NUMERICAL INVESTIGATIONS ON THE BEHAVIOUR OF REINFORCED CONCRETE COLUMNS EXPOSED TO FIRE

NUMERISCHE UNTERSUCHUNGEN ZUM VERHALTEN VON STAHLBETONSTÜTZEN UNTER BRANDBEANSPRUCHUNG

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SUMMARY

The paper presents a numerical study to investigate the response of concentrically loaded Reinforced Concrete (RC) columns exposed to design fire. The parameters investigated include three different design fires characterised by different heating & cooling phase and three different load levels between 25 - 55% of their axial load carrying capacity. It was observed that the maximum temperatures in the core of the column are always reached during the cooling phase. In terms of mechanical response, the results show that the maximum axial expansion (for lower load levels) also occurs during the cooling phase. These maximum expansions did not increase with fire duration. The results also show that there are certain combinations of loading level and fire exposure, where the columns may also fail during the cooling phase.

ZUSAMMENFASSUNG

Es wird eine numerische Studie zur Untersuchung des Verhaltens konzentrisch belasteter Stahlbetonstützen unter Brandbelastung durchgeführt. Die untersuchten Parameter umfassen drei verschiedene Brandszenarien, die durch unterschiedliche Aufheiz- & Abkühlphasen und drei unterschiedliche Belastungsniveaus, die zwischen 25% und 55% ihrer axialen Tragfähigkeit gekennzeichnet sind. Es wurde festgestellt, dass die Maximaltemperaturen im Kern der Stützen während der Abkühlphase immer erreicht werden. In Bezug auf das mechanische Verhalten zeigen die Ergebnisse, dass die maximale axiale Ausdehnung (für niedrigere Belastungsniveaus) auch während der Abkühlphase auftritt. Die maximalen Ausdehnungen nahmen nicht mit der Feuerdauer zu. Weiterhin, zeigen die Ergebnisse, dass es bestimmte Kombinationen von Belastungshöhe und Brandbelastung gibt bei denen die Stützen auch während der Abkühlphase versagen können.

KEYWORDS: Coupled thermo-mechanical model, design fire, columns, transient thermal analysis

1. INTRODUCTION

The fire rating of Reinforced Concrete (RC) structural members are defined only with respect to the heating phase of fire. The cooling phase is not considered by the current prescriptive design codes/standards. But in recent years with more emphasis being placed on the performance based design of RC structures under fire, predicting the response of RC members during the complete event of fire has gained importance.

Predicting the response of RC columns with axial load under design fire is relatively complex. This complexity arises due to the complex interaction between the degrading material properties, thermal strains, load induced thermal strains (LITS) and additional degradation of material properties during the cooling phase. The net axial deformation of column consists of the following components, which make the problem highly nonlinear:

1. Contraction due to the degrading material properties: This component is function of temperature and loading level.
2. Expansion due to thermal strain: This component is a linear function of temperature only. These strain components are reversible, i.e., upon cooling the thermal strains are recovered.
3. Contraction due to the LITS: This component is a nonlinear function of loading level and temperature. This component occurs only during the first heating cycle and is irrecoverable during the cooling phase.

The paper discusses the validation of a 3D sequentially coupled thermo-mechanical model capable of simulating the complex behaviour of axially loaded RC columns under fire. The model uses temperature dependent microplane model for concrete and classical von-Mises plasticity models for reinforcing steel. The paper also presents the results of the parametric study aimed at investigating the response of RC columns exposed to different design fire and different loading levels.

2. MODELLING APPROACH

In general, for simulating RC members under fire, it is sufficient to just consider the dependency of stress fields on temperature fields and temperature fields to be independent of the stress fields. Thus, a sequentially coupled numerical model is used for the presented study. The 3D thermo-mechanical model used for simulating the RC columns under fire for this study is briefly described in the following sections.

2.1 TRANSIENT HEAT TRANSFER ANALYSIS

As the first step of coupling between mechanical properties of concrete and temperature, for the given thermal boundary conditions at time *t* temperature distribution over a solid structure of volume Ω has to be calculated. In each point of continuum, which is defined by the Cartesian coordinates (x, y, z), the conservation of energy has to be fulfilled. This can be expressed by the following differential equation (Eq. (1)):

$$\lambda \Delta T(x,y,z,t) - c\rho \frac{\partial T}{\partial t}(x,y,z,t) = 0 \tag{1}$$

Where, T = temperature, λ = conductivity, c = heat capacity, ρ = density and Δ = Laplace-Operator. The surface boundary condition that has to be satisfied is given by Eq. (2):

$$\lambda \frac{\partial T}{\partial n} = \alpha (T_M - T) \tag{2}$$

Where, n = normal to the boundary surface Γ, α = equivalent heat transfer coefficient (given by Eq. (3)) and T_M = temperature of the media in which surface Γ of the solid Ω is exposed to (in present case temperature of air). The above mentioned differential equations are solved using finite element method.

$$\alpha = h_c + \varepsilon\sigma\left[(T_M)_t^2 + (T)_{t-1}^2\right] \times \left[(T_M)_t + (T)_{t-1}\right] \tag{3}$$

The equivalent heat transfer coefficient given by Eq. (3) is used to account for the convective and radiative heat transfer from the hot gases to the member surface, where h_c is the convective heat transfer coefficient, ε is the surface emissivity and σ is Stefan Boltzmann constant.

The thermal properties of concrete are taken from Eurocode2 [1] based on the sensitivity/validation studies conducted by Lakhani et. al., (2013) [2]. Lower bound thermal conductivity and specific heat for dry concrete were used for the presented numerical results.

2.2 MATERIAL CONSTITUTIVE LAWS

Concrete is a complex material to model at high temperatures. This is not only because of the dependency of various mechanical properties on temperature but also due to the additional strain components that appear due to temperature. The total strain tensor for concrete, given by Eq. (4), has three components: mechanical strain, free thermal strain and load induced thermal strain. The mechanical strain component is further composed of elastic, plastic and damage part. For the numerical study the temperature dependent microplane model is used as material constitutive law for concrete. In the microplane model the material is characterised by the relation between the stress and strain components on planes of various orientations. These planes may be imagined to represent the damage planes or weak planes in the microstructure, such as those that exist at the contact between aggregate and the cement matrix. The microplane model used in the present study was proposed by Ožbolt, et. al., (2001, 2005) [3, 4].

$$\varepsilon_{ij} = \varepsilon_{ij}^{m}(T,\sigma) + \varepsilon_{ij}^{ft}(T) + \varepsilon_{ij}^{lits}(T,\sigma) \tag{4}$$

Where, ε_{ij}^{m} = mechanical strain tensor; ε_{ij}^{ft} = free thermal strain tensor and ε_{ij}^{lits} = load-induced thermal strain tensor.

The temperature dependencies of various mechanical properties of concrete are shown in Fig. 1. For further details, readers may refer to Ožbolt et al., (2005) [4] and Periskic, (2009) [5]. The model considers that the degradation of mechanical properties of concrete is irrecoverable during cooling. The free thermal strains are assumed to be isotropic and recoverable upon cooling.

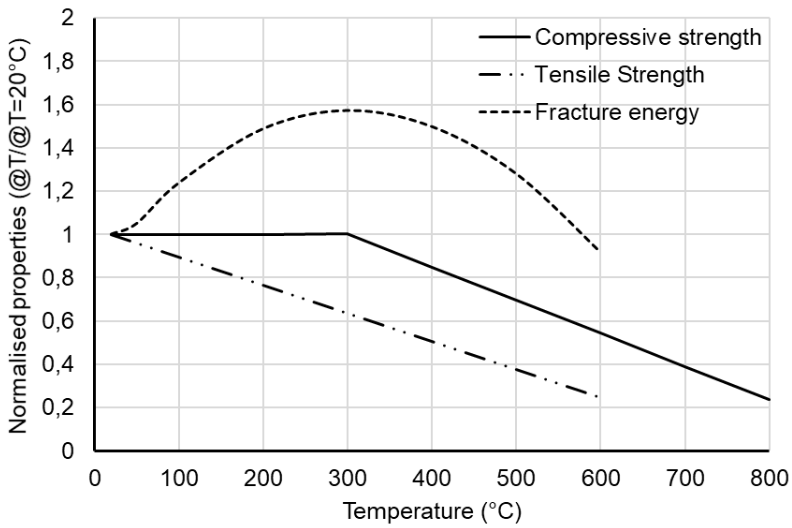


Fig. 1: Degradation of mechanical properties of concrete with temperature

Steel is modelled using temperature dependent classical plasticity model (von-Mises). Fig. 2 shows the variation of Young’s modulus, proportionality limit and yield stress with temperature, used in the model. The strength recovery after cooling was considered based on the experimental observations of Takeuchi et. al., (1993) [6], i.e., steel strength is completely recovered up to a temperature of 500°C and 80% recovered at 800°C.

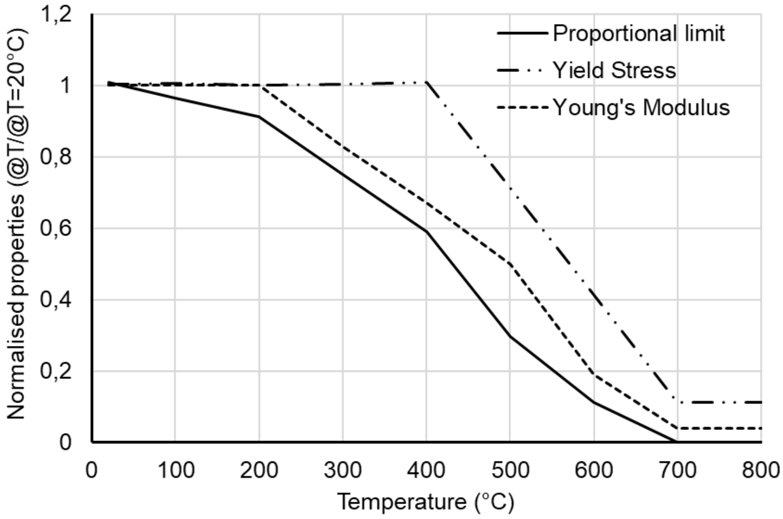


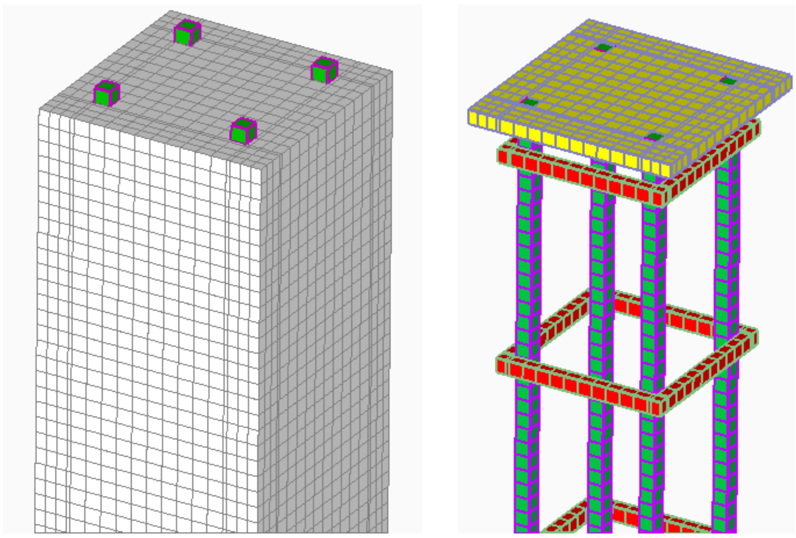
Fig. 2: Degradation of mechanical properties of reinforcing steel with temperature

3. VALIDATION

In order to validate the modelling procedure, one of the RC column tested by Lie et al., (1986) [7] was numerically simulated. The experiments performed by Lie et al (1986) consisted of 2 full scale RC columns referred to as Col-A & Col-B. Col-A was simulated to validate the model. The column had a total length of 3.81 m with a fire exposed length of 3.04 m. Column had a cross-section of 305 × 305 mm, with 4 - 25 dia (#8) as longitudinal reinforcement and 9 dia (#3) rebars at 305 mm c/c spacing for stirrups. The longitudinal reinforcement had a clear cover of 48 mm. The column was fixed at the bottom end and the top end was fixed against in-plane translations but was free to have axial translation only. The cylinder strength of the concrete on the day of testing was 38.9 MPa. The yield stress of the main reinforcement bars was 444 MPa and that of the stirrups was 427 MPa.

A concentric axial load of 992 kN was applied on the Col-A before exposing it to fire. The load was maintained constant throughout the fire test and axial displacement at the top end of the column was measured. The Col-A was exposed to a design fire with a heating phase of 60 minutes (standard temperature-time cure as per ASTM E-119) followed by cooling at the rate of 500°C/hr.

The column geometry is discretised used 8-noded solid element with three translational degree of freedoms at each node. The longitudinal reinforcement and stirrups are also modelled using solid elements. The average element size used was ≈ 20mm. The reinforcement is assumed to be perfectly bonded to concrete. A linear steel plate with 25 mm thickness was modelled at each end of the column to apply the boundary conditions and axial load. Fig. 3 shows the meshing for concrete, reinforcement and linear end plate.



(a) Concrete mesh *(b)Reinforcement mesh*

Fig. 3: Geometric discretization of Col-A

The predicted and experimental temperatures at two thermocouple (TC) locations across the column cross-section at its mid height are shown in Fig. 4. It can be seen that the predictions are in good agreement with the experimental observations. It can also be observed that the peak temperatures occur during the cooling phase of the design fire. Farther the point is from the exposed surface, later is the peak temperature achieved. This is due to the high thermal inertia of concrete.

Fig. 5 shows the comparison between the predicted and experimental mechanical response of Col-A. It should be noted that the deformed state of column after the preload is taken as the reference for the measured axial deformation. The column expands during the heating phase and continues expanding for some time during the cooling phase as well. Once again, as for temperatures, the maximum expansion of column also occurs during the cooling phase. After the column has reached its maximum expansion, it starts to contract due to the recovery of (linear) thermal strain and partially due to addition damage incurred by concrete during cooling.

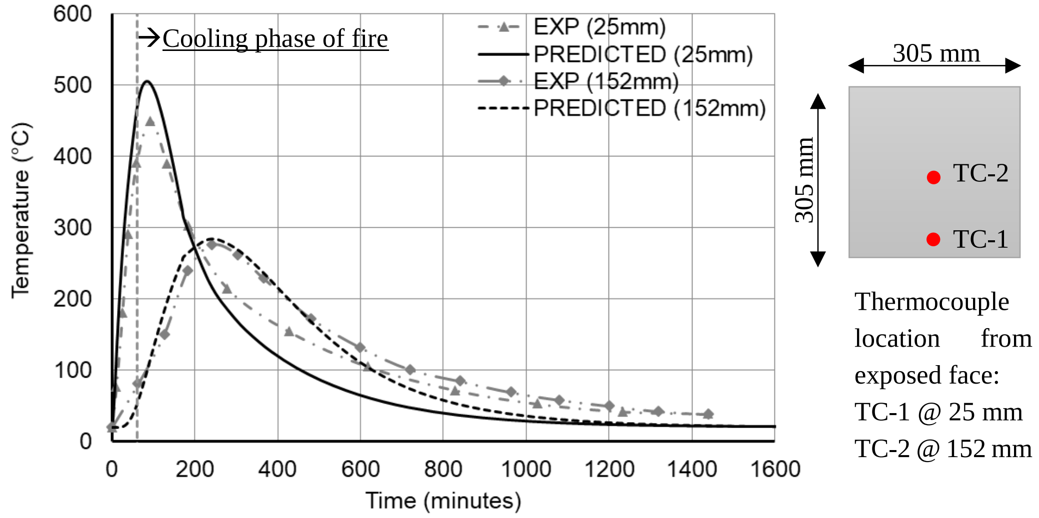


Fig. 4: Comparison between the predicted and experimental temperatures for Col-A

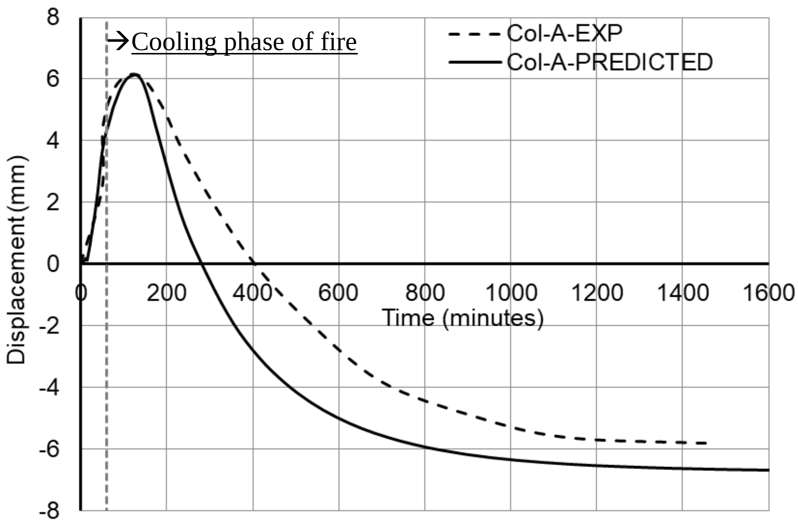


Fig. 5: Numerical and experimental variation of axial displacement with time for Col-A [8]

Since, the numerically predicted thermal and structural (mechanical) response of Col-A are in good agreement with experiments. It can be concluded that the adopted numerical model can successfully simulate the thermal and mechanical response of RC column under design fire.

4. PARAMETRIC STUDY

After having validated the numerical model, parametric study is conducted on Col-A to further investigate the response of RC columns under 3 different Design Fire (DF) and 3 different Load Levels (LL). The 3 design fire scenarios considered (as shown in Fig. 6) are characterised by different duration of heating phase (60 min; 90 min and 120 min) and cooling rates (500°C/hr; 375°C/hr and 250°C/hr). The three load levels considered were 992 kN, 1450 kN and 2100 kN, ranging between 25-55% of ultimate axial load capacity of the column.

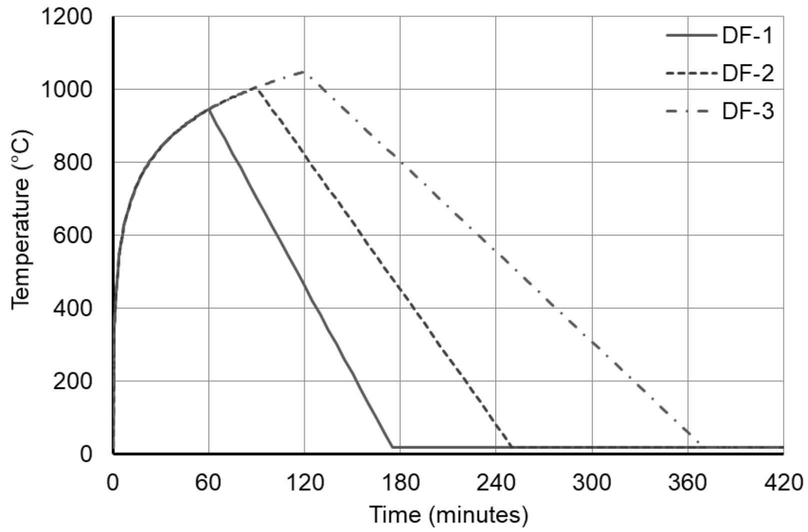


Fig. 6: Various design fire exposures investigated

Fig. 7 shows the predicted temperatures at location TC-2 (refer to Fig. 4 for TC-2 location), which is also the centre of the column cross-section. It can be observed from Fig. 7 that the peak temperatures not only increase but also occur later during the cooling phase with increase in the total fire duration (heating phase + cooling phase). The maximum core temperatures for column exposed to DF-1, DF-2 & DF-3 occur at 235 min, 310 min & 370 min, respectively.

It was observed that for load levels of 992 kN and 1450 kN, the column first expands, reaches a maximum expansion before contracting and finally stabilises at residual deformation, for all design fires. But for load level 2100 kN, the column directly starts to contract and stabilises at a residual deformation. The variation of maximum axial expansion for different loading levels and design fires is shown in Fig. 8. The results show that for lower load level (992 kN) the axial expansion reduces with increasing fire duration, which means that the total axial contraction due to degradation of mechanical properties and LITS, is more than the expansion due to thermal strain. It can also be seen that for load level 1450 kN the maximum expansion is independent of the heating phase duration.

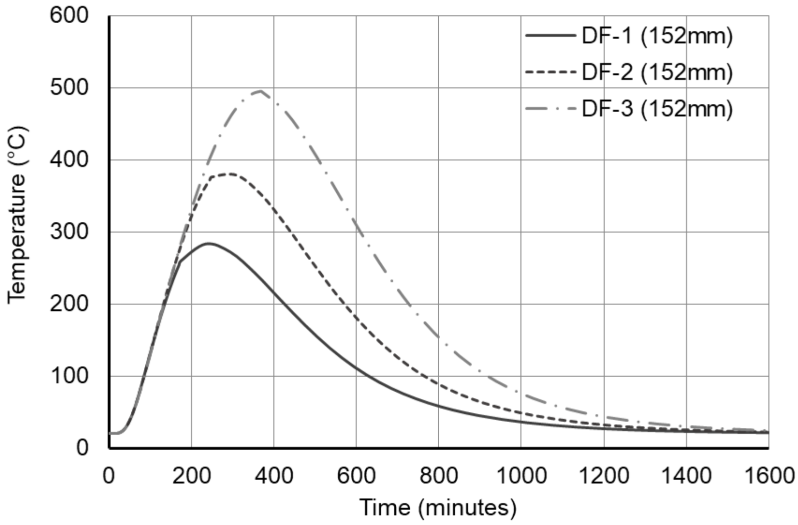


Fig. 7: Temperature variation at the column centre for different design fires

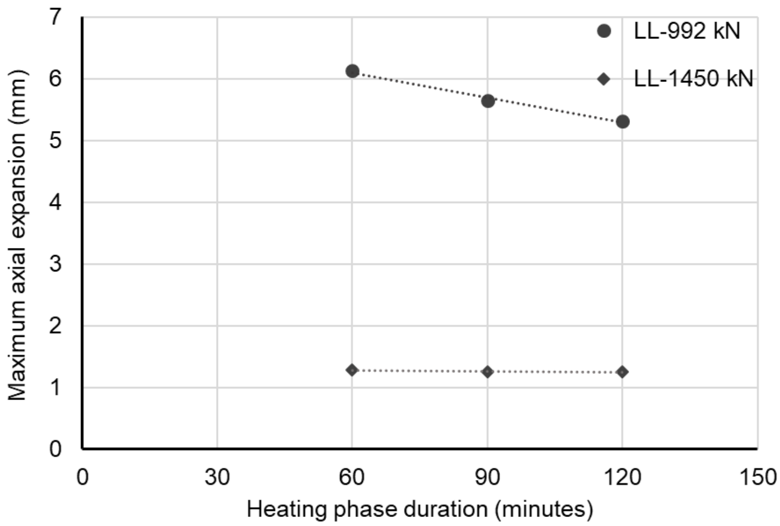


Fig. 8: Axial deformation with time curves for different design fire exposures

The mechanical response (as axial deformation-time curves) of the column with axial load 1450 kN and 2100 kN for different design fires is shown in Fig. 9 and 10, respectively. The residual axial deformation was found to increase with increasing loading level and increasing duration of the heating phase. The responses for load levels 1450 kN and 2100 kN are shown to emphasise the importance of cooling phase of fire. It was observed that column failed during the cooling phase for certain combinations of loading and design fires. These cases are summarized in Table 1.

Table 1: Summary of cases with failure during cooling phase

S. NO.	LOAD	FIRE EXPOSURE	FAILURE TIME
1	1450 kN	DF-3 (120 min heating phase + cooling @ 250°C/hr)	180 min
2	2100 kN	DF-2 (90 min heating phase + cooling @ 375°C/hr)	200 min
3	2100 kN	DF-3 (120 min heating phase + cooling @ 250°C/hr)	150 min

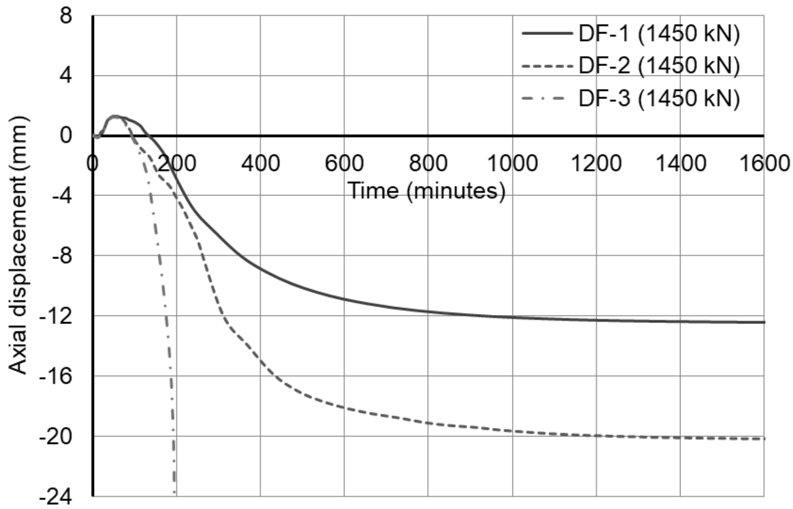


Fig. 9: Axial deformation with time curves for column with axial load of 1450 kN

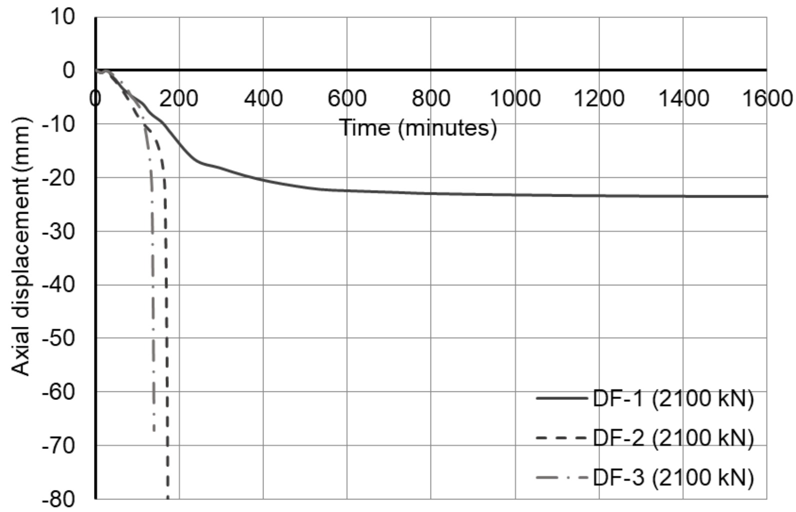


Fig. 10: Axial deformation with time curves for column with axial load of 2100 kN

5. CONCLUDING REMARKS

In this paper the response of concentrically loaded RC columns under 3 different design fires with 3 different loadings is discussed. The validation of the sequentially coupled 3D thermo-mechanical model based on microplane model for concrete and classical von-Mises plasticity model for reinforcing steel, has been presented. Based on the validation it was concluded that the numerical model is capable of simulating the behaviour of RC columns exposed to design fires. The predicted thermal and mechanical responses of RC columns are in good agreement with the experimental results.

Based on the numerical results of the parametric study, the following conclusions can be drawn:

1. Due to the high thermal inertia of concrete the peak temperatures across the column cross-section occur during the cooling phase.
2. The peak axial expansions (for lower loadings) occur during the cooling phase, thus emphasising on the importance of considering a fire scenario with cooling phase.
3. The axial expansion of column may not necessarly increase with increase in the duration of heating phase. It is dependent on the complex interaction between the degrading mechanical properties, thermal strains and load induced thermal strains.
4. The column may also fail during cooling phase for certain combinations of loadings and design fire scenarios.

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