European Journal of Engineering and Technology Research
ISSN: 2736-576X

Analytical Model for Steel Fibre-Reinforced Concrete-Filled Tubular Columns Exposed to Fire

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***Abstract* — This study proposes an analytical model for the evaluation of the mechanical behaviour of circular hollow section columns infilled with steel fibre-reinforced concrete when exposed to elevated temperatures in a fire situation. This work includes the discussion of the results from a sensitivity analysis conducted through numerical simulation and the development of an analytical model based on the prEN1994-1-2:2021 European standard for the design of reinforced concrete structures exposed to fire. The sensitivity analysis aims to compare the fire resistance of centrally and eccentrically loaded circular hollow section columns infilled with plain and steel fibre-reinforced concrete. The proposed analytical model for determining the design buckling load of composite steel fibre-reinforced concrete-filled tube columns in a fire situation is ready to serve as an annex or supplementary technical document to the EN1994-1-2:202x standard.**

***Keywords* — Analytical Model, Concrete-Filled Steel Tubes, Circular Hollow Section, Fire Resistance, Sensitivity Analysis, Steel Fibre-Reinforced Concrete.**

I. Introduction

Concrete-filled steel tubes (CFST) provide the opportunity of leveraging the advantages of both steel and concrete. In some instances, this combination of materials results in properties that are better than those of either material alone [1]. Infilling a steel tube with concrete yields better mechanical properties at ambient temperatures and bolsters their fire resistance [2]. Members with such advantages are widely used around the world in the construction of mid- and high-rise buildings. CFSTs allow for a more efficient installation process than concrete composites reinforced with steel rebars, particularly because CFSTs do not require formwork. Steel fibres have been used in addition to traditional reinforcement bars. Owing to the fire resistance requirements for structural elements, numerous studies have focused on the behaviour of fibre-reinforced concrete (FRC) at elevated temperatures [3]-[7]. Despite the limited efficiency provided by the steel fibres to CFST columns at ambient temperature compared with that provided by the steel tube, at elevated temperatures an FRC core appears to be more advantageous than plain concrete. The experimental [8], [9] and numerical studies [10] conducted by Kodur *et al.* indicate that the inclusion of steel fibres in the concrete core increases the fire resistance of the columns and has a better potential to improve their economic efficiency than the use of rebars.

Kodur [11] proposed a design equation for determining the fire resistance of CFST columns with concrete infilling and incorporating either longitudinal bars or steel fibres. Xiong [2] presented a design guide for CFST members with high-strength infill. This design guide was based on the prEN 1994-1-1 standard [12] and utilised test data from the literature on CFSTs with normal- and high-strength infills to develop design methods applicable for ambient and elevated temperatures.

Espinos *et al.* [13]-[15] conducted extensive studies on axially and eccentrically loaded slender circular CFST columns. These resulted in an improved simple European simple fire resistance calculation model for plain and bar-reinforced concrete-filled columns exposed to fire all around, according to the standard temperature-time curve [16], [17]. However, their work does not take into account the benefits of using steel fibre-reinforced concrete (SFRC) [18].

Design methods for circular hollow section (CHS) columns are continuously being developed and improved by multiple authors. Jindra [19] used advanced finite element models to investigate the flexural buckling resistance of austenitic stainless steel CHS columns with variable relative slenderness λ (range 0.2–1.8). Fellouh [20] investigated the fire performance of partially encased concrete columns under eccentric loads using an advanced calculation method.

Project GAČR 15-19073S was prepared to remedy the gaps in the European simple fire resistance calculation for CFST columns by developing an analytical design method for circular CFST columns with an SFRC infill, as shown in Fig. 1.

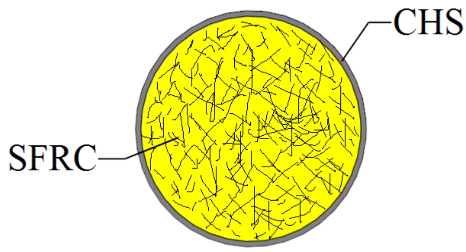

Fig. 1. Cross section of a CFST column with SFRC infill.

The project was divided into parts: experimental study, development of a numerical model, sensitivity analysis, and development of a simple analytical method. The experimental study demonstrated a broad scope including material tests [21] and full-scale experiments [22]-[24] on columns.

Submitted on December 27, 2022.
Published on February 17, 2023.
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Tretyakov *et al.* [23], [25] developed an FEM fire response model of a steel fibre-reinforced concrete-filled tubular column.

This study presents the results of a sensitivity analysis and the development of an analytical model for determining the design buckling load of a steel fibre-reinforced concrete-filled tubular column in a fire situation. A three-dimensional model built with the FEM software ATENA Červenka [26] was used to determine the fire resistance of centrically and eccentrically loaded composite columns filled with plain and steel fibre-reinforced concrete subjected to the fire conditions stipulated in the ISO 834-10:2014 standard. The proposed model modifies the flexural stiffness reduction coefficient and the reduction coefficient due to eccentricity when using concrete with a 0,5% content of steel fibres according to the method described in the prEN1994-1-2:2021 standard [18].

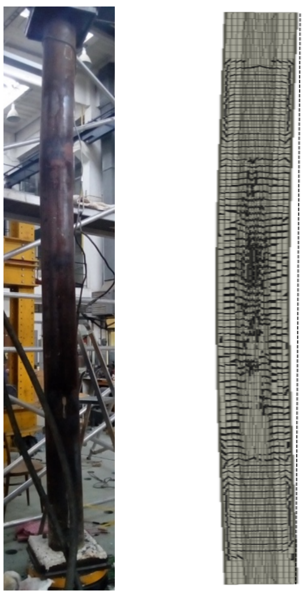
II. Additional Validation of the FEM Model

An advanced FEM model of the steady-state regime is validated against experimental data [24]. The steady-state model of the column, which was developed using the ATENA FEM software in conjunction with the GID pre-processor, combines a mechanical and a thermal model. The thermal model calculates the heat transfer, whereas the mechanical model calculates the response of the column to the vertical force and thermal loading.

The following material properties were assigned to the SFRC in the model: f_c =75 MPa, W_d = 1,33 mm, f_t = 2,6 MPa and G_f = 8100 N/m. These values correspond to those proposed by Tretykov [23], [25]. The numerical and experimental results are compared in Table I, and the typical failure modes are shown in Fig. 2.

TABLE I: Comparison of FEM and Experimental Results

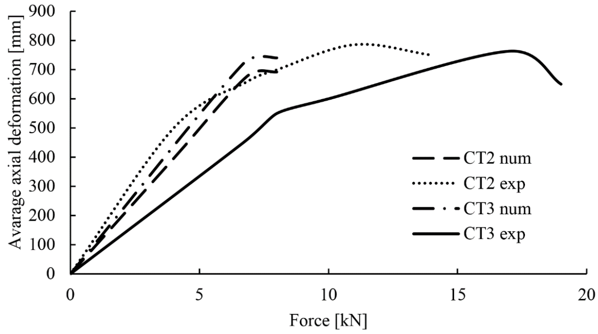
Specimen	Heating time (min)	Temperature of pads (°C)	N (kN)		$\frac{N_{FEM}}{N_{TEST}}$
			Test	FEM	
CT2	520	600 C	786	692	0,88
CT3	300	600 C	763	740	0,97
CT4	270	600 C	1082	844	0,78
CT5	170	500C	1860	1640	0,88



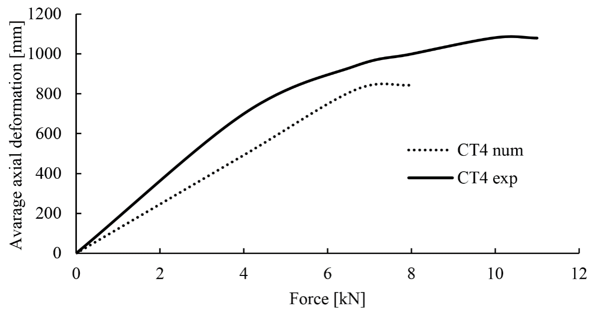
(a) (b)

Fig. 2. a) Failure mode of the tested column under steady-state regime; b) Predicted deformed shape.

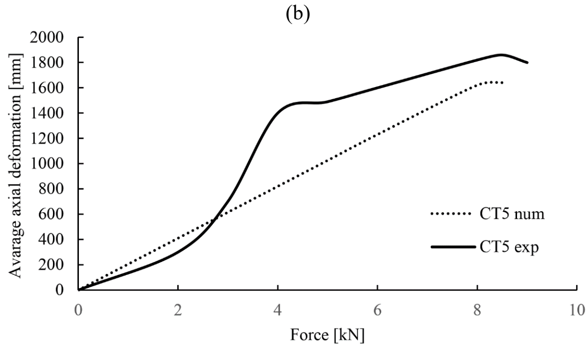
During the experiments under a steady-state regime, the columns exhibited a global buckling failure mode. The average axial deformation of the specimens during the steady-state compression tests and that predicted using the FEM are shown in Fig. 3.



(a)



(b)



(c)

Fig. 3. Measured and predicted average axial deformation of specimens: a) CT2 and CT3; b) CT4; c) CT5.

As shown in Fig. 4, the numerical results tend to be on the safe side, with average differences with respect to the experimental data of 12,25% <15 %. Thus, the proposed model can be considered validated by the steady-state experiments and appropriate for further sensitivity analyses.

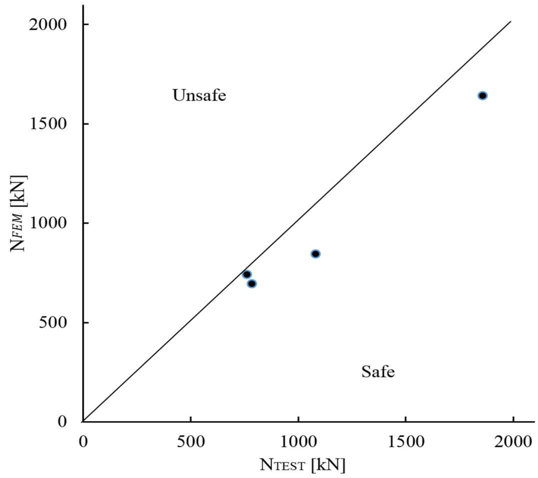


Fig. 4. Predicted and experimental buckling load for SFRC specimens.

III. Sensitivity Analysis

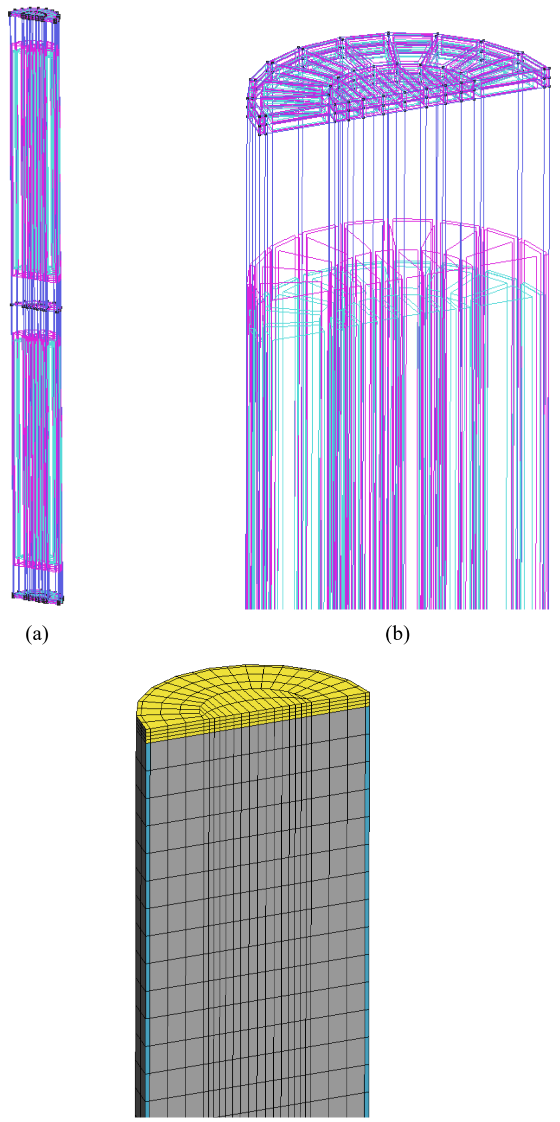
A. General

The FEM global model developed in the ATENA software was used to conduct a sensitivity analysis under the ISO standard fire conditions by Tretyakov [23], [25]. Numerical experiments were conducted based on the prepared combinations (these are the combinations described in subsection C). The sensitivity analysis aimed to compare the fire resistance of axial and eccentrically loaded CHS columns with steel fibre-reinforced and plain concrete infill in a fire situation.

B. FEM Model Description

The thermal model comprises only a steel tube and concrete core. The mechanical model was also provided with elastic plates to set the boundary conditions and contact 3D surfaces between the materials. The geometry of the model is shown in Fig. 5.

The mechanical model analyses included three stages with different boundary conditions following Tretyakov [25], imperfections; the boundary conditions and the load; the heat load from the thermal model.



(a) (b) (c)

Fig. 5. Geometry of the FEM model: a) Surfaces; b) volumes; c) mesh.

The thermal model was analyzed in a single stage with fire thermal loading. The applied thermal load is schematically shown in Fig. 6.

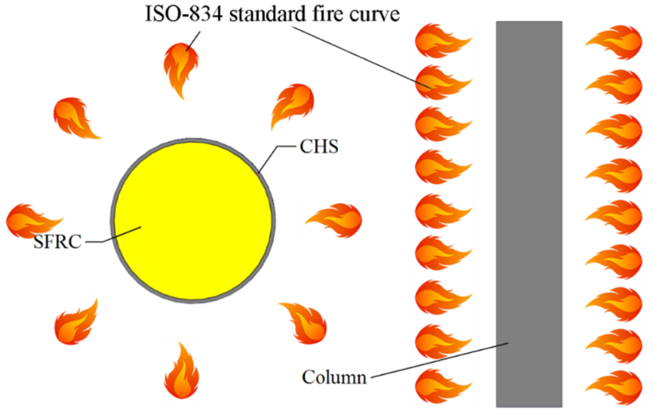


Fig. 6. Schematic description of the thermal load applied to the specimens.

The fire loading is according to the ISO-834 nominal standard fire curve. The boundary conditions in the thermal model are shown in Fig. 7.

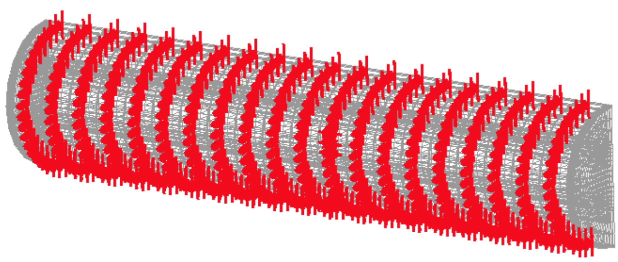


Fig. 7. Boundary conditions in the thermal model.

C. Combinations of Parameters for the Sensitivity Analysis

The parameters considered for the sensitivity analysis were the outer diameter D of the columns, relative slenderness of the columns at room temperature $\bar{\lambda}$, fibre content ρ_f, load level μ ($N_{\mathrm{fi,Rd,num}}$ is obtained by multiplying μ by N_{Rd} or $N_{\mathrm{Rd},\delta}$), and relative eccentricity e/D.

A study was conducted to define the analysis cases of interest, as well as to determine the cross-sectional dimensions and ranges of the parameter variations. TABLE shows the values of the parameters used, which led to a total of nine circular columns. The specimens were designed without rebars and a fixed steel tube wall thickness *t* of 6 mm. Other adopted input parameters included: pinned-pinned (P-P) boundary conditions, 4% moisture content, 355 MPa steel tube yield strength, and 30 MPa cylindrical concrete compressive strength.

TABLE II: Combinations of Parameter Values for the Sensitivity Analysis

Parameter	Values		
D (mm)	159	324	406,4
t (mm)	-	6	-
$\bar{\lambda}$	0,3	0,5	1
ρ_f (%)	-	0,5 0	-
Concrete	-	C 30 /37	-
e / *D*	0	0,25	0,5
μ (%)	20	40	60

TABLE III: NOTATION LIST

Notation			
CFST	Concrete-filled steel tube	$f_{c,\theta}$	Compressive strength of concrete at elevated temperature
FRC	Fibre-reinforced concrete	f_c	Compressive strength of concrete at ambient temperature
SFRC	Steel fibre-reinforced concrete	$f_{y,\theta}$	Yield strength of steel at elevated temperature
CHS	Circular hollow section	f_y	Yield strength of steel at ambient temperature
HFRC	Hybrid fibre-reinforced concrete	$f_t,$	Peak tensile stress value for SFRC
FEM	Finite element method	A	Cross-sectional area
L	Length of column	A_m/V	Section factor
D	External diameter of steel tube	$N_{fi,cr}$	Elastic critical load in a fire situation
t	Thickness of steel tube	$N_{fi,Rd}$	Design axial buckling load in a fire situation
e	Eccentricity	$N_{fi,Rd,\delta}$	Design eccentric buckling load in a fire situation
$\varphi_{i,\theta}^{f}$	Flexural stiffness reduction coefficient for steel fibre-reinforced concrete	N_{Rd}	Design axial buckling load at room temperature
$\varphi_{i,\theta}$	Flexural stiffness reduction coefficient for plain concrete	$N_{Rd,\delta}$	Design eccentric buckling load at room temperature in case of load
α_f	Reduction coefficient due to eccentricity for steel fibre-reinforced concrete	$E_{i,\theta}$	Modulus of elasticity of the *i*-th material at temperature θ
α	Reduction coefficient due eccentricity for plain concrete	$k_{i,\theta}$	Reduction factor for a material property at elevated temperature
χ	Resistance reduction factor according to European buckling curves	l_θ	Buckling length in a fire situation
$\overline{\lambda}$	Relative slenderness at ambient temperature	N	Buckling load
$\overline{\lambda_\theta}$	Relative slenderness in a fire situation	R	Fire resistance time (min)
μ	Load level	G_f	Fracture energy
ρ_f	Percent fibre content in the concrete	W_d	Critical compressive displacement of SFRC

D. Results

The sensitivity analysis was performed using the ATENA software, excluding samples with 15 min > R and R> 240 min, and limiting the total number of numerical experiments to 84. The results of the sensitivity analysis, which are presented in Fig. 8, demonstrated that, in all experiments, $R_{\text{sfrc,num}}$, min > $R_{\text{pc,num}}$, min.

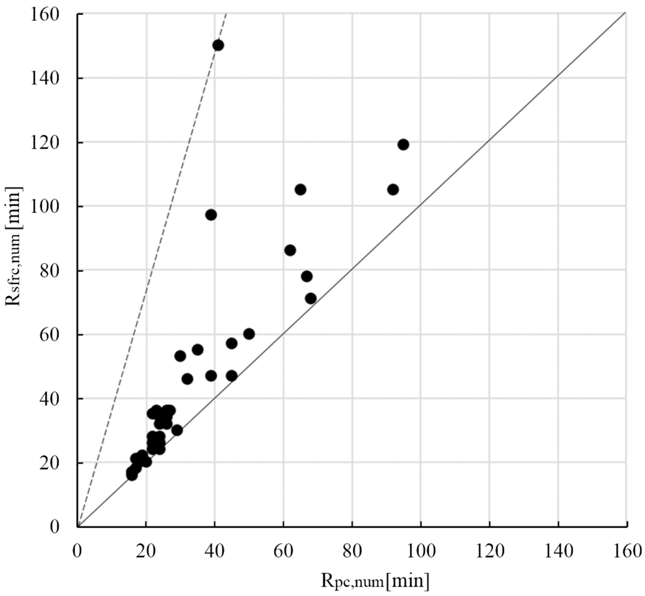


Fig. 8. Comparison of fire resistance of columns filled with PC and SFRC.

In the sensitivity analysis, R values were obtained in minutes for the same load, but different concrete infills. Based on the data obtained from the sensitivity analysis, an analytical solution was proposed for determining the design buckling load in a fire situation. The analytical model, discussed in section IV, is based on the fact that for each centrically loaded numerical experiment, the $N_{f\text{i,Rd}}$ for PC and SFRC are equal, and for each eccentrically loaded test, the $N_{\text{fi,Rd},\delta}$ for PC and SFRC are also the same.

IV. ANALYTICAL MODEL

A. General

This section focuses on the development of the analytical model and simplified method for the SFRC CHS columns, to be used in addition to the approach proposed by Romero et al. for prEN1994-1-2021 [18]. SFRC offers some advantages over plain concrete in terms of bending response. It has been demonstrated that the EC1994-1-2 estimates a very conservative fire resistance (or fire design buckling load) because it does not consider the influence of the SFRC core on the column behaviour.

The use of FRC modifies the ductility and design buckling load of the composite member. This effect can be captured by adjusting the effective flexural stiffness. Therefore, an analytical model was proposed to calculate an appropriate flexural stiffness reduction coefficient $\varphi_{\text{c},\theta}^{\text{f}}$and reduction coefficient due to eccentricity α_f . The solution is based on the fact that, under the same buckling load, different fire resistance (R) values were obtained for columns with SFRC and PC infilling.

Equation (1) is used to define the flexural stiffness reduction coefficient, while (2) is used to define the reduction coefficient due to eccentricity.

$$N_{\text{fi,Rd,PC}} = N_{\text{fi,Rd,SFRC}} \tag{1}$$

$$N_{fi,Rd,\delta,PC} = N_{fi,Rd,\delta,SFRC} \tag{2}$$

Thus, to find a solution for the reduction coefficients it is necessary to solve the inverse problem [16], [17].

B. Flexural Stiffness Reduction Coefficient

During the sensitivity analysis, the design axial buckling loads in the fire situation were the same for CHS columns with either SFRC or PC infill using (3).

$$N_{\text{fi,Rd}} = \chi \cdot N_{\text{fi,pl,Rd}} \tag{3}$$

To obtain an adequate analytical solution, it is necessary to modify the effective flexural stiffness of the entire column in the fire situation using (4).

$$(EI)_{fi,eff} = \varphi_{a,\theta} E_a(\theta_{a,eq}) I_a + \varphi_{c,\theta} E_c(\theta_{c,eq}) I_c + \varphi_{s,\theta} E_s(\theta_{s,eq}) I_s \quad (4)$$

where,
$\varphi_{a,\theta} E_a(\theta_{a,eq}) I_a$ following prEN1994-1-2:2021 [18],
$\varphi_{s,\theta} E_s(\theta_{s,eq}) I_s = 0$ because rebars are absent,
$\varphi_{c,\theta} E_c(\theta_{c,eq}) I_c$ to be modified.

For PC infilling, the CHS coefficient was fixed to $\varphi_{c,\theta} = 0{,}8$. The initial tangent stiffness (calculated as 3/2 times the secant modulus) is used [16]. An increase in the flexural stiffness is proposed to consider the improvement in the ductility and compressive and tensile strengths of concrete due to the use of steel fibres.

$$\varphi_{c,\theta} E_c(\theta_{c,eq}) I_c \rightarrow \varphi_{c,\theta} \cdot (3/2) \cdot k_c \cdot \left(\frac{f_c}{\varepsilon_{cu,\theta}}\right) \cdot I_c \quad (5)$$

Simplified cross-sectional temperature fields and reduction factors (including k_c) for material properties at elevated temperature are selected following the recommendations in prEN1994-1-2:2021.

For axially loaded cases, it is necessary to replace the flexural stiffness reduction coefficient $\varphi_{c,\theta}$ for plane concrete with $\varphi^f_{c,\theta}$ for SFRC (can be >1).

$$\varphi^{\mathrm{f}}_{\mathrm{c},\theta} = \varphi_{\mathrm{c},\theta} + \Delta\, \varphi^{\mathrm{f}}_{\mathrm{c},\theta,\mathrm{simp}} = 0{,}8 + \Delta\, \varphi^{\mathrm{f}}_{\mathrm{c},\theta,\mathrm{simp}} \quad (6)$$

To simplify the equation for $\Delta\, \varphi^{\mathrm{f}}_{\mathrm{c},\theta,\mathrm{simp}}$, it was calculated using the data from the sensitivity analysis. Simultaneously, a nonlinear relationship between l_θ/D and $\Delta\varphi^f_{c,\theta}$ was observed in (7).

$$\Delta\varphi^f_{c,\theta} = f(l_\theta/D) \quad (7)$$

Following prEN1994-1-2:2021, [16] a power function was chosen to obtain a regression equation of the data, as shown in Fig. 9.

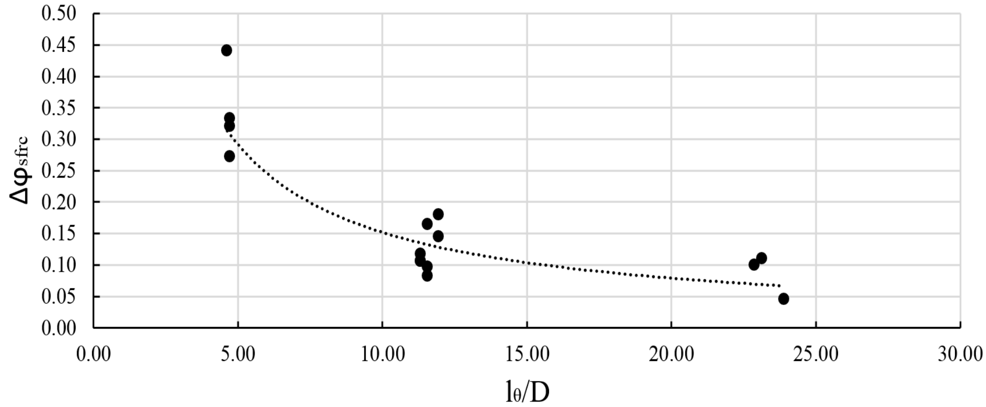


Fig. 9. Flexural stiffness reduction coefficient for SFRC.

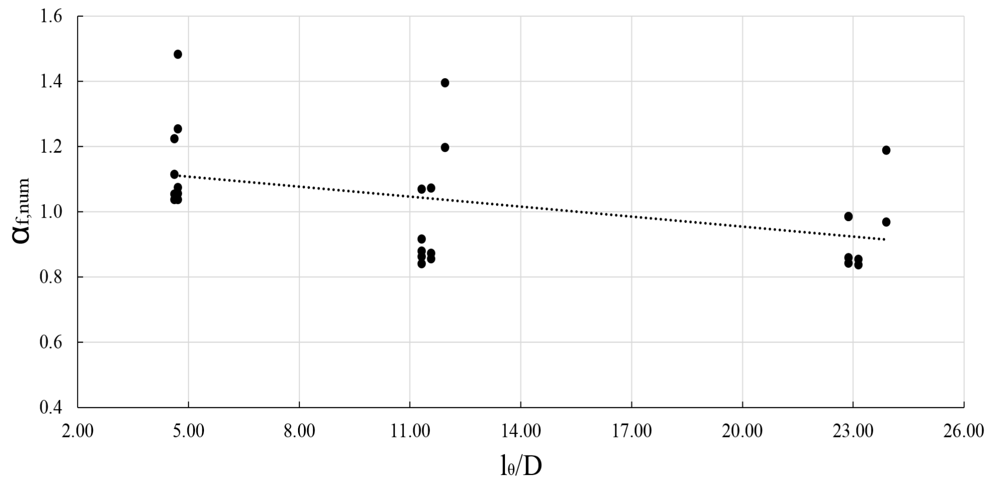


Fig. 10. Reduction coefficient for SFRC due to eccentricity.

For a fibre content of 0,5 %, (8) is used.

$$\Delta\varphi^{\mathrm{f}}_{\mathrm{c},\theta,\mathrm{simp}} = 1{,}3237 (l_\theta/D)^{-0{,}94} \quad (8)$$

with standard deviation $\Delta\varphi^{\mathrm{s}}_{\mathrm{c},\theta,\mathrm{num}}$ and $\varphi^{\mathrm{f}}_{\mathrm{c},\theta,\mathrm{simp}} = 0{,}1$.

Next, a new method for eccentrically loaded CFST columns was proposed to complete the proposed simplified design approach.

C. Reduction Coefficient due to Eccentricity

In the sensitivity analysis, the values of the design buckling load in the fire situation were the same for eccentrically loaded columns with either SFRC or PC infill.

$$N_{\mathrm{fi,Rd},\delta} = \alpha \cdot \left(\frac{N_{\mathrm{Rd},\delta}}{N_{\mathrm{Rd}}}\right)_{\mathrm{room}} \cdot N_{\mathrm{fi,Rd}} \quad (9)$$

N_{Rd} and $N_{\text{Rd},\delta}$ following the design rules in EN 1994-1-1 [12].

Reduction coefficient due to eccentricity is calculated using (10).

$$\alpha = \alpha_{\text{Am/V}} \cdot \alpha_{\text{D/t}} \cdot \alpha_{\text{R}} \cdot \alpha_{\text{s}} \quad (10)$$

where,
$\alpha_{\text{Am/V}}$ and $\cdot\, \alpha_{\text{D/t}}$ following prEN1994-1-2:2021 [18]
$\alpha_{\text{R}} = 6{,}9598 \ \cdot \ R^{-0{,}221}\ l_{\theta}\ /D \leq 15$
$\alpha_{\text{R}} = 0{,}92\ l_{\theta}\ /D > 15$
$\alpha_{\text{s}} = 0{,}8$ - plain concrete.

For eccentrically loaded SFRC columns, it is necessary to add a reduction coefficient α_f using (11).

$$\alpha = \alpha_{Am/V} \cdot \alpha_{D/t} \cdot \alpha_R \cdot \alpha_s \cdot \alpha_f \quad (11)$$
$$\alpha_f\ can\ be > 1$$

if $R \geq 60$ min, $\alpha = 0{,}92 \cdot \alpha_s \cdot \alpha_f$.

To obtain a simplified the equation for $\alpha_{f,simp}$, $\alpha_{f,num}$ were calculated using the results obtained from the sensitivity analysis given in (12).

$$\alpha_{f,simp} = f(l_\theta/D) \quad (12)$$

A linear function was chosen to approximate the data in Fig. 10, obtaining (13).

$$\alpha_{f,simp} = -0{,}0102 \cdot (l_\theta/D) \quad + 1{,}1592 \quad (13)$$

for a fibre content of 0,5%,
if p=0% and p_f=0,5%→ $\alpha_{s,sfrc} = 1$,
if R≥ 60min, $\alpha = 0{,}92 \cdot \alpha_f$.

Standard deviation $\alpha_{f,simp}$ and α_f is 0,13. Once derived, these analytical dependencies were used to verify the proposed simplified method, as discussed in the next section.

V. Verification

A. General

Three criteria (approved by the CEN/TC250/SC4 Horizontal Group Fire) must be met for the developed method to be considered accurate:

- The calculation result shall not be on the unsafe side of the reference result by more than 15%.
- A maximum of 20% of the individual calculation results shall be on the unsafe side.
- The mean percental difference between the calculation and reference results should be on the safe side.

B. Verification of Centrically Loaded SFRC CHS Columns

Fig. 11 shows the values of $N^{pre}_{fi,Rd,sfrc}$ for all the concentrically loaded cases considered in the sensitivity analysis, as predicted with the proposed method, by applying the derived reduction coefficients for SFRC.

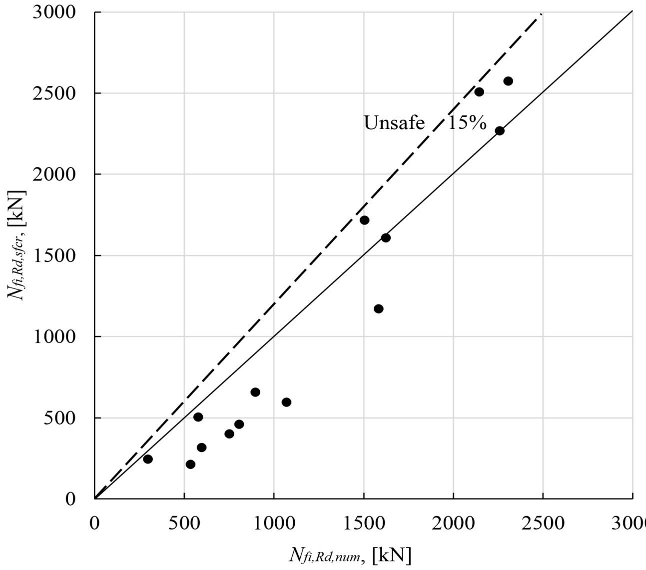


Fig. 11. Verification of the developed simplified design method for centrically loaded columns.

It can be observed that the predictions obtained were generally safe and met the criteria of CEN/TC250/SC4, indicating that the proposed model for the flexural stiffness reduction coefficient is adequate.

C. Verification of Eccentrically Loaded SFRC CHS Columns

The $N^{pr\,1}_{fi,Rd,\partial,sfrc}$ values shown in Fig. 12 were obtained by applying the proposed reduction coefficient due to eccentricity for SFRC to all the eccentrically loaded cases considered in the sensitivity analysis.

It can be observed that some of the results predicted by the proposed method lie on the unsafe side of the reference value by more than 15%. All unsafe cases occurred in slender columns with 22<l_θ/D<24. Thus, the method must be modified. By checking the regression function, the predicted buckling load at $l_\theta/D > 15$ exceeds the numerical results, therefore, at $l_\theta/D > 15$ we take $\alpha_{f,simp} = 0{,}4$.

$$\alpha_{f,simp} = -0{,}0102 \cdot (l_\theta/D) \quad + 1{,}1592,\ l_\theta/D \leq 15 \quad (14)$$

where, $\alpha_{f,simp} = 0{,}4$, $l_\theta/D > 15$ for a fibre content of 0,5%.

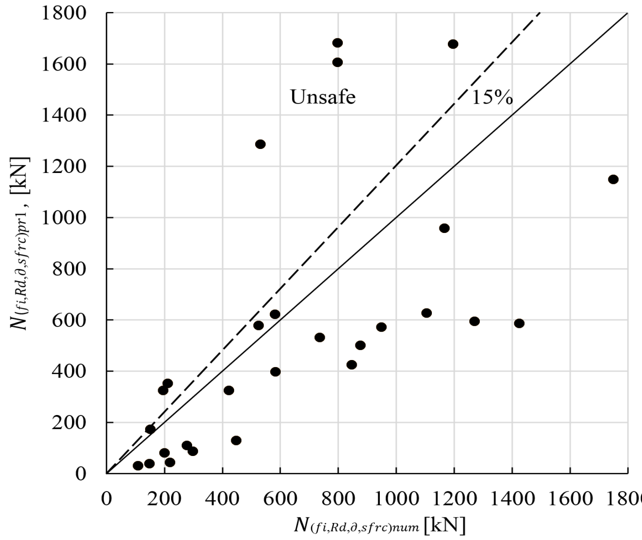


Fig. 12. Verification of the developed simplified design method for eccentrically loaded columns.

The $N^{pr\,2}_{fi,Rd,\partial,sfrc}$ values shown in Fig. 13 were obtained by applying the proposed reduction coefficient due to eccentricity for SFRC to all the eccentrically loaded cases considered in the sensitivity analysis.

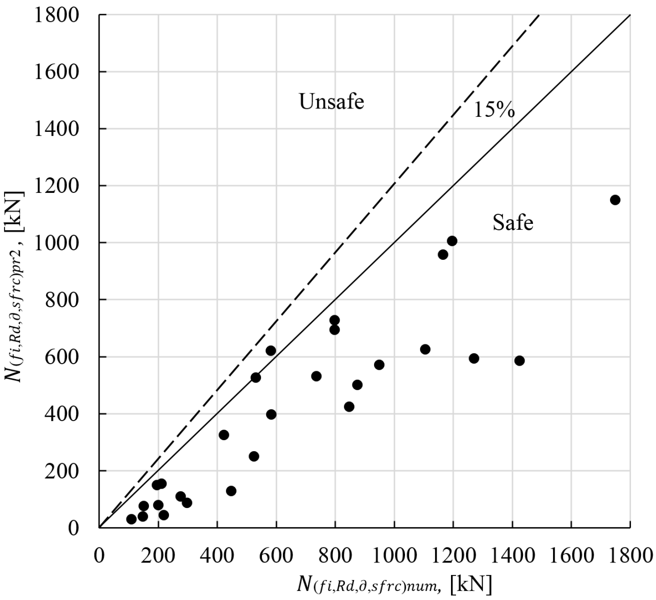


Fig. 13. Verification of the developed simplified design method for eccentrically loader columns after modification.

It can be observed that after the modification, safe predictions were obtained, indicating that the proposed model for the reduction coefficient due to eccentricity is now appropriate. The condition expressed in (14) is caused by the insufficient effect of a 0,5% fibre content in eccentrically loaded slender columns. Studies with a higher fibre content are needed. To refine the obtained formulas so as to obtain fewer conservative results, it is necessary to conduct additional numerical experiments.

D. Range of Application of the Proposed Method

The proposed design method (in audition to existing solution [16]) should be applied only to CHS columns with SFRC infilling under the conditions listed in Table IV.

TABLE IV: RANGE OF APPLICATION OF THE PROPOSED METHOD

No.	Condition
1	Section factor, cross-sectional slenderness, and member slenderness: $10 \leq Am/V \leq 25$ $27 \leq D/t \leq 68$ $4{,}6 \leq l_\theta/D \leq 24$
2	Only for columns with SFRC infill. The steel fibre content should not exceed 0,5%.
3	The concrete strength class should be limited between C30/37 and C50/60.
4	The relative load eccentricity e/D should not exceed 0,5.
5	The method can be used for specimens with a standard fire exposure time between 15 and 150 min.
6	$0{,}2 \leq \bar{\lambda}_\theta \leq 1{,}6$
7	For SFR if $\rho = 0\%$, then $\alpha_s = 1$

VI. CONCLUSION

This work is dedicated to developing an analytical model for determining the fire resistance of axially and eccentrically loaded composite SFRC columns subjected to ISO standard fire conditions. It was demonstrated that compared with plain concrete, SFRC infilling significantly improves the buckling load of CFST columns in a fire situation. The proposed model estimates the design buckling load of composite steel and fibre-reinforced concrete columns in a fire situation and can be used as an appendix or technical supplement to the prEN1994-1-2:2021 standard.

The three-dimensional FEM model built with the ATENA Červenka software [26] was used to conduct a sensitivity analysis, and its results served as a basis to propose a modification of the prEN1994-1-2:2021 European standard for the design of composite members exposed to fire.

The aforementioned FEM model was validated using experimental tests under a steady-state regime, confirming its applicability to the problem under investigation.

The developed analytical model determines the design buckling load of axially and eccentrically loaded composite steel and fibre-reinforced concrete columns in a fire situation.

The definition of the flexural stiffness reduction and reduction coefficients due to eccentricity allows the use of the proposed method within the specified range of application and its inclusion in the preparation of the prEN1994-1-2:2021 standard.

The proposed method is applicable for a fibre content of 0,5% and a relative load eccentricity of *e/D* < 0,5.

APPENDIX

This section describes a design example showing the application of the proposed design method for evaluating the R of SFRC CFST columns without rebars. The example corresponds to a standard fire period of 60 min (R60). The material and geometrical properties of the evaluated member are as follows:

Boundary conditions: P-P, $\beta = 1$
Column length *l*: 3700 mm
CHS cross-sectional dimensions: *D* = 273 mm *t* = 8 mm
Steel yield strength: f_y = 355 MPa
Concrete compressive strength: f_c = 30 MPa
Steel fibre content: 0,5%
Eccentricity:
e/D = 0,25

First, the cross-sectional slenderness, section factor, and member slenderness required for the application of the method are computed:

Section factor:
A_m/V=4/D=4/273×1000=14,65 m^{-1}.
Member slenderness in the fire situation:
$l_\theta/D = (\beta \cdot l)$/D=(1×3700)/273=13,55
Cross-sectional slenderness:
D/t=273/8=34,125

Next, the equivalent temperatures are calculated according to prEN1994-1-2:2021. However, this is beyond the scope of this study.

$$\theta_{c,eq} = 457{,}68\ °\mathrm{C}$$
$$\theta_{a,eq} = 889{,}09\ °C$$
$\theta_{s,eq}$- no rebars.

After the equivalent temperatures are calculated, the cross-sectional plastic resistance is easily obtained as (15).

$$N_{\mathrm{fi,pl,Rd}} = A_a f_y(\theta_{\mathrm{a,eq}}) + A_c f_c(\theta_{\mathrm{c,eq}}) = = 660{,}18 mm^2 \cdot k_y(889{,}09\ ^\circ\mathrm{C}) \cdot 355\ \mathrm{MPa} + 51874{,}76\ mm^2 \cdot k_c(457{,}68\ ^\circ\mathrm{C}) \cdot 30\ \mathrm{MPa} = = 6660{,}18 mm^2 \cdot 0{,}0655 \cdot 355 MPa + 51874{,}76\ mm^2 \cdot 0{,}6635 \cdot 30\ \mathrm{MPa} = 1187{,}31\ \mathrm{kN} \quad (1)$$

The partial factors of the materials in the fire situation $\gamma_{M,fi,i}$ are assumed to be unity, as per Clause 2.3 of prEN1994-1-2[18].

In the second stage, the flexural stiffness reduction coefficients are calculated using the equations and tables provided in Section VII of prEN1994-1-2:2021.

$$l_\theta / D = 13{,}55 > 12 \rightarrow \varphi_{a,\theta} = \varphi_{a,\theta,1}(A_m/V) \cdot \varphi_{a,\theta,2}(l_\theta/D) \cdot \varphi_{a,\theta,3}(D/t) \cdot \varphi_{a,\theta,3}(R) = 0.21 \quad (2)$$

$$\varphi^f_{c,\theta,simp} = 0{,}8 + 1{,}3237(l_\theta/D)^{-0{,}94} = 0{,}914 \quad (3)$$

Therefore, the effective flexural stiffness of the column in the fire situation is (18).

$$(EI)_{fi,eff} = \varphi_{a,\theta} E_a(\theta_{a,eq}) I_a + \varphi_{c,\theta} E_c(\theta_{c,eq}) I_c = \varphi_{a,\theta} \cdot E_a \cdot k_a \cdot I_a + \varphi_{c,\theta} \cdot \left(\frac{3}{2}\right) \cdot k_c \cdot \left(\frac{f_c}{\varepsilon_{cu,\theta}}\right) \cdot I_c = \ = 0.21 \cdot 210000\ \mathrm{MPa} \cdot 0.07 \cdot 58517142.6\ mm^4 + 0.914 \cdot \frac{3}{2} \cdot 0.6635 \cdot \left(\frac{30\mathrm{MPa}}{0.013}\right) \cdot 214142265.1\ mm^4 = 6.32 \cdot 10^{11} N \cdot mm^2 \quad (4)$$

The Euler buckling load and the relative slenderness at elevated temperature can be calculated as (19) and (20).

$$N_{fi,cr} = \frac{\pi^2 (EI)_{fi,eff}}{l_\theta^2} = 455.3 kN \quad (5)$$

$$\bar{\lambda}_\theta = \sqrt{\frac{N_{fi,pl,Rd}}{N_{fi,cr}}} = \sqrt{\frac{1187{,}31}{455.30}} = \sqrt{2{,}608} = 1{,}61 \quad (6)$$

Using buckling curve ‘‘a,” as corresponds to this case, a buckling coefficient $\chi = 0{,}328$ is obtained. Finally, the design buckling load of the column under axial compression after being exposed to fire for 60 min is given in (21).

$$N_{fi,Rd} = \chi \cdot N_{fi,pl,Rd} = 0{,}328 \cdot 1187{,}31 = 389.31\ \mathrm{kN} \quad (7)$$

The fire resistance of the same column loaded under a relative eccentricity of e/D=0,25 is now evaluated. Following the design rules in EN 1994-1-1 [12], the concentric and eccentric design buckling loads of the column at room temperature are computed (the calculation of these loads is out of the scope of this work):

$$N_{Rd} = 3401{,}9\ kN \quad (8)$$

$$N_{Rd,\delta} = 1871\ kN \quad (9)$$

The eccentric design buckling load in a fire situation is obtained using (24) and (25).

$$\frac{l_\theta}{D} = 13{,}55 < 15 \quad (10)$$

$$N_{fi,Rd,\delta} = \alpha \cdot \left(\frac{N_{Rd,\delta}}{N_{Rd}}\right)_{room} \cdot N_{fi,Rd} \quad (11)$$

Finally, taking the values of $\alpha_{Am/V}, \alpha_{D/t}, \alpha_R, \alpha_s$ from prEN1994-1-2:2021 [16] is given in (26)-(28).

$$\alpha_f = -0{,}0102 \cdot (13{,}55) + 1{,}1592 = 1{,}021 \quad (12)$$

$$\alpha = \alpha_{Am/V} \cdot \alpha_{D/t} \cdot \alpha_R \cdot \alpha_s \cdot \alpha_f = 0{,}573 \cdot 0{,}375 \cdot 2{,}816 \cdot 0{,}616 = 0{,}94 \quad (13)$$

$$N_{fi,Rd,\delta} = 0{,}94 \cdot \left(\frac{1872}{3401{,}9}\right) \cdot 389{,}31 = 201{,}11\ kN \quad (14)$$

ACKNOWLEDGMENT

We thank Czech Technical University in Prague for the help provided through grant SGS22/141/OHK1/3T/11, and to the Czech Science Foundation for its financial support to the project “Models of steel and fibre concrete composite columns exposed to fire” through grant GAČR 15-19073S.

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