# Stress–strain Model for FRP Confined Heat-damaged Concrete Columns

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## Abstract:

This paper is dedicated to the development of a new analysis-oriented model to simulate the axial and dilation behavior of FRP confined heat-damaged concrete columns under axial compressive loading. The model's calibration has considered the experimental results from concrete circular/square cross-section specimens submitted to a certain level of heat-induced damage, which after attained the environmental temperature, were fully confined with FRP jacket and tested. New equations were developed to determine the mechanical characteristics of unconfined heat-damaged concrete by performing regression analysis on a large database of experimental tests. Based on a parametric study on dilation behavior of FRP confined heat-damaged columns, a new dilation model was developed to predict concrete lateral strain at a given axial strain, dependent on the thermal damage level. By using this dilation model, a new methodology was introduced for predicting the axial stress-strain response of FRP confined heat-damaged columns in compliance with the active confinement approach. The adequate predictive performance of the model is demonstrated by estimating experimental axial stress-strain results.

**Keywords**: FRP confined heat-damaged concrete; thermal damage; confinement model; dilation behavior

## 1- Introduction

During fire, concrete buildings generally demonstrate a better fire performance compared to timber and steel buildings due to concrete non-combustibility and relatively low thermal conductivity (Kodur [1], Bamonte and Lo Monte [2]). Nonetheless, depending on the fire intensity imposed to the structural elements, material deteriorations occur during fire exposure, resulting detrimental effects on the performance of concrete structures at their serviceability and ultimate limit state conditions (Demir *et al.* [3]). Considering the pre-existing thermal-induced damage, a post-fire strengthening solution for restoring its structural performance can be an environmental and economic sustainable solution over the demolishing and rebuilding alternative. To reinstate sufficiently axial responses of heat-damaged concrete columns (HC), externally bonded fiber-reinforced-polymer (FRP) composites have been demonstrated as a viable solution (Bisby *et al.* [4]).

Several experimental and analytical studies (i.e. Barros and Ferreira [5], Wang and Wu [6], Janwaen *et al.* [7], Shayanfar *et al.* [8]) were conducted to evaluate the capability of FRP confining strategy in upgrading the axial and dilation behavior of FRP confined concrete columns under axial compression at ambient conditions. For the case of FRP fully confined concrete columns of circular cross section at ambient conditions (FFCC-A, as shown in Fig. 1a), Barros and Ferreira [5] experimentally evidenced that the confinement-induced enhancements for normal-strength concrete is more pronounced than those registered in high-strength concrete. For the case of FRP fully confined concrete columns of square cross section at ambient conditions (FFSC-A, as shown in Fig. 1a), Wang and Wu [6] and Shan *et al.* [9] conducted experimental studies to evaluate the influence of cross-section circularity in the effectiveness of the confining strategy. It was evidenced that decreasing the corner radius ($r$) from $r = b/2$ (circular columns where $b$ defines the length of the cross-section dimension) to $r$

= 0 (square columns with sharp edges) results in a significant reduction in terms of confinement efficiency.

On the other hand, very limited experimental studies have been conducted for assessing the strengthening efficiency of FRP confining systems applied in the post-cooling regime of concrete specimens subjected to a certain maximum exposure temperature according to the heating scheme demonstrated in Fig. 1b. Bisby *et al.* [4] performed an experimental research to assess the effectiveness of FRP fully confinement on circular concrete columns subjected to different levels of maximum temperature (300, 500 and 700 °C) (FFCC-H, as shown in Fig. 1a). It was evidenced that FRP confinement could significantly increase the axial compressive strength and stiffness of unconfined heat-damaged specimens (Fig. 1c). The peak axial compressive strength and the corresponding axial strain of FFCC-H specimens subjected to severe thermal exposure (700 °C) were almost 90% and 145%, respectively, of those of FFCC at ambient conditions. However, the secant axial stiffness of FFCC-H specimens (as the ratio of axial stress to its corresponding axial strain) was reported to be considerably lower than that of FFCC, and this difference has increased with maximum exposure temperature ($T_m$). Lenwari *et al.* [10] experimentally evidenced that the axial stress versus axial strain response of FFCC-H is dependent on the used heating scheme, i.e. $T_m$, exposure duration and cooling regime (air or water cooling methods), and on the axial compressive strength of AC, which was also confirmed by Luo *et al.* [11]. Furthermore, it was shown that the level of axial strength improvements, induced by FRP confining system, is higher in HC than in AC. Accordingly, for HC with a high level of thermal damage, FRP effectiveness tends to be more significant. Ouyang *et al.* [12] investigated experimentally the dilation behavior of heat-damaged circular concrete specimens confined by Basalt FRP (BFRP) jacket (FFCC-H) under axial compressive loading. It was demonstrated that there is a noticeable difference between the transverse expansibility of HC subjected to different level of the imposed $T_m$. Furthermore, the recorded hoop strains at BFRP rupture were almost independent of the exposure temperature. Song *et al.* [13] experimentally evidenced the high potential of BFRP confining system for improving the axial and dilation responses of HC with square cross-section (FFSC-H, as shown in Fig. 1a). The confinement-induced enhancements were more pronounced in HC exposed to high temperatures and confined by thicker BFRP jackets.

For the prediction of the axial stress-strain response of FRP confined AC columns, several analysis-oriented models, based on active confinement approach, have been recommended i.e. Teng *et al.* [14], Lim and Ozbakkaloglu [15], and Shayanfar *et al.* [16 and 17]. Teng *et al.* [14] proposed an analysis-oriented model for passively confined concrete, whose calibration was based on experimental observations in FRP fully confined concrete specimens of circular cross section (FFCC). Based on theoretical principles and experimental evidences, Shayanfar *et al.* [16] developed a generalized analysis-oriented model for passive confinement arrangements, whose parameters were derived from the experimental results with FRP fully/partially confined circular concrete specimens. Shayanfar *et al.* [17] extended Shayanfar *et al.* [16]'s model to make it applicable to the case of columns of square-cross section, by simulating the influence of the non-circularity in terms of confinement-induced enhancements. Bisby *et al.* [4] proposed a design-oriented model to determine the axial response of heat-damaged circular cross-section concrete columns confined by FRP (FFCC-H). In this model, the confinement-induced improvements were expressed as a main function of maximum exposure temperature ($T_m$) imposed to the column. Nevertheless, an analysis-oriented model to simulate the full-range of dilation and axial responses of FRP fully confined HC columns with square cross-section is still lacking.

This paper aims to introduce a new methodology to determine the axial stress-strain response of FRP fully confined circular/square HC columns (the concrete specimens were submitted to a certain $T_m$, and after having attained the environmental temperature, the fully confining FRP jacket was applied to the heat-damaged concrete). For the case of unconfined heat-damaged concrete, through regression analysis performed on a large test database, new expressions are developed to determine its mechanical characteristics in terms of axial compressive strength and its corresponding axial strain. By performing parametric studies with the available experimental data, the significant influence of the pre-existing thermal-induced damage on the establishment of the axial and dilation behavior of FRP fully confined heat-damaged concrete columns is demonstrated. By performing a parametric study to assess the influence of $T_m$ on the concrete transverse expansion, a new dilation model depending on the heat-damaged level is proposed, which has a unified character with the dilation model developed by Shayanfar *et al.* [17] for concrete specimens at room temperature. By using the developed dilation model, a new methodology is introduced, based on active confinement approach, for the simulation of the axial stress-strain response of FRP fully confined circular/square HC columns at the different levels of $T_m$. The model's adequate predictive performance is demonstrated by estimating experimental axial stress-strain responses.

## 2- Unconfined heat-damaged concrete columns (HC)

Kodur [1] evidenced that post-fire response of HC columns significantly depends on concrete mechanical, thermal (including thermal conductivity, thermal diffusivity, specific heat, and mass loss) and deformation (including concrete thermal expansion) properties, as well as on the spalling response. In general, concrete submitted to maximum exposure temperature ($T_m$) up to 100 °C can be considered almost undamaged. Afterward, water loss (causing shrinkage) and the expansion of aggregates induce internal stresses in the concrete, particularly for $T_m \geq 300\ °C$. Furthermore, thermal-induced chemical processes and thermo-mechanical damages lead to a significant strength degradation for the concrete submitted to high levels of exposure temperature [1]. A comprehensive review of concrete properties at elevated temperatures can be found in Kodur and Sultan [18], Hertz [19], Raut and Kodur [20], Aslani and Bastami [21].

In terms of mechanical properties, by submitting concrete to elevated temperature with a heating scheme including maximum exposure temperature ($T_m$), the axial compressive stress of HC ($f_c^T$) and the modulus of elasticity ($E_{cT}$) decrease depending upon its peak axial strength ($f_{c0}^T$). However, axial strains corresponding to the peak ($\varepsilon_{c0}^T$) and ultimate stages ($\varepsilon_{cu0}^T$) increase, demonstrating a significant reduction on the axial stiffness (defined as $f_c^T/\varepsilon_c$ where $\varepsilon_c$ is the axial strain) of HC compared to AC (Hertz [19], Chang *et al.* [22], Al-Salloum *et al.* [23], Sharma *et al.* [24], Geng *et al.* [25], Xiao *et al.* [26], Xiang *et al.* [27]).

### 2-1- Peak axial strength of HC ($f_{c0}^T$)

Experimental studies evidenced that the heat-induced damages in HC columns lead to a reduction in terms of peak axial strength. Accordingly, by defining the axial strength ratio, $\beta_{0T}$, (the ratio of $f_{c0}^T$ and $f_{c0}$), $f_{c0}^T$ can be expressed as:

$$f_{c0}^T = \beta_{0T} f_{c0} \tag{1}$$

The variation of ${\beta_{0T}}^{Exp} = {f_{c0}^T}^{Exp} / {f_{c0}}^{Exp}$ with respect to $T_m$ can be obtained from experimental results of axial compression tests on HC columns. In the present study, a database of 292 HC column specimens with a wide range of concrete properties and exposure temperature was collected, as briefly presented in Table 1. The following criteria were adopted to include/exclude experimental data: 1) ones obtained from circular/square/rectangular heat-damaged concrete specimens tested under concentric were included; 2) heat-damaged concrete specimens subjected to a maximum exposure temperature more than 800 °C were excluded; 3) light-weight concrete specimens were excluded; 4) light-weight concrete, and recycled aggregate concrete specimens were excluded; 5) data registered experimentally with incomplete documented information, i.e. maximum exposure temperature, geometry details and material properties, were excluded.

Fig. 2a shows the variation of $\beta_{0T}{}^{Exp}$ with respect to $T_m$, based on the results of the database, where a decrease of $\beta_{0T}{}^{Exp}$ with the increase of $T_m$, from 1 to almost 0.2 corresponding to room conditions and 800 °C, respectively, is visible. Based on the best-fit relation obtained from regression analysis on the database information, $\beta_{0T}$ versus $T_m$ data can be obtained as $\beta_{0T} = 1.087 - 0.00116T_m$. By assuming *Error Index* as $\left(1.087 - 0.00116T_m\right) / \beta_{0T}{}^{Exp}$, Fig. 2b demonstrates that there is a slight variation in Error Index versus concrete strength relationship which is less than one up to almost $f_{c0}$ = 110 MPa, representing underestimation. However, beyond $f_{c0}$ = 110 MPa, the Error Index tend to be more than 1 resulting in overestimation of the experimental counterparts. Accordingly, based on regression analysis performed on the Error Index and concrete strength relationship, $\beta_{0T}$ can be calculated by:

$$\beta_{0T} = \frac{1.087 - 0.00116T_m}{\gamma_f} \leq 1 \qquad (2)$$

in which

$$\gamma_f = 1 + \left(\gamma_0 - 1\right)\left(\frac{T_m - 25}{100}\right) \qquad \text{for } T_m \leq 100\ °\text{C} \qquad (3a)$$

$$\gamma_f = \gamma_0 \qquad \text{for } T_m \geq 100\ °\text{C} \qquad (3b)$$

$$\gamma_0 = 3415\left(\frac{f_{c0}}{1000}\right)^3 - 721\left(\frac{f_{c0}}{1000}\right)^2 + 44.5\left(\frac{f_{c0}}{1000}\right) + 0.178 \qquad (4)$$

where the developed expression is valid for $T_m \leq 800$ °C based on the interval of the submitted maximum exposure temperatures ($T_m = [25$ °C , $800$ °C$]$) in the database used for the regression analysis. $\gamma_0$ reflects the influence of $f_{c0}$ (in MPa) in the determination of $\beta_{0T}$ empirically.

In Fig. 2c, the results predicted by the proposed model are compared to those reported by the experiments (Table 2), with a mean = 0.964, a coefficient of variation (CoV) = 0.279, a mean absolute percentage error (MAPE) = 0.203, and an R-squared value ($R^2$) = 0.876, revealing an acceptable predictive performance. Table 2 also shows that the proposed model provides a predictive performance better than of the existing models.

## 2-2- Axial strain at the peak stage of AC columns ($\varepsilon_{c0}$)

For the case of AC columns, the experimental and analytical studies conducted by [33-38] evidenced that $\varepsilon_{c0}$ increases with the concrete compressive strength ($f_{c0}$). Jansen and Shah [34] experimentally demonstrated that the column aspect ratio ($\lambda_L$ as the ratio of the column height to its diameter) has also considerable influence on $\varepsilon_{c0}$ due to the occurrence of strain-localization within a finite zone with a pronounced gradient of deformations due to the concrete post-peak strain-softening behavior. In this study, in order to estimate $\varepsilon_{c0}$, a large database (Table 3) was compiled from the experimental results available in the literature, resulting in 604 unconfined concrete specimens (AC) with a broad range of concrete properties and geometry configurations. Note that for the case of non-circular columns with a total cross-section area of $A_g$, based on Yang *et al.* [38] recommendations, $\lambda_L$ can be determined as $L/d_{eq}$

where $L$ is the columns' height and $d_{eq}$ is the equivalent circular diameter ($d_{eq} = \sqrt{4A_g/\pi}$ [38]).

Based on a preliminary sensitivity analysis, a low effect was achieved for the influence of the column size (i.e. the normalized $d_{eq}/150$, with $d_{eq}$ in mm) to estimate $\varepsilon_{c0}$ when compared to other influencing factors ($f_{c0}$ and $\lambda_L$). Accordingly, using regression analysis, the best-fit expression to predict $\varepsilon_{c0}$ was derived as a function of $f_{c0}$ and $\lambda_L$ regardless of the column's cross-section dimension influence:

$$\varepsilon_{c0} = 0.0011\left(\frac{f_{c0}}{\lambda_L}\right)^{0.25} \tag{5}$$

Table 4 evaluates the predictive performance of this relation with the results of the experimental tests, and also compares with that of existing models. Based on the assessment indicators (values of mean = 0.977, CoV = 0.184 and MAPE = 0.138), there is an acceptable agreement between model prediction and the experimental results. Furthermore, compared to the models recommended by Popovics [39], Karthik and Mander [40] and Lim and Ozbakkaloglu [41], it is the most accurate one, confirming its reliability.

## 2-3- Axial strain at the peak stage of HC columns ($\varepsilon_{c0}^T$)

The experiments with HC columns conducted by Chang *et al.* [22], Sharma *et al.* [24], Xiao *et al.* [26], Xiang *et al.* [27] evidenced that the axial strain ($\varepsilon_{c0}^T$) corresponding to $f_{c0}^T$ tends to increase significantly from $\varepsilon_{c0}$ at ambient condition to $\varepsilon_{c0}^T \gg \varepsilon_{c0}$ at elevated temperature, as shown in Fig. 3a.

The details of the experimental specimens in the assembled database, including 225 tested HC columns, is presented in Table 5.

The best-fit relation between $\varepsilon_{c0}^{T\ Exp}/\varepsilon_{c0}-1$ (representing the thermal damage-induced strain) and $T_m$ was obtained from regression analysis by considering the influence of $f_{c0}$, resulting:

$$\varepsilon_{c0}^{T}=\left(1+63 f_{c0}^{-0.5}\left(\frac{T_m}{1000}\right)^{4.2}\right)\frac{\varepsilon_{c0}}{\alpha_T}\leq 4.5\frac{\varepsilon_{c0}}{\alpha_T} \qquad (6)$$

in which

$$\alpha_T=1 \qquad \text{for } T_m\leq 100\ °\text{C} \qquad (7a)$$

$$\alpha_T=1.22-0.0025T_m+3\times 10^{-6}T_m^{\ 2} \qquad \text{for } T_m>100\ °\text{C} \qquad (7b)$$

where the developed expression is valid for $T_m\leq 800\ °\text{C}$ based on the interval of the submitted maximum exposure temperatures ($T_m=\left[25\ °\text{C}\ ,\ 800\ °\text{C}\right]$) in the database used for the regression analysis. $\alpha_T$ is the calibration factor for the influence of $T_m$ in the increase of axial strain induced by thermal damage, obtained from the regression analysis. In Fig. 3b and Table 6, the predictive performance of this model is assessed based on 225 test specimen results. As can be seen, Eq. (6) provides the most accurate model compared to the existing models in the prediction of the experimental counterparts, even though conservative results were achieved for some cases submitted to high level of exposure temperature.

## 3-Dilation behavior of FRP confined HC columns

### 3-1- Confinement pressure developed for ambient condition

For the case of FRP fully confined square AC columns (FFSC), based on the force equilibrium at the cross-sectional level, confinement pressure ($f_{l,f}$) generated by the FRP confining stress ($f_f$) can be expressed as (Shayanfar *et al.* [17]):

$$f_{l,f} = 2K_e \frac{n_f t_f}{D_{eq}} f_f \tag{8}$$

where $n_f$ is the number of FRP layers; and $t_f$ is the nominal thickness of one FRP layer. In Eq. (8), $D_{eq}$ defines the diameter of the equivalent circular cross-section for columns of square cross section with *b* edge and *r* corner radius, which can be calculated as recommended by Shayanfar *et al.* [17]:

$$D_{eq} = \frac{1-0.215{R_b}^2}{1-0.215R_b} b \tag{a9}$$

where

$$R_b = 2r/b \tag{b9}$$

is the corner radius ratio. Note that by using $D_{eq}$ in the determination of FRP confinement pressure, FRP volumetric ratio in the equivalent circular cross-section would be identical to that of original square cross-section column. In Eq. (8), $K_e$ is the confinement efficiency factor. Shayanfar *et al.* [17] modified the original concept of '*confinement efficiency factor*' by considering the impact of concrete expansion gradient in the establishment of confinement pressure, besides the well-known phenomenon of arching action. By using this concept, the actual confinement pressure acting non-homogenously on the concrete is converted to an equivalent confinement pressure with uniform distribution along transverse and longitudinal directions of the column. This factor includes two components, which can be determined as suggested by Shayanfar *et al.* [17]:

$$K_e = K_H K_V \tag{10}$$

where $K_H$ is the horizontal component, reflecting the influence of horizontal arching action on the distribution of confinement pressure within the cross-section of a non-circular columns (for circular columns, $K_H = 1$), determined as:

$$K_H = R_b \geq 0.07 \tag{11}$$

In Eq. (10), $K_V$ is the vertical component reflecting the influence of the gradient of concrete lateral expansion along the column height, depending on the level of confinement stiffness (the ratio of confinement pressure to concrete lateral strain). It was demonstrated by Shayanfar *et al.* [16] that above a certain level of confinement stiffness, the confinement imposed to the concrete is strong enough to strictly control the evolution of concrete expansion leading to an almost null gradient along the vertical direction ($K_V = 1$). However, for the cases with an insufficient confinement stiffness, due to the lack of strong restriction in the curtailment of concrete expansibility, the concrete column is expected to experience a highly non-homogenous distribution of concrete expansion and, consequently, the confinement pressure in the axial loading direction is non-uniform. Shayanfar *et al.* [17] suggested a design-based formulation to calculate $K_V$ as follows:

$$K_V = \frac{1}{3} + \frac{2}{3} k_\varepsilon \tag{12}$$

in which

$$k_\varepsilon = 0.08 + 0.92\left[2\frac{I_f}{I_f^*} - \left(\frac{I_f}{I_f^*}\right)^2\right] \leq 1 \qquad \text{for } I_f \leq I_f^* \tag{13a}$$

$$k_\varepsilon = 1 \qquad \text{for } I_f > I_f^* \tag{13b}$$

$$I_f^* = 0.06 + 0.0005 f_{c0} \tag{14}$$

$$I_f = 2K_H \frac{n_f t_f E_f \varepsilon_{c0}}{D_{eq} f_{c0}} \simeq K_H \frac{n_f t_f E_f}{550 D_{eq} f_{c0}^{\ 0.75}} \tag{15}$$

where $k_\varepsilon$ represents the ratio of minimum and maximum concrete lateral expansion along the column height. $I_f$ represents the confinement stiffness index regardless the influence of the gradient of concrete expansion along the column height. Finally, $I_f^*$ is the confinement stiffness index above which $K_V = 1$, representing the homogenous concrete expansion along the column due to strong restrictions imposed to the concrete.

In this paper, for further simplification of the relative complexity of Eqs. (12-15) in the calculation of $K_V$, a simplified equation was developed based on a preliminary sensitivity analysis on the influencing factors in Eq. (12) as:

$$K_V = 2.2\left(I_f\right)^{0.3} \leq 1 \tag{16}$$

Accordingly, by using the design-based Eqs. (11, 15 and 16), the two components involved in $K_e$ (Eq. (10)) can be calculated.

### 3-2- Confinement pressure developed for elevated condition

Based on the model developed for ambient conditions in the previous section, the confinement pressure ( $f_{l,f}^T$ ) imposed by the FRP confining stress ( $f_f$ ) to HC column can be expressed as:

$$f_{l,f}^T = 2K_e^T \frac{n_f t_f}{D_{eq}} f_f^T \tag{17}$$

in which

$$K_e^T = K_H K_V^T \tag{18}$$

where $K_H$ can be determined by Eq. (11). Through the substitution of confinement stiffness index of FRP confined HC columns ($I_f^T$) with that of FRP confined AC ones ($I_f$) in Eq. (16), $K_V^T$ can be expressed as:

$$K_V^T = 2.2\left(I_f^T\right)^{0.3} = 2.2\beta_{0T}{}^{-0.45}\left(I_f\right)^{0.3} \le 1 \tag{19}$$

in which based on Eq. (15),

$$I_f^T = K_H \frac{n_f t_f E_f}{550 D_{eq}\left(f_{c0}^T\right)^{0.75}} = \beta_{0T}{}^{-0.75} I_f \tag{20}$$

where $\beta_{0T}$ is the axial strength ratio calculated by Eq. (2), which is equal to 1 for concrete at ambient condition; Considering that $f_f^T$ can be calculated as $E_f \varepsilon_h^T = E_f \varepsilon_l^T$ (where $\varepsilon_h^T$ and $\varepsilon_l^T$ are the generated circumferential (hoop) and radial strains, respectively, and $E_f$ is the FRP modulus elasticity), Eq. (17) is rearranged as:

$$f_{l,f}^T = 2K_e^T \frac{n_f t_f}{D_{eq}} E_f \varepsilon_h{}^T \tag{21}$$

Based on Poisson's ratio effect ($\varepsilon_h^T = \varepsilon_l^T = v_s^T \varepsilon_c$, where $v_s^T$ is the secant Poisson's ratio), Eq. (21) can be rearranged as:

$$f_{l,f}{}^T = 2K_e \frac{n_f t_f}{D_{eq}} E_f v_s{}^T \varepsilon_c \tag{22}$$

Accordingly, in order to calculate $f_{l,f}^T$ imposed to the concrete at a certain level of $\varepsilon_c$, the corresponding $v_s^T$ is required to be addressed, which will be presented in the following section.

### 3-3-Dilation mechanism at elevated conditions

During axial compressive loading, after splitting cracks have occurred, by increasing the axial strain, the development of concrete lateral expansion abruptly increases due to Poisson's ratio effect. Experimental studies conducted by Barros and Ferreira [5], Mirmiran and Shahawy [46], Lim and Ozbakkaloglu [47] and Zeng *et al.* [48] evidenced that the magnitude of concrete dilatancy is strongly dependent on confinement stiffness imposed to the concrete. For the case of AC with a high level of confinement stiffness capable of limiting the evolution of concrete expansion and splitting cracks, a remarkable enhancement in terms of axial strength and deformability is obtained (Barros and Ferreira [5]). However, for the case of low confinement stiffness, the confinement pressure imposed to the concrete is not able to overcome the concrete tendency for abrupt expansion, leading to lower confinement-induced enhancements [49].

For a preliminary assessment of the dilation response of FRP confined HC columns, the experimental dilation results conducted by Ouyang *et al.* [12] are analyzed. All tests were conducted with specimens of diameter and height of 150 mm and 300 mm, respectively. The unconfined concrete compressive strength at the ambient condition was reported 45.1 MPa. Basalt FRP (BFRP) was used with the values of thickness, modulus of elasticity and rupture strain of 0.121 mm, 108.3 GPa and 2.18%, respectively. The HC columns were subjected initially to various levels of maximum temperature (200 °C, 400 °C, 600 °C and 800 °C). Then, they were fully confined with two and four layers of BFRP. Fig. 4 demonstrates the experimental dilation responses of BFRP confined HC specimens reported by Ouyang *et al.* [12]. Here, $\varepsilon_V$ is the volumetric strain determined as $\varepsilon_V = \varepsilon_c - 2\varepsilon_l = (1-2\nu_s)\varepsilon_c$ in which $\nu_s$ is the secant Poisson's ratio as $\nu_s = \varepsilon_l / \varepsilon_c$. Moreover, the negative and positive values of $\varepsilon_V$ represent volumetric expansion and contraction, respectively. To better demonstrate the contribution of thermal-induced damage level in terms of dilation behavior, the model developed by Shayanfar *et al.* [8 and 17] was followed to determine the dilation results associated with FRP confined AC specimens (***T25-L2*** and ***T25-L4*** with red solid lines). Note that Ti-Lj refers to the concrete column heated up to the i-th maximum exposure temperature (Ti) and then, confined by j layers of BFRP.

As can be seen in Fig. 4, for all cases, regardless the level of thermal-induced damage, initial behavior up to transition zone is virtually the same. However, beyond the transition zone, there is a noticeable difference between the transverse expansibility of HC and AC specimens. Fig. 4a reveals that at a certain axial strain ($\varepsilon_c$), lateral strain ($\varepsilon_l$) for the cases of T200-L2 and T400-L2 was obtained significantly higher than that of T25-L2, demonstrating the effect of thermal damage on increase of $\varepsilon_l$. Likewise, from Fig. 4b, T200-L2 and T400-L2 have experienced a large incremental volumetric expansion. However, in the T25-L2, beyond $\varepsilon_c = 0.008$, a considerable decrease in the magnitude of the increase in volumetric strain ($\varepsilon_v$) with respect to $\varepsilon_c$, followed by a reverse in volumetric evolution around $\varepsilon_c$ =0.02, reveals the capability of the confinement system imposed to AC column (T25-L2) in limiting the transverse expansibility of AC columns. Likewise, as demonstrated in Fig. 4c, for T25-L2, due to the adequate activated confinement imposed to AC column to overcome its tendency for lateral expansibility, beyond the peak stage, $v_s$ trend followed a decreasing branch. Even though, heat-induced expansion for HC columns leads to an earlier activation in passive confining system of T200-L2 and T400-L2, the applied confinement was not adequate to strongly constrain the concrete expansion evolution, based on its abrupt increase in $v_s$ after transition zone. On the other hand, for the cases subjected to high level of temperature (T600-L2 and T800-L2), as shown in Fig. 4a, at a certain level of $\varepsilon_l$, T600-L2 and T800-L2 experienced a larger axial deformation depending on thermal damage level, compared to T25-L2, T200-L2 and T400-L2. Likewise, Fig. 4b reveals that up to a certain level of axial strain, the changes in volumetric evolution for T600-L2 and T800-L2 were almost marginal, while they underwent large axial deformations. Nonetheless, above this axial strain level, as a consequence of the degeneration of micro- into meso- and macro-cracks along with heat-induced damage in the concrete, the volumetric change evolution was suddenly reversed triggering an abrupt increase in volumetric expansion. Fig. 4c also shows that for T600-L2 and T800-L2, compared to the other cases, larger axial strains were obtained for a certain Poisson's ratio. Furthermore, a closer evaluation of the data demonstrates that the maximum secant Poisson's ratio decreases significantly with increasing thermal damage, which can be attributed to the substantial contribution of the heat-induced damage level in the establishment of dilation behavior of HC. Accordingly, by applying a certain level of axial loading, the heat-induced damage leads to an additional axial strain in HC columns, and alters their transverse expansibility, dependent strongly on thermal damage level. The comparison of dilation responses shown in Fig. 4a and Fig. 4d confirms a significant reduction in terms of lateral strain by increasing FRP thickness (confinement stiffness), predominantly beyond the transition zone. Fig. 4e reveals that an increase in confinement stiffness leads to shorter volumetric expansion due to the stronger restrictions imposed to the concrete expansibility. The relations of $v_s$ and $\varepsilon_c$ shown in Fig. 4f also confirms this behavior, where the specimens with more FRP thickness experienced a lower value of $v_s$ than those with less thickness (Fig. 4c).

In this study, it is aimed to extend the dilation model of Shayanfar *et al.* [8,17] originally suggested for FRP confined AC specimens to the case of FRP confined HC column through formulating the relation between $v_s^T$ and $\varepsilon_c$ at different levels of heat-induced damage ($T_m$) based on regression analysis. For this purpose, a dataset of the dilation responses obtained from existing experimental data was collected as presented by Table 7. It should be noted that the following criteria were adopted to include/exclude experimental data: 1) ones obtained from concentric loading tests were included; 2) Specimens with a full confinement of unidirectional fibers installed transversely to the axial compression direction were included; 3) Specimens with helical wrapping configuration or hybrid confinement arrangements were excluded; 4) specimens of rectangular cross-section with transverse and longitudinal steel reinforcements were excluded; 5) ones registered experimentally with incomplete documented information, i.e. maximum exposure temperature, geometry details and material properties, were excluded; 6) ones obtained from test specimens with a premature failure mode caused by FRP debonding were excluded. In this table, $v_{s,\max}^{T}$ represents the peak Poisson's ratio of HC columns confined by FRP. $\eta_T$ defines the ratio of $v_{s,\max}^{T}$ to $v_{s,\max}^{A}$ (where $v_{s,\max}^{A}$ is the peak Poisson's ratio of FRP confined AC columns at ambient temperature). For the calculation of $v_{s,\max}^{A}$, in the present study, the confinement stiffness-based model developed by Shayanfar *et al.* [17] was followed, which is determined as a main function of the confinement stiffness ($\rho_{K,f}$), namely:

$$v_{s,\max}^{A} = \frac{0.25}{\left(1+L_{d0}/D_{eq}\right)\sqrt{\rho_{K,f}}} \tag{23}$$

in which (by considering Eqs. (10), (15) and (16))

$$\rho_{K,f} = K_V I_f = K_e \frac{n_f t_f E_f}{550 D_{eq} f_{c0}^{0.75}} \tag{24}$$

$$0.57 \le \frac{L_{d0}}{\sqrt{A_g}\psi_f} = 1.71 - 3.53\times10^{-5} A_g \le 1.36 \tag{25}$$

$$\psi_f = \frac{6.3}{\sqrt{f_{c0}}} \le 1 \tag{26}$$

where $L_{d0}$ is the compression damage zone length of unconfined concrete columns which was determined as recommended by Lertsrisakulrat *et al.* [50]; $A_g$ is the total area of the column's cross section.

For the determination of $v_{s,\max}^{T}$, it can be considered a function of $v_{s,\max}^{A}$ by

$$v_{s,\max}^{T}=\frac{v_{s,\max}^{T}}{v_{s,\max}^{A}}v_{s,\max}^{A}=\eta_T v_{s,\max}^{A} \tag{27}$$

Therefore, by calculating $v_{s,\max}^{A}$ using Eq. (23) for the specimens assembled in the database, their corresponding values of $\eta_T$ can be obtained as $\eta_T{}^{Exp}=v_{s,\max}^{T\ \text{Exp}}/v_{s,\max}^{A}$ (Table 7). Fig. 5a shows the variation of $\eta_T{}^{Exp}$ with respect to $T_m$. As can be seen, by increasing $T_m$ in the interval $25-400$ °C, $\eta_T{}^{Exp}$ significantly increases up to the peak, while for $T_m>400$ °C, a noticeable reduction of $\eta_T{}^{Exp}$ with the increase of $T_m$ is observed. Based on the best-fit relation obtained from regression analysis performed on 78 experimental data, the following equation was derived for determining $\eta_T$ from $T_m$ and $R_b$ (Fig. 5a):

$$\eta_T=\frac{33.2\left(\frac{T_m}{1000}\right)^3-51\left(\frac{T_m}{1000}\right)^2+21.2\left(\frac{T_m}{1000}\right)-0.49}{1.65-0.65R_b}\geq 1 \qquad \text{for } T_m\leq 100\ °\text{C} \tag{28a}$$

$$\eta_T=\frac{33.2\left(\frac{T_m}{1000}\right)^3-51\left(\frac{T_m}{1000}\right)^2+21.2\left(\frac{T_m}{1000}\right)-0.49}{1.65-0.65R_b}\leq 2 \qquad \text{for } 100\ °\text{C}<T_m\leq 800\ °\text{C} \tag{28b}$$

where the developed expression is valid for $T_m\leq 800$ °C based on the interval of the submitted maximum exposure temperatures ($T_m=[25\ °\text{C},\ 800\ °\text{C}]$) in the database used for the regression analysis. Therefore, by addressing $v_{s,\max}^{A}$ and $\eta_T$ in Eq. (27) through Eqs. (23, 28), the peak Poisson's ratio of FRP confined HC columns ($v^T_{s,\max}$) can be calculated. Fig. 5b evaluates the performance of Eq. (27). Based on the obtained assessment indicators, the established expression revealed a good predictive performance.

For the determination of Poisson's ratio of HC columns confined by FRP ($v^T_s$) during axial loading, $v^T_s$ is considered as a function of $v^T_{s,\max}$, resulting in

$$v^T_s = \frac{v^T_s}{v^T_{s,\max}} v^T_{s,\max} = \eta_\varepsilon v^T_{s,\max} \tag{29}$$

where $\eta_\varepsilon$ represents the ratio of $v^T_s$ and $v^T_{s,\max}$ at a given axial strain ($\varepsilon_c$). Introducing Eq. (27) into Eq. (29), $v^T_s$ is suggested by

$$v^T_s = \eta_\varepsilon \eta_T v^A_{s,\max} \tag{30}$$

To calculate $v^T_s$ by Eq. (30), $\eta_\varepsilon$ needs to be addressed as an input parameter. For the determination of $\eta_\varepsilon$ at a given level of $\varepsilon_c$ for FRP confined AC columns (at ambient condition), Shayanfar *et al.* [8] proposed a simple but reliable multilinear model as shown in Fig. 6a. in this figure, $c_0$ $c_1$, $c_2$, $c_3$, $c_4$ and $c_5$ are the calibration factors reflecting the influence of confinement stiffness on $\eta_\varepsilon$ versus $\varepsilon_c$ relation; $\varepsilon_{c,m}$ represents the axial strain corresponding to $v^A_{s,\max}$; $v_{s,0}$ is the initial Poisson's coefficient of unconfined AC columns, which was determined by (Candappa *et al.* [51]):

$$v_{s,0} = 8\times10^{-6} f_{c0}{}^2 + 2\times10^{-4} f_{c0} + 0.138 \tag{31}$$

As can be seen in Fig. 6a, the pre- and post-peak phases are dependent of $\rho_{K,f}$. The pre-peak relation demonstrates that concrete initially behaves similar to unconfined AC column with initial Poisson's coefficient as $v_{s,0}$. Afterward ($\varepsilon_c > \varepsilon_{c0}$), $\eta_\varepsilon$ increases up to the peak stage ($\eta_\varepsilon = 1$) at $\varepsilon_{c,m}$. The post-peak relation shows that $\eta_\varepsilon$ decreases with the increase of $\varepsilon_c$, whose reduction magnitude is dependent on $\rho_{K,f}$. In the present study, the relation developed by Shayanfar *et al.* [8] was extended for the case of FRP confined HC columns. For this purpose, as presented in Fig. 6b, a new relation of $\eta_\varepsilon$ versus $\varepsilon_c$ was developed where the influence of thermal damage was reflected by the parameter $\beta_\varepsilon$.

$$\beta_\varepsilon = \beta_\rho \left( \varepsilon_{c0}^T - \varepsilon_{c0} \right) \tag{32}$$

in which

$$0.4 \leq \beta_\rho = 11 \rho_{K,f}^{\ 0.75} \leq 1.4 \tag{33}$$

Accordingly, by calculating $\beta_\varepsilon$ through Eq. (32), $\eta_\varepsilon$ at a given $\varepsilon_c$ can be obtained from the data presented in Fig 6b. Then, the corresponding $v_s^T$ can be calculated by Eq. (30). To evaluate the reliability of the proposed relation, the results obtained from the experiments conducted by Ouyang *et al.* [12] and those determined by Eq. (30) are compared Fig. 7. As can be seen, the model has a good agreement with the experimental counterparts.

To obtain $\varepsilon_l^T$ versus $\varepsilon_c$ response of FFCC-H/FFSC-H based on the developed dilation model, the calculation procedure is summarized as:

1. Determine $K_e$ using Eqs. (10), (11) and (16)
2. Determine $\rho_{K,f}$ using Eq. (24)
3. Determine $v_{s,\max}^A$ using Eqs. (23)
4. Determine $\eta_T$ using Eq. (28)

**5.** Assume a value for axial strain ($\varepsilon_c$)

**6.** Determine $\beta_\varepsilon$ using Eqs. (32), (5) to (7)

**7.** Determine $\eta_\varepsilon$ using the developed multilinear model in Fig. 6b

**8.** Determine $v_s^T$ using Eq. (29)

**9.** Determine $\varepsilon_l^T$ as $\varepsilon_l^T = v_s^T \varepsilon_c$

**10.** Continue the steps 5 to 9 up to the aim maximum value of $\varepsilon_c$, resulting a $\varepsilon_l^T$ versus $\varepsilon_c$ relation.

For the assessment of the model capability in the prediction of dilation response, Fig. 8 compares $\varepsilon_l^T$ versus $\varepsilon_c$ relations extracted from the experimental tests conducted by Ouyang *et al.* [12] and those simulated by the proposed model. As shown, the experimental results were simulated suitably by the model confirming its reliable predictive performance.

## 4- Axial Compressive Stress-strain Relation

Fig. 9 demonstrates the remarkable influence of the exposure temperature on the axial response of HC columns fully confined by FRP, tested by Bisby *et al.* [4] and Ouyang *et al.* [12]. As can be seen in Fig. 9a for the case of the results reported by Bisby *et al.* [4], compared to T25-L1 (control experimental specimen), even though T300-L1 experienced a slight heat-induced reduction in terms of initial axial stiffness, it showed higher axial strength and deformability. For T500-L1, heat- induced damage did not lead to a significant difference in the axial stress versus axial strain relationship. However, for the case of severely heat-damaged specimens (T686-L1), there is a noticeable reduction in its axial stiffness, compared to T25-L1. Moreover, the axial stress versus axial strain relationship of T686-L1 was almost linear, compared to almost bi-linear curves of specimens with the exposure temperatures lower than or equal to 500 °C. From the secant axial stiffness versus axial strain curves in Fig. 9b, T25-L1, T300-L1 and

T500-L1 presented a reduction of the axial stiffness during the axial compressive loading. However, in T686-L1, after an initial relatively small reduction of the stiffness, it was preserved almost constant during the loading process, a consequence of the confinement effect provided by the FRP. The assessment of the exposure temperature effects on the test specimens conducted by Ouyang *et al.* [12] revealed that the axial strength and stiffness of specimens subjected up to 400 °C were higher than of T25-L2. It can be attributed to earlier activation of confining system in the cases of HC than AC due to the increase of concrete expansibility caused by the heat-induced damage. However, by increasing the temperature above that limit of 400 °C, thermal-induced damage increases significantly the axial deformation of the concrete specimen, converting a bi-linear stress-strain response into a linear one (Fig. 9c), and consequently an almost constant axial stiffness during axial loading process (Fig. 9d). Accordingly, the experimental results presented in Fig. 9 evidence the imperious impact of exposure maximum temperatures on the axial stress-strain response of FRP confined HC columns.

In order to calculate the axial stress versus axial strain relationship ( $f_c^A$ *vs* $\varepsilon_c$ curve) of FRP passively confined AC columns (passively-confined-concrete), *Active Confinement Approach* (i.e. [14-17, 52-54] can be followed. In this approach, the axial response of concrete with passive confinement is derived based on that of actively-confined-concrete subjected to a constant confinement pressure during the entire axial loading history. Accordingly, $f_c^A$ corresponding to $\varepsilon_c$ at a certain level of FRP confinement pressure ( $f_{l,f}^A$ ) can be calculated by adopting an axial stress-strain base relation model ( $f_c^A = g_1\left(f_{cc}^A, \varepsilon_c\right)$ ) coupled with an axial strength model, also known as failure surface function, ( $f_{cc}^A = g_2\left(f_{l,f}^A\right)$ ) developed for actively-confined-concrete. Here, $f_{cc}^A$ is the peak axial strength of axial stress-strain base relation model (defined from function $g_1$). However, since actively-confined-concrete case is under a constant $f_{l,f}{}^{A}$ during the entire axial loading history, contrary to passively-confined-concrete, the studies [15-17] evidenced that the original *Active Confinement Approach* overestimates the FRP-induced enhancement of passively-confined-concrete, which is generally recognized as *Confinement Path Effect*. To consider this effect, by adopting functions $g_1$ and $g_2$ from those exclusively developed for actively-confined-concrete, Lim and Ozbakkaloglu [15] recommended a reduction factor in the confinement-induced enhancements obtained for actively-confined-concrete, by decreasing the level of the confinement pressure $f_{l,f}{}^{A}$ in the function $g_2$. Shayanfar *et al.* [16 and 17] proposed a new axial strength framework model (function $g_2$) exclusively suggested for passively-confined-concrete to predict enhancements offered by a passive confining system.

In the present paper, for the calculation of $f_c{}^{A}$ and $\varepsilon_c$ relation of FRP confined AC columns taking into account *confinement path effect*, the model developed by Shayanfar *et al.* [17] presenting a unified character for both full and partial confinement configurations and both circular and square cross-sections, is followed. In this model, the axial stress-strain base framework ( $f_c{}^{A}$ versus $\varepsilon_c$ relation using function $g_1$) is given by:

$$f_c{}^{A} = f_{cc}{}^{A} \frac{\left(\varepsilon_c / \varepsilon_{cc}{}^{A}\right) n_A}{n_A - 1 + \left(\varepsilon_c / \varepsilon_{cc}{}^{A}\right)^{n_A}} \tag{34}$$

in which

$$\frac{f_{cc}{}^{A}}{f_{c0}} = 1 + \frac{R_1}{R_2}\left(\frac{f_{l,f}{}^{A}}{f_{c0}}\right)^{R_2} \tag{35}$$

$$\frac{\varepsilon_{cc}{}^{A}}{\varepsilon_{c0}} = 1 + 5\left(\frac{f_{cc}{}^{A}}{f_{c0}} - 1\right) \tag{36}$$

$$n_A = \frac{E_c}{E_c - \dfrac{f_{cc}{}^{A}}{\varepsilon_{cc}{}^{A}}} \simeq \frac{1}{1 - 2.1\times10^{-4}\psi_A} \geq 1.1 \tag{37}$$

$$\psi_A = \frac{f_{cc}{}^{A}}{\varepsilon_{cc}{}^{A}\sqrt{f_{c0}}} \tag{38}$$

where $f_{cc}{}^{A}$ and $\varepsilon_{cc}{}^{A}$ are the peak axial strength and its corresponding axial strain, calibrated based on experimental AC specimens with passively confining system; $R_1$ and $R_2$ are the calibration factors in the determination of $f_{cc}{}^{A}$; $n_A$ is the concrete brittleness at the ambient condition depending on $\psi_A$, as recommended by Carreira and Chu [55]. Accordingly, at a certain level of $\varepsilon_c$, $f_{cc}{}^{A}$ can be determined as a function of $f_{l,f}{}^{A}$ based on Eq. (35), and $\varepsilon_{cc}{}^{A}$ can be, subsequently, calculated by Eq. (36), as input parameters for the determination of $f_c{}^{A}$ by Eq. (34).

In this study, the model developed by Shayanfar *et al.* [17] was extended to be applicable for the establishment of axial response of the concrete being subjected to a certain exposure temperature. Accordingly, by substituting the mechanical characteristics of HC column with those of AC column, at a certain level of $\varepsilon_c$ leading to FRP confinement pressure ( $f_{l,f}^{T}$ ), axial stress-strain base relation model ( $f_c{}^{T}$ versus $\varepsilon_c$ curve using function *g1*) can be expressed as:

$$f_c^T = f_{cc}^T \frac{\left(\varepsilon_c/\varepsilon_{cc}^T\right)n_T}{n_T - 1 + \left(\varepsilon_c/\varepsilon_{cc}^T\right)^{n_T}} \tag{39}$$

in which

$$n_T = \frac{1}{1-2.1\times10^{-4}\psi_T} \geq 1.1 \qquad \text{for } T_m \leq 400\ °\text{C} \tag{40a}$$

$$n_T = 2 - \frac{1-4.2\times10^{-4}\psi_T}{1-2.1\times10^{-4}\psi_T}\left(2-\frac{T_m}{400}\right) \geq 1.1 \qquad \text{for } 400\ °\text{C} \leq T_m \leq 800\ °\text{C} \tag{40b}$$

$$\psi_T = \frac{f_{cc}^T}{\varepsilon_{cc}^T\sqrt{f_{c0}^T}} \tag{41}$$

where $f_{cc}^T$ and $\varepsilon_{cc}^T$ are the peak axial strength and its corresponding axial strain at a certain level of $\varepsilon_c$ as input parameters for the axial stress-strain base relation model. $n_T$ is the concrete brittleness that considers the influence of the maximum exposure temperature on $f_{cc}^T$, $\varepsilon_{cc}^T$ and $f_{c0}^T$ through the parameter $\psi_T$. Note that the axial stiffness of FRP confined HC columns with severe thermal-induced damages ($T_m \simeq 600-800\ °\text{C}$) seems to be almost constant with a linear axial behavior during axial loading as demonstrated in Fig. 9. Accordingly, in the present study, $n_T$ corresponding to $T_m = 800\ °\text{C}$ was assumed equal to a constant value of $n_T = 2$ during the entire loading history, adjusted based on the best-fit relation of the proposed model with the experimental axial stress-strain responses reported by Bisby *et al.* [4], Lenwari *et al.* [10], Ouyang *et al.* [12] and Song *et al.* [13]. Consequently, in Eq. (40b) ($400\ °\text{C} \leq T_m \leq 800\ °\text{C}$), $n_T$ was considered on the interval of the $n_T$ obtained from Eq. (40a) for $T_m = 400\ °\text{C}$ and $n_T = 2$ corresponding to $T_m = 800\ °\text{C}$, with a linear relationship with $T_m$.

For the establishment of $\varepsilon_{cc}^T$ at high temperatures, the preliminary comparative assessment of the proposed model with $\varepsilon_{cc}^T$ obtained based on Eq. (36) demonstrated a very significant underestimation in its model predictive performance, particularly for the cases with severe thermal-induced damages. On the other hand, based on experimental studies it was evidenced that the axial behavior of FRP confined HC columns tends to be similar to actively-confined-concrete due to more expansive behavior of HC than AC (Fig. 4). Therefore, the $\varepsilon_{cc}^{T}$ of HC columns with high exposure temperature is calculated according to the approach proposed by Lim and Ozbakkaloglu [41], exclusively developed for the case of actively-confined-concrete:

$$\varepsilon_{cc}^{T} = \varepsilon_{c0}^{T}\left[1+5\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}}-1\right)\right] \qquad \text{for } T_m \geq 100\ °\text{C} \tag{42a}$$

$$\varepsilon_{cc}^{T} = \varepsilon_{c0}^{T} + 0.045\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}}\right)^{1.15} + \lambda_{\varepsilon} \qquad \text{for } 100\ °\text{C} \leq T_m \leq 200\ °\text{C} \tag{42b}$$

$$\varepsilon_{cc}^{T} = \varepsilon_{c0}^{T} + 0.045\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}}\right)^{1.15} \qquad \text{for } T_m \geq 200\ °\text{C} \tag{42c}$$

in which

$$\lambda_{\varepsilon} = \left[0.045\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}}\right)^{1.15} - 5\varepsilon_{c0T}\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}}-1\right)\right]\left(\frac{T_m}{100}-2\right) \tag{43}$$

In this study, a new axial strength model was developed, applicable to FRP confined HC columns, having a unified character with Eq. (35) when the concrete is under ambient conditions:

$$\frac{f_{cc}^{T}}{f_{c0}^{T}} = 1+\frac{R_1}{R_2}\left(\overline{m}\frac{f_{l,f}^{T}}{f_{c0}^{T}}\right)^{R_2} + \frac{R_3}{R_4}\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}}\right)^{R_4} \tag{44}$$

where $R_1$, $R_2$, $R_3$ and $R_4$ are the calibration factors in the determination of $f_{cc}^{T}$ obtained from the regression analyses on FRP passively confined HC columns; $\overline{m}$ is the calibration factor reflecting the influence of more expansive behavior of the concrete with a certain level of thermal damage, which activates earlier the confining system compared to the AC column at ambient temperature (based on the comparative assessment on the dilation behavior of FFCC-

H/FFSC-H presented in Fig. 4). In Eq. (44), the second term ( $\frac{R_1}{R_2}\left(\overline{m}\frac{f_{l,f}^{T}}{f_{c0}^{T}}\right)^{R_2}$ ) represents the improvements obtained from the confinement of an AC column with axial strength equal to $f_{c0}^{T}$, considering the effect of earlier activation (through $\overline{m}$) compared to AC; The third term ( $\frac{R_3}{R_4}\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}}\right)^{R_4}$ ) in Eq. (44) considers the increase of confinement-induced improvements due to thermal-induced damage. Based on experimental axial stress versus axial strain relations, Shayanfar *et al.* [17] proposed new expressions to determine the calibration factors of $R_1$ and $R_2$, which was rearranged by substituting $f_{c0}$ by $f_{c0}^{T}$:

$$R_1 = \frac{23.9}{\beta_{0T}^{\ 0.5}\lambda_{fc}\lambda_{Rb}}\rho_{K,f}^{\ 0.67} \le 4.25 \tag{45}$$

$$R_2 = \frac{1.85}{\beta_{0T}^{\ 0.2}}\rho_{K,f}^{\ 0.26} \ge 0.3 \tag{46}$$

in which

$$\lambda_{fc} = 0.75 + 0.008\beta_{0T} f_{c0} \tag{47}$$

$$\lambda_{Rb} = 1.5\left(1 - 1.1R_b\right) \ge 1 \tag{48}$$

where $\lambda_{fc}$ and $\lambda_{Rb}$ reflect the impact of concrete axial strength and the dimension of corner radius in the calibration of $R_1$, respectively. In order to determine the calibration factors of $R_3$ and $R_4$ representing the effect of thermal-induced damage in terms of confinement-induced enhancements, the experimental axial stress-strain responses reported by Bisby *et al.* [4], Lenwari *et al.* [10], Ouyang *et al.* [12] and Song *et al.* [13] were used for statistical modeling.

Through regression analyses on the assembled data, the calibration factors $R_3$ and $R_4$ are determined from:

$$R_3 = \frac{\lambda_T}{\lambda_{rb}\lambda_K} \geq 0 \tag{49}$$

$$R_4 = 0.92\left(\frac{f_{l,f}^T}{f_{c0}^T}\right)^{0.1} \tag{50}$$

in which

$$\lambda_T = 3.55\left(\frac{T_m}{1000}\right) - 1.55 \geq 0 \tag{51}$$

$$\lambda_K = 1.15 - 0.022K_H \frac{n_f t_f E_f}{D_{eq} f_{c0}} \tag{52}$$

$$\lambda_{rb} = 1.22R_b^{0.25} \geq 0.85 \tag{53}$$

where $\lambda_T$, $\lambda_K$ and $\lambda_{rb}$ reflect the impact of maximum exposure temperature ($T_m$), confinement stiffness and the dimension of corner radius in the calibration of $R_3$, respectively. As shown in Fig. 10a, the developed model parameter ($\lambda_T$) provides a sufficient agreement with the experimental counterparts, which was determined based on the best-fit relation of the proposed model with the experimental axial stress-strain responses.

Due to the fact that pre-existing micro-cracks in thermally damaged concrete tend to activate passively confining system at the initial stage of axial compressive loading, the calibration factor of $\overline{m}$ was considered in the establishment of Eq. (44), having been obtained from regression analysis to enhance the confinement effectiveness, resulting:

$$\overline{m} = 1 + m_0 \exp\left(-11.2\frac{f_{l,f}^T}{f_{c0}^T}\right) \tag{54}$$

in which

$$m_0 = \frac{m_T}{m_\rho m_r} \tag{55}$$

$$0 \le m_T = 0.025\left(T_m - 100\right) \le 2.5 \qquad \text{for } T_m \le 400\ ^\circ\text{C} \tag{56a}$$

$$m_T = 2.5 - 0.01\left(T_m - 400\right) \ge 0.3 \qquad \text{for } T_m \ge 400\ ^\circ\text{C} \tag{56b}$$

$$m_r = 0.3 + 0.7 R_b \tag{57}$$

$$m_\rho = 0.2 {\beta_{0T}}^{0.3} {\rho_{K,f}}^{-0.4} \tag{58}$$

where $m_T$, $m_r$ and $m_\rho$ are the calibration factors determined based on the regression analysis reflecting the effect of maximum exposure temperature ($T_m$), corner radius ratio ($R_b$) and confinement stiffness ($\rho_{K,f}$). As shown in Fig. 10b, the developed model parameter ($m_T$) has a good agreement with the experimental counterparts, which extracted based on Eqs. (51 and 52) ($m_T^{Exp} = m_\rho m_r \dfrac{\overline{m}^{Exp} - 1}{\exp\left(-11.2 f_{l,f}^T / f_{c0}^T\right)}$ where $\overline{m}^{Exp}$ was determined using back analysis from the best fit of the experimental results of the axial stress-strain curve of test specimens with those obtained from the developed model).

## 5- Calculation Process

In the following, the calculation methodology of the proposed model, which can be implemented into a spreadsheet, for determining the axial response of FRP confined concrete submitted to a certain level of thermal damage is presented. The calculation procedure is as:

**1.** Determine the mechanical characteristics of unconfined heat-damaged concrete:

- Peak axial strength ($f_{c0}^T$) by Eq. (1)

- Axial strain corresponding to $f_{c0}^T$ ($\varepsilon_{c0}^T$) by Eq. (6)

**2.** Assume a value for $\varepsilon_c$ on the interval (0, $\varepsilon_{cu}$] where $\varepsilon_{cu}$ is the ultimate axial strain when FRP rupture occurs.

**3.** Determine the dilation response corresponding to $\varepsilon_c$

- Secant Poisson's ratio ($\nu_s^T$) by Eq. (29) and the data presented in Fig. 6

- Confinement pressure ( $f_{l,f}^T$ ) by Eq. (22)

**4.** Determine the axial response corresponding to $\varepsilon_c$

- Calibration factors $R_1$, $R_2$, $R_3$ and $R_4$, by Eqs. (45), (46), (49), and (50)

- $\overline{m}$ factor from Eq. (54)

- Peak axial strength ( $f_{cc}^T$ ) by Eq. (44)

- Axial strain ($\varepsilon_{cc}^T$) corresponding to $f_{cc}^T$ by Eq. (42)

- Axial stress ( $f_c^T$ ) by Eq. (39)

**5.** Continue the aforementioned incremental procedure up to $\varepsilon_{cu}$, $f_c^T$ versus $\varepsilon_c$ relation can be calculated.

Since the focus of the current study was given on the simulation of global axial stress-strain curves, the experimental values of ultimate axial strain ($\varepsilon_{cu}$) was used to terminate the computation process.

It is noteworthy that the reliability of regression analyses performed for developing predictive equations (as key components of the proposed model) is limited to the range of input/output variables supported in the used database. Accordingly, the predictive performance of these equations can be improved through recalibrating the model components based on new datasets consisting of the relevant variables with a broader range. Furthermore, the experimental data used for calibrating the failure surface function and the coupled dilation model was obtained from tests on relatively small FRP confined prototypes of square/circular concrete specimens where thermal distribution inside the concrete can be reasonably considered uniform. Therefore, for real cases with a larger dimension and non-uniform thermal distribution whose relative variables might not be in the aforementioned interval, the key components of the proposed model might need to be recalibrated, which will be the focus of a future study. Considering the relatively simple methodology of the proposed analytical-based model, it can be extended potentially to FRP confined heat-damaged RC columns of rectangular cross-section, through addressing properly the influences of dual FRP-steel confinement, sectional aspect ratio (the ratio of longer and shorter cross-section dimensions) and their interactions with exposure temperature in terms of axial and dilation behavior.

## 6- Verification

This section assesses the predictive performance of the developed analysis-oriented model for the prediction of axial response of concrete specimens submitted to a certain maximum exposure temperature, and after the specimens have attained the environmental temperature, subsequently, confined with FRP confining system. For this purpose, the axial stress-strain curve obtained from the proposed model is compared to that obtained from the experimental studies conducted by Bisby *et al.* [4], Lenwari *et al.* [10], Ouyang *et al.* [12] and Song *et al.* [13].

Bisby *et al.* [4] performed an experimental study to investigate the axial stress-strain response of FRP fully confined circular HC columns. The diameter and height of the test specimens were 100 mm and 200 mm, respectively. The axial compressive strength of AC column at the room temperature was 28 MPa. The values of nominal thickness, modulus of elasticity and rupture strain of CFRP jackets were reported as 0.12 mm, 241.1 GPa and 1.7%, respectively. All the specimens submitted to a certain level of maximum exposure temperature were confined by one CFRP layer. Complete details regarding the test specimens can be found from Bisby *et al.* [4]. Fig. 11 presents the comparison of the results obtained from the analytical model with those measured experimentally. As can be seen, the model has a good predictive performance in the estimation of axial responses of the test specimens submitted to 300 °C, 500 °C and 700 °C, even though there is a slight overestimation in terms of maximum axial strength for T500-L1.

Lenwari *et al.* [10] experimentally determined the axial stress-strain response of FRP fully confined circular HC columns. The diameter and height of the test specimens were 150 mm and 300 mm, respectively. The axial compressive strength of AC column at the room temperature was 40.5 MPa. The values of nominal thickness, modulus of elasticity and rupture strain of CFRP jackets were reported as 0.131 mm, 234.1 GPa and 1.8%, respectively. All the specimens submitted to a certain level of maximum exposure temperature were confined by one CFRP layer. Complete details regarding the test specimens can be found from Lenwari *et al.* [10]. Fig. 12 demonstrates the comparison of the results obtained from the analytical model with those measured experimentally. As can be seen, the model was able to predict the experimental counterparts with a good agreement, even though there is a slight overestimation in terms of maximum axial strength for T300-L1 and T500-L1.

For further investigation of the capability of the developed confinement model, the axial stress-strain curves obtained from experimental studies conducted by Ouyang *et al.* [12] and Song *et al.* [13], where the test specimens were submitted to a high level of thermal-induced damage (

$T_m \geq 600$ °C ), are compared with those analytically obtained from the model, as shown in Figs. 13 and 14. As can be observed, the model could accurately predict the full range of the experimentally measured axial responses, except for a slight underestimation for the cases of T600-L3 and T600-L3 tested by Song *et al.* [13] (Figs. 14e and f).

Due to the unified character of the proposed analysis-oriented model at ambient and elevated temperature conditions, Fig. 15 compares the axial stress- strain relations of FFSC-A reported from the experimental study conducted by Wang and Wu [6] with those analytically obtained from the model to assess its capability for the cases at ambient condition. All specimens had a section dimension of 150 mm and a height of 300 mm, with a sectional corner radius ratio ( $R_b$ ) varying from 0 ( $r = 0$ ) to 1 ( $r = 75$ mm ) representing a square cross section with sharp edges and circular cross-section, respectively. Two series of FFSC-A specimens with concrete strengths of $f_{c0} \simeq 30$ MPa and $f_{c0} \simeq 50$ MPa were tested. The CFRP thickness, tensile elastic modulus and rupture strain were 0.165 mm, 219 GPa, and 1.99%, obtained from flat coupon tensile tests. The complete details of the experimental program can be found in Wang and Wu [6]. As can be seen in Fig. 15, in general, there are a good agreement between the experimental axial stress-strain relationships with those obtained from the proposed model, confirming the successful simulation of the corner radius ratio ( $R_b$ ) influence on confinement-induced enhancements of FFSC-A at ambient conditions.

## 7- Summary and Conclusion

This paper has addressed the development of a new analysis-oriented model to predict the axial and dilation behavior of FRP confined HC circular/square concrete columns. Through regression analyses performed on a large database of unconfined heat-damaged concrete experimentally tested specimens, new expressions were developed to determine its mechanical characteristics in terms of axial compressive strength and its corresponding axial strain. A new model was developed to determine dilation response of FRP confined HC columns by formulating the effect of thermal damage level on Poisson's coefficient versus axial strain relationship. Subsequently, a new axial stress-strain model, coupled with the developed dilation model, was proposed to calculate the axial behavior of FRP confined HC columns with the different levels of maximum exposure temperature. Comparisons with axial and dilation results reported by available experiment studies in the literature verified that the developed analysis-oriented model is able to predict the experimental counterparts with good accuracy, and has a relatively simple format for design purposes by using a data sheet.

## Data Availability Statement

All data and models related to the present study could be available from the corresponding author upon rational request.


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# Stress–strain Model for FRP Confined Heat-damaged Concrete Columns

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## Abstract:

This paper is dedicated to the development of a new analysis-oriented model to simulate the axial and dilation behavior of FRP confined heat-damaged concrete columns under axial compressive loading. The model's calibration has considered the experimental results from concrete circular/square cross-section specimens submitted to a certain level of heat-induced damage, which after attained the environmental temperature, were fully confined with FRP jacket and tested. New equations were developed to determine the mechanical characteristics of unconfined heat-damaged concrete by performing regression analysis on a large database of experimental tests. Based on a parametric study on dilation behavior of FRP confined heat-damaged columns, a new dilation model was developed to predict concrete lateral strain at a given axial strain, dependent on the thermal damage level. By using this dilation model, a new methodology was introduced for predicting the axial stress-strain response of FRP confined heat-damaged columns in compliance with the active confinement approach. The adequate predictive performance of the model is demonstrated by estimating experimental axial stress-strain results.

**Keywords**: FRP confined heat-damaged concrete; thermal damage; confinement model; dilation behavior

## 1- Introduction

During fire, concrete buildings generally demonstrate a better fire performance compared to timber and steel buildings due to concrete non-combustibility and relatively low thermal conductivity (Kodur [1], Bamonte and Lo Monte [2]). Nonetheless, depending on the fire intensity imposed to the structural elements, material deteriorations occur during fire exposure, resulting detrimental effects on the performance of concrete structures at their serviceability and ultimate limit state conditions (Demir *et al.* [3]). Considering the pre-existing thermal-induced damage, a post-fire strengthening solution for restoring its structural performance can be an environmental and economic sustainable solution over the demolishing and rebuilding alternative. To reinstate sufficiently axial responses of heat-damaged concrete columns (HC), externally bonded fiber-reinforced-polymer (FRP) composites have been demonstrated as a viable solution (Bisby *et al.* [4]).

Several experimental and analytical studies (i.e. Barros and Ferreira [5], Wang and Wu [6], Janwaen *et al.* [7], Shayanfar *et al.* [8]) were conducted to evaluate the capability of FRP confining strategy in upgrading the axial and dilation behavior of FRP confined concrete columns under axial compression at ambient conditions. For the case of FRP fully confined concrete columns of circular cross section at ambient conditions (FFCC-A, as shown in Fig. 1a), Barros and Ferreira [5] experimentally evidenced that the confinement-induced enhancements for normal-strength concrete is more pronounced than those registered in high-strength concrete. For the case of FRP fully confined concrete columns of square cross section at ambient conditions (FFSC-A, as shown in Fig. 1a), Wang and Wu [6] and Shan *et al.* [9] conducted experimental studies to evaluate the influence of cross-section circularity in the effectiveness of the confining strategy. It was evidenced that decreasing the corner radius ($r$) from $r = b/2$ (circular columns where $b$ defines the length of the cross-section dimension) to $r$

= 0 (square columns with sharp edges) results in a significant reduction in terms of confinement efficiency.

On the other hand, very limited experimental studies have been conducted for assessing the strengthening efficiency of FRP confining systems applied in the post-cooling regime of concrete specimens subjected to a certain maximum exposure temperature according to the heating scheme demonstrated in Fig. 1b. Bisby *et al.* [4] performed an experimental research to assess the effectiveness of FRP fully confinement on circular concrete columns subjected to different levels of maximum temperature (300, 500 and 700 °C) (FFCC-H, as shown in Fig. 1a). It was evidenced that FRP confinement could significantly increase the axial compressive strength and stiffness of unconfined heat-damaged specimens (Fig. 1c). The peak axial compressive strength and the corresponding axial strain of FFCC-H specimens subjected to severe thermal exposure (700 °C) were almost 90% and 145%, respectively, of those of FFCC at ambient conditions. However, the secant axial stiffness of FFCC-H specimens (as the ratio of axial stress to its corresponding axial strain) was reported to be considerably lower than that of FFCC, and this difference has increased with maximum exposure temperature ($T_m$). Lenwari *et al.* [10] experimentally evidenced that the axial stress versus axial strain response of FFCC-H is dependent on the used heating scheme, i.e. $T_m$, exposure duration and cooling regime (air or water cooling methods), and on the axial compressive strength of AC, which was also confirmed by Luo *et al.* [11]. Furthermore, it was shown that the level of axial strength improvements, induced by FRP confining system, is higher in HC than in AC. Accordingly, for HC with a high level of thermal damage, FRP effectiveness tends to be more significant. Ouyang *et al.* [12] investigated experimentally the dilation behavior of heat-damaged circular concrete specimens confined by Basalt FRP (BFRP) jacket (FFCC-H) under axial compressive loading. It was demonstrated that there is a noticeable difference between the transverse expansibility of HC subjected to different level of the imposed $T_m$. Furthermore, the recorded hoop strains at BFRP rupture were almost independent of the exposure temperature. Song *et al.* [13] experimentally evidenced the high potential of BFRP confining system for improving the axial and dilation responses of HC with square cross-section (FFSC-H, as shown in Fig. 1a). The confinement-induced enhancements were more pronounced in HC exposed to high temperatures and confined by thicker BFRP jackets.

For the prediction of the axial stress-strain response of FRP confined AC columns, several analysis-oriented models, based on active confinement approach, have been recommended i.e. Teng *et al.* [14], Lim and Ozbakkaloglu [15], and Shayanfar *et al.* [16 and 17]. Teng *et al.* [14] proposed an analysis-oriented model for passively confined concrete, whose calibration was based on experimental observations in FRP fully confined concrete specimens of circular cross section (FFCC). Based on theoretical principles and experimental evidences, Shayanfar *et al.* [16] developed a generalized analysis-oriented model for passive confinement arrangements, whose parameters were derived from the experimental results with FRP fully/partially confined circular concrete specimens. Shayanfar *et al.* [17] extended Shayanfar *et al.* [16]'s model to make it applicable to the case of columns of square-cross section, by simulating the influence of the non-circularity in terms of confinement-induced enhancements. Bisby *et al.* [4] proposed a design-oriented model to determine the axial response of heat-damaged circular cross-section concrete columns confined by FRP (FFCC-H). In this model, the confinement-induced improvements were expressed as a main function of maximum exposure temperature ($T_m$) imposed to the column. Nevertheless, an analysis-oriented model to simulate the full-range of dilation and axial responses of FRP fully confined HC columns with square cross-section is still lacking.

This paper aims to introduce a new methodology to determine the axial stress-strain response of FRP fully confined circular/square HC columns (the concrete specimens were submitted to a certain $T_m$, and after having attained the environmental temperature, the fully confining FRP jacket was applied to the heat-damaged concrete). For the case of unconfined heat-damaged concrete, through regression analysis performed on a large test database, new expressions are developed to determine its mechanical characteristics in terms of axial compressive strength and its corresponding axial strain. By performing parametric studies with the available experimental data, the significant influence of the pre-existing thermal-induced damage on the establishment of the axial and dilation behavior of FRP fully confined heat-damaged concrete columns is demonstrated. By performing a parametric study to assess the influence of $T_m$ on the concrete transverse expansion, a new dilation model depending on the heat-damaged level is proposed, which has a unified character with the dilation model developed by Shayanfar *et al.* [17] for concrete specimens at room temperature. By using the developed dilation model, a new methodology is introduced, based on active confinement approach, for the simulation of the axial stress-strain response of FRP fully confined circular/square HC columns at the different levels of $T_m$. The model's adequate predictive performance is demonstrated by estimating experimental axial stress-strain responses.

## 2- Unconfined heat-damaged concrete columns (HC)

Kodur [1] evidenced that post-fire response of HC columns significantly depends on concrete mechanical, thermal (including thermal conductivity, thermal diffusivity, specific heat, and mass loss) and deformation (including concrete thermal expansion) properties, as well as on the spalling response. In general, concrete submitted to maximum exposure temperature ($T_m$) up to 100 °C can be considered almost undamaged. Afterward, water loss (causing shrinkage) and the expansion of aggregates induce internal stresses in the concrete, particularly for $T_m \geq 300$ °C. Furthermore, thermal-induced chemical processes and thermo-mechanical damages lead to a significant strength degradation for the concrete submitted to high levels of exposure temperature [1]. A comprehensive review of concrete properties at elevated temperatures can be found in Kodur and Sultan [18], Hertz [19], Raut and Kodur [20], Aslani and Bastami [21].

In terms of mechanical properties, by submitting concrete to elevated temperature with a heating scheme including maximum exposure temperature ($T_m$), the axial compressive stress of HC ($f_c^T$) and the modulus of elasticity ($E_{cT}$) decrease depending upon its peak axial strength ($f_{c0}^T$). However, axial strains corresponding to the peak ($\varepsilon_{c0}^T$) and ultimate stages ($\varepsilon_{cu0}^T$) increase, demonstrating a significant reduction on the axial stiffness (defined as $f_c^T / \varepsilon_c$ where $\varepsilon_c$ is the axial strain) of HC compared to AC (Hertz [19], Chang *et al.* [22], Al-Salloum *et al.* [23], Sharma *et al.* [24], Geng *et al.* [25], Xiao *et al.* [26], Xiang *et al.* [27]).

**2-1- Peak axial strength of HC ($f_{c0}^T$)**

Experimental studies evidenced that the heat-induced damages in HC columns lead to a reduction in terms of peak axial strength. Accordingly, by defining the axial strength ratio, $\beta_{0T}$, (the ratio of $f_{c0}^T$ and $f_{c0}$), $f_{c0}^T$ can be expressed as:

$$f_{c0}^T = \beta_{0T} f_{c0} \tag{1}$$

The variation of $\beta_{0T}{}^{Exp} = f_{c0}^{T\ Exp} / f_{c0}{}^{Exp}$ with respect to $T_m$ can be obtained from experimental results of axial compression tests on HC columns. In the present study, a database of 292 HC column specimens with a wide range of concrete properties and exposure temperature was collected, as briefly presented in Table 1. The following criteria were adopted to include/exclude experimental data: 1) ones obtained from circular/square/rectangular heat-damaged concrete specimens tested under concentric were included; 2) heat-damaged concrete specimens subjected to a maximum exposure temperature more than 800 °C were excluded; 3) light-weight concrete specimens were excluded; 4) light-weight concrete, and recycled aggregate concrete specimens were excluded; 5) data registered experimentally with incomplete documented information, i.e. maximum exposure temperature, geometry details and material properties, were excluded.

Fig. 2a shows the variation of $\beta_{0T}{}^{Exp}$ with respect to $T_m$, based on the results of the database, where a decrease of $\beta_{0T}{}^{Exp}$ with the increase of $T_m$, from 1 to almost 0.2 corresponding to room conditions and 800 °C, respectively, is visible. Based on the best-fit relation obtained from regression analysis on the database information, $\beta_{0T}$ versus $T_m$ data can be obtained as $\beta_{0T} = 1.087 - 0.00116T_m$. By assuming *Error Index* as $(1.087 - 0.00116T_m)/\beta_{0T}{}^{Exp}$, Fig. 2b demonstrates that there is a slight variation in Error Index versus concrete strength relationship which is less than one up to almost $f_{c0}$ = 110 MPa, representing underestimation. However, beyond $f_{c0}$ = 110 MPa, the Error Index tend to be more than 1 resulting in overestimation of the experimental counterparts. Accordingly, based on regression analysis performed on the Error Index and concrete strength relationship, $\beta_{0T}$ can be calculated by:

$$\beta_{0T} = \frac{1.087 - 0.00116T_m}{\gamma_f} \leq 1 \qquad (2)$$

in which

$$\gamma_f = 1 + (\gamma_0 - 1)\left(\frac{T_m - 25}{100}\right) \qquad \text{for } T_m \leq 100\ °\text{C} \qquad (3a)$$

$$\gamma_f = \gamma_0 \qquad \text{for } T_m \geq 100\ °\text{C} \qquad (3b)$$

$$\gamma_0 = 3415\left(\frac{f_{c0}}{1000}\right)^3 - 721\left(\frac{f_{c0}}{1000}\right)^2 + 44.5\left(\frac{f_{c0}}{1000}\right) + 0.178 \qquad (4)$$

where the developed expression is valid for $T_m \leq 800$ °C based on the interval of the submitted maximum exposure temperatures ($T_m = [25$ °C , $800$ °C$]$) in the database used for the regression analysis. $\gamma_0$ reflects the influence of $f_{c0}$ (in MPa) in the determination of $\beta_{0T}$ empirically.

In Fig. 2c, the results predicted by the proposed model are compared to those reported by the experiments (Table 2), with a mean = 0.964, a coefficient of variation (CoV) = 0.279, a mean absolute percentage error (MAPE) = 0.203, and an R-squared value ($R^2$) = 0.876, revealing an acceptable predictive performance. Table 2 also shows that the proposed model provides a predictive performance better than of the existing models.

**2-2- Axial strain at the peak stage of AC columns ($\varepsilon_{c0}$)**

For the case of AC columns, the experimental and analytical studies conducted by [33-38] evidenced that $\varepsilon_{c0}$ increases with the concrete compressive strength ($f_{c0}$). Jansen and Shah [34] experimentally demonstrated that the column aspect ratio ($\lambda_L$ as the ratio of the column height to its diameter) has also considerable influence on $\varepsilon_{c0}$ due to the occurrence of strain-localization within a finite zone with a pronounced gradient of deformations due to the concrete post-peak strain-softening behavior. In this study, in order to estimate $\varepsilon_{c0}$, a large database (Table 3) was compiled from the experimental results available in the literature, resulting in 604 unconfined concrete specimens (AC) with a broad range of concrete properties and geometry configurations. Note that for the case of non-circular columns with a total cross-section area of $A_g$, based on Yang *et al.* [38] recommendations, $\lambda_L$ can be determined as $L/d_{eq}$

where $L$ is the columns' height and $d_{eq}$ is the equivalent circular diameter ($d_{eq} = \sqrt{4A_g/\pi}$ [38]).

Based on a preliminary sensitivity analysis, a low effect was achieved for the influence of the column size (i.e. the normalized $d_{eq}/150$, with $d_{eq}$ in mm) to estimate $\varepsilon_{c0}$ when compared to other influencing factors ($f_{c0}$ and $\lambda_L$). Accordingly, using regression analysis, the best-fit expression to predict $\varepsilon_{c0}$ was derived as a function of $f_{c0}$ and $\lambda_L$ regardless of the column's cross-section dimension influence:

$$\varepsilon_{c0} = 0.0011\left(\frac{f_{c0}}{\lambda_L}\right)^{0.25} \tag{5}$$

Table 4 evaluates the predictive performance of this relation with the results of the experimental tests, and also compares with that of existing models. Based on the assessment indicators (values of mean = 0.977, CoV = 0.184 and MAPE = 0.138), there is an acceptable agreement between model prediction and the experimental results. Furthermore, compared to the models recommended by Popovics [39], Karthik and Mander [40] and Lim and Ozbakkaloglu [41], it is the most accurate one, confirming its reliability.

## 2-3- Axial strain at the peak stage of HC columns ($\varepsilon_{c0}^T$)

The experiments with HC columns conducted by Chang *et al.* [22], Sharma *et al.* [24], Xiao *et al.* [26], Xiang *et al.* [27] evidenced that the axial strain ($\varepsilon_{c0}^T$) corresponding to $f_{c0}^T$ tends to increase significantly from $\varepsilon_{c0}$ at ambient condition to $\varepsilon_{c0}^T \gg \varepsilon_{c0}$ at elevated temperature, as shown in Fig. 3a.

The details of the experimental specimens in the assembled database, including 225 tested HC columns, is presented in Table 5.

The best-fit relation between $\varepsilon_{c0}^{T\ Exp}/\varepsilon_{c0}-1$ (representing the thermal damage-induced strain) and $T_m$ was obtained from regression analysis by considering the influence of $f_{c0}$, resulting:

$$\varepsilon_{c0}^{T}=\left(1+63f_{c0}^{\ -0.5}\left(\frac{T_m}{1000}\right)^{4.2}\right)\frac{\varepsilon_{c0}}{\alpha_T}\leq 4.5\frac{\varepsilon_{c0}}{\alpha_T} \tag{6}$$

in which

$$\alpha_T=1 \qquad \text{for } T_m\leq 100\ °\text{C} \tag{7a}$$

$$\alpha_T=1.22-0.0025T_m+3\times10^{-6}T_m^{\ 2} \qquad \text{for } T_m>100\ °\text{C} \tag{7b}$$

where the developed expression is valid for $T_m\leq 800$ °C based on the interval of the submitted maximum exposure temperatures ($T_m=\left[25\ °\text{C}\ ,\ 800\ °\text{C}\right]$) in the database used for the regression analysis. $\alpha_T$ is the calibration factor for the influence of $T_m$ in the increase of axial strain induced by thermal damage, obtained from the regression analysis. In Fig. 3b and Table 6, the predictive performance of this model is assessed based on 225 test specimen results. As can be seen, Eq. (6) provides the most accurate model compared to the existing models in the prediction of the experimental counterparts, even though conservative results were achieved for some cases submitted to high level of exposure temperature.

## 3-Dilation behavior of FRP confined HC columns

### 3-1- Confinement pressure developed for ambient condition

For the case of FRP fully confined square AC columns (FFSC), based on the force equilibrium at the cross-sectional level, confinement pressure ($f_{l,f}$) generated by the FRP confining stress ($f_f$) can be expressed as (Shayanfar *et al.* [17]):

$$f_{l,f} = 2K_e \frac{n_f t_f}{D_{eq}} f_f \tag{8}$$

where $n_f$ is the number of FRP layers; and $t_f$ is the nominal thickness of one FRP layer. In Eq. (8), $D_{eq}$ defines the diameter of the equivalent circular cross-section for columns of square cross section with *b* edge and *r* corner radius, which can be calculated as recommended by Shayanfar *et al.* [17]:

$$D_{eq} = \frac{1-0.215R_b{}^2}{1-0.215R_b} b \tag{a9}$$

where

$$R_b = 2r/b \tag{b9}$$

is the corner radius ratio. Note that by using $D_{eq}$ in the determination of FRP confinement pressure, FRP volumetric ratio in the equivalent circular cross-section would be identical to that of original square cross-section column. In Eq. (8), $K_e$ is the confinement efficiency factor. Shayanfar *et al.* [17] modified the original concept of '*confinement efficiency factor*' by considering the impact of concrete expansion gradient in the establishment of confinement pressure, besides the well-known phenomenon of arching action. By using this concept, the actual confinement pressure acting non-homogenously on the concrete is converted to an equivalent confinement pressure with uniform distribution along transverse and longitudinal directions of the column. This factor includes two components, which can be determined as suggested by Shayanfar *et al.* [17]:

$$K_e = K_H K_V \tag{10}$$

where $K_H$ is the horizontal component, reflecting the influence of horizontal arching action on the distribution of confinement pressure within the cross-section of a non-circular columns (for circular columns, $K_H = 1$), determined as:

$$K_H = R_b \geq 0.07 \tag{11}$$

In Eq. (10), $K_V$ is the vertical component reflecting the influence of the gradient of concrete lateral expansion along the column height, depending on the level of confinement stiffness (the ratio of confinement pressure to concrete lateral strain). It was demonstrated by Shayanfar *et al.* [16] that above a certain level of confinement stiffness, the confinement imposed to the concrete is strong enough to strictly control the evolution of concrete expansion leading to an almost null gradient along the vertical direction ($K_V = 1$). However, for the cases with an insufficient confinement stiffness, due to the lack of strong restriction in the curtailment of concrete expansibility, the concrete column is expected to experience a highly non-homogenous distribution of concrete expansion and, consequently, the confinement pressure in the axial loading direction is non-uniform. Shayanfar *et al.* [17] suggested a design-based formulation to calculate $K_V$ as follows:

$$K_V = \frac{1}{3} + \frac{2}{3} k_\varepsilon \tag{12}$$

in which

$$k_\varepsilon = 0.08 + 0.92\left[2\frac{I_f}{I_f^*} - \left(\frac{I_f}{I_f^*}\right)^2\right]^2 \leq 1 \qquad \text{for } I_f \leq I_f^* \tag{13a}$$

$$k_\varepsilon = 1 \qquad \text{for } I_f > I_f^* \tag{13b}$$

$$I_f^* = 0.06 + 0.0005 f_{c0} \tag{14}$$

$$I_f = 2K_H \frac{n_f t_f E_f \varepsilon_{c0}}{D_{eq} f_{c0}} \simeq K_H \frac{n_f t_f E_f}{550 D_{eq} f_{c0}^{0.75}} \tag{15}$$

where $k_\varepsilon$ represents the ratio of minimum and maximum concrete lateral expansion along the column height. $I_f$ represents the confinement stiffness index regardless the influence of the gradient of concrete expansion along the column height. Finally, $I_f^*$ is the confinement stiffness index above which $K_V = 1$, representing the homogenous concrete expansion along the column due to strong restrictions imposed to the concrete.

In this paper, for further simplification of the relative complexity of Eqs. (12-15) in the calculation of $K_V$, a simplified equation was developed based on a preliminary sensitivity analysis on the influencing factors in Eq. (12) as:

$$K_V = 2.2\left(I_f\right)^{0.3} \leq 1 \tag{16}$$

Accordingly, by using the design-based Eqs. (11, 15 and 16), the two components involved in $K_e$ (Eq. (10)) can be calculated.

**3-2- Confinement pressure developed for elevated condition**

Based on the model developed for ambient conditions in the previous section, the confinement pressure ( $f_{l,f}^T$ ) imposed by the FRP confining stress ( $f_f$ ) to HC column can be expressed as:

$$f_{l,f}^T = 2K_e^T \frac{n_f t_f}{D_{eq}} f_f^T \tag{17}$$

in which

$$K_e^T = K_H K_V^T \tag{18}$$

where $K_H$ can be determined by Eq. (11). Through the substitution of confinement stiffness index of FRP confined HC columns ($I_f^T$) with that of FRP confined AC ones ($I_f$) in Eq. (16), $K_V^T$ can be expressed as:

$$K_V^T = 2.2\left(I_f^T\right)^{0.3} = 2.2\beta_{0T}{}^{-0.45}\left(I_f\right)^{0.3} \leq 1 \tag{19}$$

in which based on Eq. (15),

$$I_f^T = K_H \frac{n_f t_f E_f}{550 D_{eq}\left(f_{c0}^T\right)^{0.75}} = \beta_{0T}{}^{-0.75} I_f \tag{20}$$

where $\beta_{0T}$ is the axial strength ratio calculated by Eq. (2), which is equal to 1 for concrete at ambient condition; Considering that $f_f^T$ can be calculated as $E_f \varepsilon_h^T = E_f \varepsilon_l^T$ (where $\varepsilon_h^T$ and $\varepsilon_l^T$ are the generated circumferential (hoop) and radial strains, respectively, and $E_f$ is the FRP modulus elasticity), Eq. (17) is rearranged as:

$$f_{l,f}^T = 2K_e^T \frac{n_f t_f}{D_{eq}} E_f \varepsilon_h{}^T \tag{21}$$

Based on Poisson's ratio effect ($\varepsilon_h^T = \varepsilon_l^T = v_s^T \varepsilon_c$, where $v_s^T$ is the secant Poisson's ratio), Eq. (21) can be rearranged as:

$$f_{l,f}{}^T = 2K_e \frac{n_f t_f}{D_{eq}} E_f v_s{}^T \varepsilon_c \tag{22}$$

Accordingly, in order to calculate $f_{l,f}^T$ imposed to the concrete at a certain level of $\varepsilon_c$, the corresponding $v_s^T$ is required to be addressed, which will be presented in the following section.

### 3-3-Dilation mechanism at elevated conditions

During axial compressive loading, after splitting cracks have occurred, by increasing the axial strain, the development of concrete lateral expansion abruptly increases due to Poisson's ratio effect. Experimental studies conducted by Barros and Ferreira [5], Mirmiran and Shahawy [46], Lim and Ozbakkaloglu [47] and Zeng *et al.* [48] evidenced that the magnitude of concrete dilatancy is strongly dependent on confinement stiffness imposed to the concrete. For the case of AC with a high level of confinement stiffness capable of limiting the evolution of concrete expansion and splitting cracks, a remarkable enhancement in terms of axial strength and deformability is obtained (Barros and Ferreira [5]). However, for the case of low confinement stiffness, the confinement pressure imposed to the concrete is not able to overcome the concrete tendency for abrupt expansion, leading to lower confinement-induced enhancements [49].

For a preliminary assessment of the dilation response of FRP confined HC columns, the experimental dilation results conducted by Ouyang *et al.* [12] are analyzed. All tests were conducted with specimens of diameter and height of 150 mm and 300 mm, respectively. The unconfined concrete compressive strength at the ambient condition was reported 45.1 MPa. Basalt FRP (BFRP) was used with the values of thickness, modulus of elasticity and rupture strain of 0.121 mm, 108.3 GPa and 2.18%, respectively. The HC columns were subjected initially to various levels of maximum temperature (200 °C, 400 °C, 600 °C and 800 °C). Then, they were fully confined with two and four layers of BFRP. Fig. 4 demonstrates the experimental dilation responses of BFRP confined HC specimens reported by Ouyang *et al.* [12]. Here, $\varepsilon_V$ is the volumetric strain determined as $\varepsilon_V = \varepsilon_c - 2\varepsilon_l = \left(1-2v_s\right)\varepsilon_c$ in which $v_s$ is the secant Poisson's ratio as $v_s = \varepsilon_l / \varepsilon_c$. Moreover, the negative and positive values of $\varepsilon_V$ represent volumetric expansion and contraction, respectively. To better demonstrate the contribution of thermal-induced damage level in terms of dilation behavior, the model developed by Shayanfar *et al.* [8 and 17] was followed to determine the dilation results associated with FRP confined AC specimens (***T25-L2*** and ***T25-L4*** with red solid lines). Note that Ti-Lj refers to the concrete column heated up to the i-th maximum exposure temperature (Ti) and then, confined by j layers of BFRP.

As can be seen in Fig. 4, for all cases, regardless the level of thermal-induced damage, initial behavior up to transition zone is virtually the same. However, beyond the transition zone, there is a noticeable difference between the transverse expansibility of HC and AC specimens. Fig. 4a reveals that at a certain axial strain ($\varepsilon_c$), lateral strain ($\varepsilon_l$) for the cases of T200-L2 and T400-L2 was obtained significantly higher than that of T25-L2, demonstrating the effect of thermal damage on increase of $\varepsilon_l$. Likewise, from Fig. 4b, T200-L2 and T400-L2 have experienced a large incremental volumetric expansion. However, in the T25-L2, beyond $\varepsilon_c = 0.008$, a considerable decrease in the magnitude of the increase in volumetric strain ($\varepsilon_v$) with respect to $\varepsilon_c$, followed by a reverse in volumetric evolution around $\varepsilon_c$=0.02, reveals the capability of the confinement system imposed to AC column (T25-L2) in limiting the transverse expansibility of AC columns. Likewise, as demonstrated in Fig. 4c, for T25-L2, due to the adequate activated confinement imposed to AC column to overcome its tendency for lateral expansibility, beyond the peak stage, $v_s$ trend followed a decreasing branch. Even though, heat-induced expansion for HC columns leads to an earlier activation in passive confining system of T200-L2 and T400-L2, the applied confinement was not adequate to strongly constrain the concrete expansion evolution, based on its abrupt increase in $v_s$ after transition zone. On the other hand, for the cases subjected to high level of temperature (T600-L2 and T800-L2), as shown in Fig. 4a, at a certain level of $\varepsilon_l$, T600-L2 and T800-L2 experienced a larger axial deformation depending on thermal damage level, compared to T25-L2, T200-L2 and T400-L2. Likewise, Fig. 4b reveals that up to a certain level of axial strain, the changes in volumetric evolution for T600-L2 and T800-L2 were almost marginal, while they underwent large axial deformations. Nonetheless, above this axial strain level, as a consequence of the degeneration of micro- into meso- and macro-cracks along with heat-induced damage in the concrete, the volumetric change evolution was suddenly reversed triggering an abrupt increase in volumetric expansion. Fig. 4c also shows that for T600-L2 and T800-L2, compared to the other cases, larger axial strains were obtained for a certain Poisson's ratio. Furthermore, a closer evaluation of the data demonstrates that the maximum secant Poisson's ratio decreases significantly with increasing thermal damage, which can be attributed to the substantial contribution of the heat-induced damage level in the establishment of dilation behavior of HC. Accordingly, by applying a certain level of axial loading, the heat-induced damage leads to an additional axial strain in HC columns, and alters their transverse expansibility, dependent strongly on thermal damage level. The comparison of dilation responses shown in Fig. 4a and Fig. 4d confirms a significant reduction in terms of lateral strain by increasing FRP thickness (confinement stiffness), predominantly beyond the transition zone. Fig. 4e reveals that an increase in confinement stiffness leads to shorter volumetric expansion due to the stronger restrictions imposed to the concrete expansibility. The relations of $v_s$ and $\varepsilon_c$ shown in Fig. 4f also confirms this behavior, where the specimens with more FRP thickness experienced a lower value of $v_s$ than those with less thickness (Fig. 4c).

In this study, it is aimed to extend the dilation model of Shayanfar *et al.* [8,17] originally suggested for FRP confined AC specimens to the case of FRP confined HC column through formulating the relation between $v_s^T$ and $\varepsilon_c$ at different levels of heat-induced damage ($T_m$) based on regression analysis. For this purpose, a dataset of the dilation responses obtained from existing experimental data was collected as presented by Table 7. It should be noted that the following criteria were adopted to include/exclude experimental data: 1) ones obtained from concentric loading tests were included; 2) Specimens with a full confinement of unidirectional fibers installed transversely to the axial compression direction were included; 3) Specimens with helical wrapping configuration or hybrid confinement arrangements were excluded; 4) specimens of rectangular cross-section with transverse and longitudinal steel reinforcements were excluded; 5) ones registered experimentally with incomplete documented information, i.e. maximum exposure temperature, geometry details and material properties, were excluded; 6) ones obtained from test specimens with a premature failure mode caused by FRP debonding were excluded. In this table, $v_{s,\max}^{T}$ represents the peak Poisson's ratio of HC columns confined by FRP. $\eta_T$ defines the ratio of $v_{s,\max}^{T}$ to $v_{s,\max}^{A}$ (where $v_{s,\max}^{A}$ is the peak Poisson's ratio of FRP confined AC columns at ambient temperature). For the calculation of $v_{s,\max}^{A}$, in the present study, the confinement stiffness-based model developed by Shayanfar *et al.* [17] was followed, which is determined as a main function of the confinement stiffness ($\rho_{K,f}$), namely:

$$v_{s,\max}^{A} = \frac{0.25}{\left(1 + L_{d0}/D_{eq}\right)\sqrt{\rho_{K,f}}} \tag{23}$$

in which (by considering Eqs. (10), (15) and (16))

$$\rho_{K,f} = K_V I_f = K_e \frac{n_f t_f E_f}{550 D_{eq} f_{c0}^{\,0.75}} \tag{24}$$

$$0.57 \le \frac{L_{d0}}{\sqrt{A_g \psi_f}} = 1.71 - 3.53 \times 10^{-5} A_g \le 1.36 \tag{25}$$

$$\psi_f = \frac{6.3}{\sqrt{f_{c0}}} \le 1 \tag{26}$$

where $L_{d0}$ is the compression damage zone length of unconfined concrete columns which was determined as recommended by Lertsrisakulrat *et al.* [50]; $A_g$ is the total area of the column's cross section.

For the determination of $v_{s,\max}^{T}$, it can be considered as a function of $v_{s,\max}^{A}$ by

$$v_{s,\max}^{T} = \frac{v_{s,\max}^{T}}{v_{s,\max}^{A}} v_{s,\max}^{A} = \eta_T v_{s,\max}^{A} \tag{27}$$

Therefore, by calculating $v_{s,\max}^{A}$ using Eq. (23) for the specimens assembled in the database, their corresponding values of $\eta_T$ can be obtained as $\eta_T^{Exp} = v_{s,\max}^{T\ \mathrm{Exp}} / v_{s,\max}^{A}$ (Table 7). Fig. 5a shows the variation of $\eta_T^{Exp}$ with respect to $T_m$. As can be seen, by increasing $T_m$ in the interval $25-400$ °C, $\eta_T^{Exp}$ significantly increases up to the peak, while for $T_m > 400$ °C, a noticeable reduction of $\eta_T^{Exp}$ with the increase of $T_m$ is observed. Based on the best-fit relation obtained from regression analysis performed on 78 experimental data, the following equation was derived for determining $\eta_T$ from $T_m$ and $R_b$ (Fig. 5a):

$$\eta_T = \frac{33.2\left(\frac{T_m}{1000}\right)^3 - 51\left(\frac{T_m}{1000}\right)^2 + 21.2\left(\frac{T_m}{1000}\right) - 0.49}{1.65 - 0.65R_b} \geq 1 \qquad \text{for } T_m \leq 100 \text{ °C} \tag{28a}$$

$$\eta_T = \frac{33.2\left(\frac{T_m}{1000}\right)^3 - 51\left(\frac{T_m}{1000}\right)^2 + 21.2\left(\frac{T_m}{1000}\right) - 0.49}{1.65 - 0.65R_b} \leq 2 \qquad \text{for } 100 \text{ °C} < T_m \leq 800 \text{ °C} \tag{28b}$$

where the developed expression is valid for $T_m \leq 800$ °C based on the interval of the submitted maximum exposure temperatures ($T_m = [25 \text{ °C}, 800 \text{ °C}]$) in the database used for the regression analysis. Therefore, by addressing $v_{s,\max}^{A}$ and $\eta_T$ in Eq. (27) through Eqs. (23, 28), the peak Poisson's ratio of FRP confined HC columns ($v_{s,\max}^{T}$) can be calculated. Fig. 5b evaluates the performance of Eq. (27). Based on the obtained assessment indicators, the established expression revealed a good predictive performance.

For the determination of Poisson's ratio of HC columns confined by FRP ($v_s^T$) during axial loading, $v_s^T$ is considered as a function of $v_{s,\max}^{T}$, resulting in

$$v_s^T = \frac{v_s^T}{v_{s,\max}^{T}} v_{s,\max}^{T} = \eta_\varepsilon v_{s,\max}^{T} \tag{29}$$

where $\eta_\varepsilon$ represents the ratio of $v_s^T$ and $v_{s,\max}^{T}$ at a given axial strain ($\varepsilon_c$). Introducing Eq. (27) into Eq. (29), $v_s^T$ is suggested by

$$v_s^T = \eta_\varepsilon \eta_T v_{s,\max}^{A} \tag{30}$$

To calculate $v_s^T$ by Eq. (30), $\eta_\varepsilon$ needs to be addressed as an input parameter. For the determination of $\eta_\varepsilon$ at a given level of $\varepsilon_c$ for FRP confined AC columns (at ambient condition), Shayanfar *et al.* [8] proposed a simple but reliable multilinear model as shown in Fig. 6a. in this figure, $c_0$ $c_1$, $c_2$, $c_3$, $c_4$ and $c_5$ are the calibration factors reflecting the influence of confinement stiffness on $\eta_\varepsilon$ versus $\varepsilon_c$ relation; $\varepsilon_{c,m}$ represents the axial strain corresponding to $v_{s,\max}^{A}$; $v_{s,0}$ is the initial Poisson's coefficient of unconfined AC columns, which was determined by (Candappa *et al.* [51]):

$$v_{s,0} = 8\times10^{-6} f_{c0}^{\ 2} + 2\times10^{-4} f_{c0} + 0.138 \tag{31}$$

As can be seen in Fig. 6a, the pre- and post-peak phases are dependent of $\rho_{K,f}$. The pre-peak relation demonstrates that concrete initially behaves similar to unconfined AC column with initial Poisson's coefficient as $v_{s,0}$. Afterward ($\varepsilon_c > \varepsilon_{c0}$), $\eta_\varepsilon$ increases up to the peak stage ($\eta_\varepsilon = 1$) at $\varepsilon_{c,m}$. The post-peak relation shows that $\eta_\varepsilon$ decreases with the increase of $\varepsilon_c$, whose reduction magnitude is dependent on $\rho_{K,f}$. In the present study, the relation developed by Shayanfar *et al.* [8] was extended for the case of FRP confined HC columns. For this purpose, as presented in Fig. 6b, a new relation of $\eta_\varepsilon$ versus $\varepsilon_c$ was developed where the influence of thermal damage was reflected by the parameter $\beta_\varepsilon$.

$$\beta_\varepsilon = \beta_\rho \left( \varepsilon_{c0}^T - \varepsilon_{c0} \right) \tag{32}$$

in which

$$0.4 \leq \beta_\rho = 11 \rho_{K,f}{}^{0.75} \leq 1.4 \tag{33}$$

Accordingly, by calculating $\beta_\varepsilon$ through Eq. (32), $\eta_\varepsilon$ at a given $\varepsilon_c$ can be obtained from the data presented in Fig 6b. Then, the corresponding $v_s^T$ can be calculated by Eq. (30). To evaluate the reliability of the proposed relation, the results obtained from the experiments conducted by Ouyang *et al.* [12] and those determined by Eq. (30) are compared Fig. 7. As can be seen, the model has a good agreement with the experimental counterparts.

To obtain $\varepsilon_l^T$ versus $\varepsilon_c$ response of FFCC-H/FFSC-H based on the developed dilation model, the calculation procedure is summarized as:

**1.** Determine $K_e$ using Eqs. (10), (11) and (16)

**2.** Determine $\rho_{K,f}$ using Eq. (24)

**3.** Determine $v_{s,\max}^A$ using Eqs. (23)

**4.** Determine $\eta_T$ using Eq. (28)

**5.** Assume a value for axial strain ($\varepsilon_c$)

**6.** Determine $\beta_\varepsilon$ using Eqs. (32), (5) to (7)

**7.** Determine $\eta_\varepsilon$ using the developed multilinear model in Fig. 6b

**8.** Determine $v_s^T$ using Eq. (29)

**9.** Determine $\varepsilon_l^T$ as $\varepsilon_l^T = v_s^T \varepsilon_c$

**10.** Continue the steps 5 to 9 up to the aim maximum value of $\varepsilon_c$, resulting a $\varepsilon_l^T$ versus $\varepsilon_c$ relation.

For the assessment of the model capability in the prediction of dilation response, Fig. 8 compares $\varepsilon_l^T$ versus $\varepsilon_c$ relations extracted from the experimental tests conducted by Ouyang *et al.* [12] and those simulated by the proposed model. As shown, the experimental results were simulated suitably by the model confirming its reliable predictive performance.

## 4- Axial Compressive Stress-strain Relation

Fig. 9 demonstrates the remarkable influence of the exposure temperature on the axial response of HC columns fully confined by FRP, tested by Bisby *et al.* [4] and Ouyang *et al.* [12]. As can be seen in Fig. 9a for the case of the results reported by Bisby *et al.* [4], compared to T25-L1 (control experimental specimen), even though T300-L1 experienced a slight heat-induced reduction in terms of initial axial stiffness, it showed higher axial strength and deformability. For T500-L1, heat- induced damage did not lead to a significant difference in the axial stress versus axial strain relationship. However, for the case of severely heat-damaged specimens (T686-L1), there is a noticeable reduction in its axial stiffness, compared to T25-L1. Moreover, the axial stress versus axial strain relationship of T686-L1 was almost linear, compared to almost bi-linear curves of specimens with the exposure temperatures lower than or equal to 500 °C. From the secant axial stiffness versus axial strain curves in Fig. 9b, T25-L1, T300-L1 and

T500-L1 presented a reduction of the axial stiffness during the axial compressive loading. However, in T686-L1, after an initial relatively small reduction of the stiffness, it was preserved almost constant during the loading process, a consequence of the confinement effect provided by the FRP. The assessment of the exposure temperature effects on the test specimens conducted by Ouyang *et al.* [12] revealed that the axial strength and stiffness of specimens subjected up to 400 °C were higher than of T25-L2. It can be attributed to earlier activation of confining system in the cases of HC than AC due to the increase of concrete expansibility caused by the heat-induced damage. However, by increasing the temperature above that limit of 400 °C, thermal-induced damage increases significantly the axial deformation of the concrete specimen, converting a bi-linear stress-strain response into a linear one (Fig. 9c), and consequently an almost constant axial stiffness during axial loading process (Fig. 9d). Accordingly, the experimental results presented in Fig. 9 evidence the imperious impact of exposure maximum temperatures on the axial stress-strain response of FRP confined HC columns.

In order to calculate the axial stress versus axial strain relationship ( $f_c{}^A$ *vs* $\varepsilon_c$ curve) of FRP passively confined AC columns (passively-confined-concrete), *Active Confinement Approach* (i.e. [14-17, 52-54] can be followed. In this approach, the axial response of concrete with passive confinement is derived based on that of actively-confined-concrete subjected to a constant confinement pressure during the entire axial loading history. Accordingly, $f_c{}^A$ corresponding to $\varepsilon_c$ at a certain level of FRP confinement pressure ( $f_{l,f}{}^A$ ) can be calculated by adopting an axial stress-strain base relation model ( $f_c{}^A = g_1\left(f_{cc}{}^A, \varepsilon_c\right)$ ) coupled with an axial strength model, also known as failure surface function, ( $f_{cc}{}^A = g_2\left(f_{l,f}{}^A\right)$ ) developed for actively-confined-concrete. Here, $f_{cc}{}^A$ is the peak axial strength of axial stress-strain base relation model (defined from function $g_1$). However, since actively-confined-concrete case is under a constant $f_{l,f}{}^{A}$ during the entire axial loading history, contrary to passively-confined-concrete, the studies [15-17] evidenced that the original *Active Confinement Approach* overestimates the FRP-induced enhancement of passively-confined-concrete, which is generally recognized as *Confinement Path Effect*. To consider this effect, by adopting functions $g_1$ and $g_2$ from those exclusively developed for actively-confined-concrete, Lim and Ozbakkaloglu [15] recommended a reduction factor in the confinement-induced enhancements obtained for actively-confined-concrete, by decreasing the level of the confinement pressure $f_{l,f}{}^{A}$ in the function $g_2$. Shayanfar *et al.* [16 and 17] proposed a new axial strength framework model (function $g_2$) exclusively suggested for passively-confined-concrete to predict enhancements offered by a passive confining system.

In the present paper, for the calculation of $f_c{}^{A}$ and $\varepsilon_c$ relation of FRP confined AC columns taking into account *confinement path effect*, the model developed by Shayanfar *et al.* [17] presenting a unified character for both full and partial confinement configurations and both circular and square cross-sections, is followed. In this model, the axial stress-strain base framework ( $f_c{}^{A}$ versus $\varepsilon_c$ relation using function $g_1$) is given by:

$$f_c{}^{A} = f_{cc}{}^{A} \frac{\left(\varepsilon_c / \varepsilon_{cc}{}^{A}\right) n_A}{n_A - 1 + \left(\varepsilon_c / \varepsilon_{cc}{}^{A}\right)^{n_A}} \tag{34}$$

in which

$$\frac{f_{cc}{}^{A}}{f_{c0}} = 1 + \frac{R_1}{R_2}\left(\frac{f_{l,f}{}^{A}}{f_{c0}}\right)^{R_2} \tag{35}$$

$$\frac{\varepsilon_{cc}{}^{A}}{\varepsilon_{c0}}=1+5\left(\frac{f_{cc}{}^{A}}{f_{c0}}-1\right) \tag{36}$$

$$n_A=\frac{E_c}{E_c-\dfrac{f_{cc}{}^{A}}{\varepsilon_{cc}{}^{A}}}\simeq\frac{1}{1-2.1\times10^{-4}\psi_A}\geq 1.1 \tag{37}$$

$$\psi_A=\frac{f_{cc}{}^{A}}{\varepsilon_{cc}{}^{A}\sqrt{f_{c0}}} \tag{38}$$

where $f_{cc}{}^{A}$ and $\varepsilon_{cc}{}^{A}$ are the peak axial strength and its corresponding axial strain, calibrated based on experimental AC specimens with passively confining system; $R_1$ and $R_2$ are the calibration factors in the determination of $f_{cc}{}^{A}$; $n_A$ is the concrete brittleness at the ambient condition depending on $\psi_A$, as recommended by Carreira and Chu [55]. Accordingly, at a certain level of $\varepsilon_c$, $f_{cc}{}^{A}$ can be determined as a function of $f_{l,f}{}^{A}$ based on Eq. (35), and $\varepsilon_{cc}{}^{A}$ can be, subsequently, calculated by Eq. (36), as input parameters for the determination of $f_c{}^{A}$ by Eq. (34).

In this study, the model developed by Shayanfar *et al.* [17] was extended to be applicable for the establishment of axial response of the concrete being subjected to a certain exposure temperature. Accordingly, by substituting the mechanical characteristics of HC column with those of AC column, at a certain level of $\varepsilon_c$ leading to FRP confinement pressure ( $f_{l,f}^{T}$ ), axial stress-strain base relation model ( $f_c{}^{T}$ versus $\varepsilon_c$ curve using function $g_1$) can be expressed as:

$$f_c^T=f_{cc}^T\,\frac{\left(\varepsilon_c/\varepsilon_{cc}^T\right)n_T}{n_T-1+\left(\varepsilon_c/\varepsilon_{cc}^T\right)^{n_T}} \tag{39}$$

in which

$$n_T = \frac{1}{1-2.1\times10^{-4}\psi_T} \geq 1.1 \qquad \text{for } T_m \leq 400\ °\text{C} \tag{40a}$$

$$n_T = 2 - \frac{1-4.2\times10^{-4}\psi_T}{1-2.1\times10^{-4}\psi_T}\left(2-\frac{T_m}{400}\right) \geq 1.1 \qquad \text{for } 400\ °\text{C} \leq T_m \leq 800\ °\text{C} \tag{40b}$$

$$\psi_T = \frac{f_{cc}^T}{\varepsilon_{cc}^T\sqrt{f_{c0}^T}} \tag{41}$$

where $f_{cc}^T$ and $\varepsilon_{cc}^T$ are the peak axial strength and its corresponding axial strain at a certain level of $\varepsilon_c$ as input parameters for the axial stress-strain base relation model. $n_T$ is the concrete brittleness that considers the influence of the maximum exposure temperature on $f_{cc}^T$, $\varepsilon_{cc}^T$ and $f_{c0}^T$ through the parameter $\psi_T$. Note that the axial stiffness of FRP confined HC columns with severe thermal-induced damages ($T_m \simeq 600-800\ °\text{C}$) seems to be almost constant with a linear axial behavior during axial loading as demonstrated in Fig. 9. Accordingly, in the present study, $n_T$ corresponding to $T_m = 800\ °\text{C}$ was assumed equal to a constant value of $n_T = 2$ during the entire loading history, adjusted based on the best-fit relation of the proposed model with the experimental axial stress-strain responses reported by Bisby *et al.* [4], Lenwari *et al.* [10], Ouyang *et al.* [12] and Song *et al.* [13]. Consequently, in Eq. (40b) ($400\ °\text{C} \leq T_m \leq 800\ °\text{C}$), $n_T$ was considered on the interval of the $n_T$ obtained from Eq. (40a) for $T_m = 400\ °\text{C}$ and $n_T = 2$ corresponding to $T_m = 800\ °\text{C}$, with a linear relationship with $T_m$.

For the establishment of $\varepsilon_{cc}^T$ at high temperatures, the preliminary comparative assessment of the proposed model with $\varepsilon_{cc}^T$ obtained based on Eq. (36) demonstrated a very significant underestimation in its model predictive performance, particularly for the cases with severe thermal-induced damages. On the other hand, based on experimental studies it was evidenced that the axial behavior of FRP confined HC columns tends to be similar to actively-confined-concrete due to more expansive behavior of HC than AC (Fig. 4). Therefore, the $\varepsilon_{cc}^{T}$ of HC columns with high exposure temperature is calculated according to the approach proposed by Lim and Ozbakkaloglu [41], exclusively developed for the case of actively-confined-concrete:

$$\varepsilon_{cc}^{T} = \varepsilon_{c0}^{T}\left[1 + 5\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}} - 1\right)\right] \qquad \text{for } T_m \geq 100\ ^{\circ}\text{C} \tag{42a}$$

$$\varepsilon_{cc}^{T} = \varepsilon_{c0}^{T} + 0.045\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}}\right)^{1.15} + \lambda_{\varepsilon} \qquad \text{for } 100\ ^{\circ}\text{C} \leq T_m \leq 200\ ^{\circ}\text{C} \tag{42b}$$

$$\varepsilon_{cc}^{T} = \varepsilon_{c0}^{T} + 0.045\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}}\right)^{1.15} \qquad \text{for } T_m \geq 200\ ^{\circ}\text{C} \tag{42c}$$

in which

$$\lambda_{\varepsilon} = \left[0.045\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}}\right)^{1.15} - 5\varepsilon_{c0T}\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}} - 1\right)\right]\left(\frac{T_m}{100} - 2\right) \tag{43}$$

In this study, a new axial strength model was developed, applicable to FRP confined HC columns, having a unified character with Eq. (35) when the concrete is under ambient conditions:

$$\frac{f_{cc}^{T}}{f_{c0}^{T}} = 1 + \frac{R_1}{R_2}\left(\overline{m}\frac{f_{l,f}^{T}}{f_{c0}^{T}}\right)^{R_2} + \frac{R_3}{R_4}\left(\frac{f_{l,f}^{T}}{f_{c0}^{T}}\right)^{R_4} \tag{44}$$

where $R_1$, $R_2$, $R_3$ and $R_4$ are the calibration factors in the determination of $f_{cc}^{T}$ obtained from the regression analyses on FRP passively confined HC columns; $\overline{m}$ is the calibration factor reflecting the influence of more expansive behavior of the concrete with a certain level of thermal damage, which activates earlier the confining system compared to the AC column at ambient temperature (based on the comparative assessment on the dilation behavior of FFCC-

H/FFSC-H presented in Fig. 4). In Eq. (44), the second term ( $\frac{R_1}{R_2}\left(\overline{m}\frac{f_{l,f}^T}{f_{c0}^T}\right)^{R_2}$ ) represents the improvements obtained from the confinement of an AC column with axial strength equal to $f_{c0}^T$, considering the effect of earlier activation (through $\overline{m}$) compared to AC; The third term ( $\frac{R_3}{R_4}\left(\frac{f_{l,f}^T}{f_{c0}^T}\right)^{R_4}$ ) in Eq. (44) considers the increase of confinement-induced improvements due to thermal-induced damage. Based on experimental axial stress versus axial strain relations, Shayanfar *et al.* [17] proposed new expressions to determine the calibration factors of $R_1$ and $R_2$, which was rearranged by substituting $f_{c0}$ by $f_{c0}^T$:

$$R_1 = \frac{23.9}{{\beta_{0T}}^{0.5}\lambda_{fc}\lambda_{Rb}}{\rho_{K,f}}^{0.67} \leq 4.25 \tag{45}$$

$$R_2 = \frac{1.85}{{\beta_{0T}}^{0.2}}{\rho_{K,f}}^{0.26} \geq 0.3 \tag{46}$$

in which

$$\lambda_{fc} = 0.75 + 0.008\beta_{0T} f_{c0} \tag{47}$$

$$\lambda_{Rb} = 1.5\left(1 - 1.1R_b\right) \geq 1 \tag{48}$$

where $\lambda_{fc}$ and $\lambda_{Rb}$ reflect the impact of concrete axial strength and the dimension of corner radius in the calibration of $R_1$, respectively. In order to determine the calibration factors of $R_3$ and $R_4$ representing the effect of thermal-induced damage in terms of confinement-induced enhancements, the experimental axial stress-strain responses reported by Bisby *et al.* [4], Lenwari *et al.* [10], Ouyang *et al.* [12] and Song *et al.* [13] were used for statistical modeling.

Through regression analyses on the assembled data, the calibration factors $R_3$ and $R_4$ are determined from:

$$R_3 = \frac{\lambda_T}{\lambda_{rb}\lambda_K} \geq 0 \tag{49}$$

$$R_4 = 0.92\left(\frac{f_{l,f}^T}{f_{c0}^T}\right)^{0.1} \tag{50}$$

in which

$$\lambda_T = 3.55\left(\frac{T_m}{1000}\right) - 1.55 \geq 0 \tag{51}$$

$$\lambda_K = 1.15 - 0.022 K_H \frac{n_f t_f E_f}{D_{eq} f_{c0}} \tag{52}$$

$$\lambda_{rb} = 1.22 R_b^{0.25} \geq 0.85 \tag{53}$$

where $\lambda_T$, $\lambda_K$ and $\lambda_{rb}$ reflect the impact of maximum exposure temperature ($T_m$), confinement stiffness and the dimension of corner radius in the calibration of $R_3$, respectively. As shown in Fig. 10a, the developed model parameter ($\lambda_T$) provides a sufficient agreement with the experimental counterparts, which was determined based on the best-fit relation of the proposed model with the experimental axial stress-strain responses.

Due to the fact that pre-existing micro-cracks in thermally damaged concrete tend to activate passively confining system at the initial stage of axial compressive loading, the calibration factor of $\overline{m}$ was considered in the establishment of Eq. (44), having been obtained from regression analysis to enhance the confinement effectiveness, resulting:

$$\overline{m} = 1 + m_0 \exp\left(-11.2 \frac{f_{l,f}^T}{f_{c0}^T}\right) \tag{54}$$

in which

$$m_0 = \frac{m_T}{m_\rho m_r} \tag{55}$$

$$0 \le m_T = 0.025\left(T_m - 100\right) \le 2.5 \qquad \text{for } T_m \le 400 \text{ °C} \tag{56a}$$

$$m_T = 2.5 - 0.01\left(T_m - 400\right) \ge 0.3 \qquad \text{for } T_m \ge 400 \text{ °C} \tag{56b}$$

$$m_r = 0.3 + 0.7R_b \tag{57}$$

$$m_\rho = 0.2{\beta_{0T}}^{0.3}{\rho_{K,f}}^{-0.4} \tag{58}$$

where $m_T$, $m_r$ and $m_\rho$ are the calibration factors determined based on the regression analysis reflecting the effect of maximum exposure temperature ($T_m$), corner radius ratio ($R_b$) and confinement stiffness ($\rho_{K,f}$). As shown in Fig. 10b, the developed model parameter ($m_T$) has a good agreement with the experimental counterparts, which extracted based on Eqs. (51 and 52) ($m_T^{Exp} = m_\rho m_r \dfrac{\overline{m}^{Exp} - 1}{\exp\left(-11.2 f_{l,f}^T / f_{c0}^T\right)}$ where $\overline{m}^{Exp}$ was determined using back analysis from the best fit of the experimental results of the axial stress-strain curve of test specimens with those obtained from the developed model).

## 5- Calculation Process

In the following, the calculation methodology of the proposed model, which can be implemented into a spreadsheet, for determining the axial response of FRP confined concrete submitted to a certain level of thermal damage is presented. The calculation procedure is as:

**1.** Determine the mechanical characteristics of unconfined heat-damaged concrete:

- Peak axial strength ($f_{c0}^T$) by Eq. (1)

- Axial strain corresponding to $f_{c0}^T$ ($\varepsilon_{c0}^T$) by Eq. (6)

**2.** Assume a value for $\varepsilon_c$ on the interval (0, $\varepsilon_{cu}$] where $\varepsilon_{cu}$ is the ultimate axial strain when FRP rupture occurs.

**3.** Determine the dilation response corresponding to $\varepsilon_c$

- Secant Poisson's ratio ($\nu_s^T$) by Eq. (29) and the data presented in Fig. 6
- Confinement pressure ($f_{l,f}^T$) by Eq. (22)

**4.** Determine the axial response corresponding to $\varepsilon_c$

- Calibration factors $R_1$, $R_2$, $R_3$ and $R_4$, by Eqs. (45), (46), (49), and (50)
- $\overline{m}$ factor from Eq. (54)
- Peak axial strength ($f_{cc}^T$) by Eq. (44)
- Axial strain ($\varepsilon_{cc}^T$) corresponding to $f_{cc}^T$ by Eq. (42)
- Axial stress ($f_c^T$) by Eq. (39)

**5.** Continue the aforementioned incremental procedure up to $\varepsilon_{cu}$, $f_c^T$ versus $\varepsilon_c$ relation can be calculated.

Since the focus of the current study was given on the simulation of global axial stress-strain curves, the experimental values of ultimate axial strain ($\varepsilon_{cu}$) was used to terminate the computation process.

It is noteworthy that the reliability of regression analyses performed for developing predictive equations (as key components of the proposed model) is limited to the range of input/output variables supported in the used database. Accordingly, the predictive performance of these equations can be improved through recalibrating the model components based on new datasets consisting of the relevant variables with a broader range. Furthermore, the experimental data used for calibrating the failure surface function and the coupled dilation model was obtained from tests on relatively small FRP confined prototypes of square/circular concrete specimens where thermal distribution inside the concrete can be reasonably considered uniform. Therefore, for real cases with a larger dimension and non-uniform thermal distribution whose relative variables might not be in the aforementioned interval, the key components of the proposed model might need to be recalibrated, which will be the focus of a future study. Considering the relatively simple methodology of the proposed analytical-based model, it can be extended potentially to FRP confined heat-damaged RC columns of rectangular cross-section, through addressing properly the influences of dual FRP-steel confinement, sectional aspect ratio (the ratio of longer and shorter cross-section dimensions) and their interactions with exposure temperature in terms of axial and dilation behavior.

## 6- Verification

This section assesses the predictive performance of the developed analysis-oriented model for the prediction of axial response of concrete specimens submitted to a certain maximum exposure temperature, and after the specimens have attained the environmental temperature, subsequently, confined with FRP confining system. For this purpose, the axial stress-strain curve obtained from the proposed model is compared to that obtained from the experimental studies conducted by Bisby *et al.* [4], Lenwari *et al.* [10], Ouyang *et al.* [12] and Song *et al.* [13].

Bisby *et al.* [4] performed an experimental study to investigate the axial stress-strain response of FRP fully confined circular HC columns. The diameter and height of the test specimens were 100 mm and 200 mm, respectively. The axial compressive strength of AC column at the room temperature was 28 MPa. The values of nominal thickness, modulus of elasticity and rupture strain of CFRP jackets were reported as 0.12 mm, 241.1 GPa and 1.7%, respectively. All the specimens submitted to a certain level of maximum exposure temperature were confined by one CFRP layer. Complete details regarding the test specimens can be found from Bisby *et al.* [4]. Fig. 11 presents the comparison of the results obtained from the analytical model with those measured experimentally. As can be seen, the model has a good predictive performance in the estimation of axial responses of the test specimens submitted to 300 °C, 500 °C and 700 °C, even though there is a slight overestimation in terms of maximum axial strength for T500-L1.

Lenwari *et al.* [10] experimentally determined the axial stress-strain response of FRP fully confined circular HC columns. The diameter and height of the test specimens were 150 mm and 300 mm, respectively. The axial compressive strength of AC column at the room temperature was 40.5 MPa. The values of nominal thickness, modulus of elasticity and rupture strain of CFRP jackets were reported as 0.131 mm, 234.1 GPa and 1.8%, respectively. All the specimens submitted to a certain level of maximum exposure temperature were confined by one CFRP layer. Complete details regarding the test specimens can be found from Lenwari *et al.* [10]. Fig. 12 demonstrates the comparison of the results obtained from the analytical model with those measured experimentally. As can be seen, the model was able to predict the experimental counterparts with a good agreement, even though there is a slight overestimation in terms of maximum axial strength for T300-L1 and T500-L1.

For further investigation of the capability of the developed confinement model, the axial stress-strain curves obtained from experimental studies conducted by Ouyang *et al.* [12] and Song *et al.* [13], where the test specimens were submitted to a high level of thermal-induced damage (

$T_m \geq 600$ °C), are compared with those analytically obtained from the model, as shown in Figs. 13 and 14. As can be observed, the model could accurately predict the full range of the experimentally measured axial responses, except for a slight underestimation for the cases of T600-L3 and T600-L3 tested by Song *et al.* [13] (Figs. 14e and f).

Due to the unified character of the proposed analysis-oriented model at ambient and elevated temperature conditions, Fig. 15 compares the axial stress- strain relations of FFSC-A reported from the experimental study conducted by Wang and Wu [6] with those analytically obtained from the model to assess its capability for the cases at ambient condition. All specimens had a section dimension of 150 mm and a height of 300 mm, with a sectional corner radius ratio ($R_b$) varying from 0 ($r = 0$) to 1 ($r = 75$ mm) representing a square cross section with sharp edges and circular cross-section, respectively. Two series of FFSC-A specimens with concrete strengths of $f_{c0} \simeq 30$ MPa and $f_{c0} \simeq 50$ MPa were tested. The CFRP thickness, tensile elastic modulus and rupture strain were 0.165 mm, 219 GPa, and 1.99%, obtained from flat coupon tensile tests. The complete details of the experimental program can be found in Wang and Wu [6]. As can be seen in Fig. 15, in general, there are a good agreement between the experimental axial stress-strain relationships with those obtained from the proposed model, confirming the successful simulation of the corner radius ratio ($R_b$) influence on confinement-induced enhancements of FFSC-A at ambient conditions.

## 7- Summary and Conclusion

This paper has addressed the development of a new analysis-oriented model to predict the axial and dilation behavior of FRP confined HC circular/square concrete columns. Through regression analyses performed on a large database of unconfined heat-damaged concrete experimentally tested specimens, new expressions were developed to determine its mechanical characteristics in terms of axial compressive strength and its corresponding axial strain. A new model was developed to determine dilation response of FRP confined HC columns by formulating the effect of thermal damage level on Poisson's coefficient versus axial strain relationship. Subsequently, a new axial stress-strain model, coupled with the developed dilation model, was proposed to calculate the axial behavior of FRP confined HC columns with the different levels of maximum exposure temperature. Comparisons with axial and dilation results reported by available experiment studies in the literature verified that the developed analysis-oriented model is able to predict the experimental counterparts with good accuracy, and has a relatively simple format for design purposes by using a data sheet.

## Data Availability Statement

All data and models related to the present study could be available from the corresponding author upon rational request.

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