a. Satisfactory agreement was found again between the measured values and the predicted response for the confined concrete compressive strength. A Poisson’s ratio was estimated of between 0.10 and 0.25 and a peak concrete compressive strain of 0.2%.

**Tests by Kawashima et al. (1997)**

Two 200 x 600 mm reinforced concrete specimens with 39 MPa concrete and wrapped with carbon fiber-reinforced polymer and tested under axial compression were simulated. These solid cylindrical specimens were provided with a longitudinal steel reinforcement ratio of 1%, with a yield stress of 295 MPa, whose contribution was subtracted from the experimentally measured stresses (Spoelstra and Monti 1999). The specimens, both of them with solid section, were wrapped with two different high-modulus carbon fiber-reinforced polymer sheets, with jacket thicknesses of 0.338 mm (H3) and 0.676 mm (H4) with tensile strengths of 2810 and 2327 MPa, respectively.

The Young Modulus of the jacket was 439 GPa. The stress versus axial strain response was predicted and compared to the measured values in figure 3b. The Poisson’s ratio was estimated of between 0.12 and 0.15 and a peak concrete compressive strain of 0.34%.
Tests were carried out on 15 confined 152.5 x 305 mm concrete cylinders using different configurations of glass fiber filament-wound plies. The average 28-day compressive unconfined concrete strength was 36.3 MPa.

The analyzed specimen with a solid circular section was wrapped with four plies of E-glass fibers. The glass fiber-reinforced polymer jacket, with tube thicknesses of 1.2 mm, had hoop strength and Young Modulus, reported by the manufacturer, of 583 MPa and 52 GPa, respectively. The hoop fibers had a 90-degree orientation (confinement). The stress versus axial strain response was predicted and compared to the experimentally measured values in figure 4. Good agreement was found between the measured values and the predicted response for the shape of the curve and the second slope, Poisson’s ratio was estimated at 0.13 and peak concrete compressive strain at 0.5%.
CONCLUSIONS

The present work falls within broader research activity to advance knowledge and develop a cost- and time-effective design method for fast FRP strengthening of hollow bridge columns so that bridge functions can be quickly restored. Such strengthening seeks to upgrade seismic capacity in terms of strength and ductility.

Present simplified model is derived from a more refined confinement model proposed by the same authors (Lignola et al., 2007b). The confinement model is able to predict the fundamentals of the behavior of hollow members confined with FRP both in terms of strength and ductility, giving a clear picture of the effectiveness of confinement on the response of this kind of elements. This model can be extended to the case of a solid section and, in this sense, is a unified model. In the case of a hollow core section, it is imperative to introduce the relative confinement stiffness $E_f t/R_e (R_e^2 - R_i^2)$. If the stiffness is evaluated in the traditional manner, $E_f t/R_e$ for the same external radius and confining device thickness $t$, increments of hole size result in increases of the mechanical confinement ratio, thus resulting in increases in global confinement performance. The larger the hole, the higher is the deformability of the element thus resulting, for a similar level of dilation, in different stress paths: in the case of a solid section the dilation of concrete is restrained by the FRP wraps and this interaction causes a strength enhancement, while in the case of thin walls, the greater deformability does not allow any strength improvement to be gained, even though significant ductility enhancement is achieved. In this case the state of stress becomes mostly circumferential.

The model is able to trace the evolution of stresses and strains in the confinement wraps and concrete, allowing the multiaxial state of stress and the potential failure of the external reinforcement to be evaluated at each load step. Once the complete 3D stress strain step-by-step evolution is known, any multiaxial plasticity and failure criteria can be applied to concrete material and FRP jackets, and the model can be further improved in this direction. Through the developed model, direct, closed form solution has been derived to determine ultimate stress-strain confined concrete properties and curves. At present, theoretical results based on the above models and simplified assumptions, in satisfactory agreement with the available experimental data, show that FRP jacketing can enhance ultimate load and ductility significantly, also in the case of hollow concrete cross sections.

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