NUMERICAL SIMULATION OF REINFORCED CONCRETE FRAME STRUCTURAL FIRE RESPONSE

Igor Džolev[1]
Đorđe Lađinović[2]
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Summary: *Reinforced concrete (RC) structures exhibit complex behaviour when subjected to fire. Severe thermal action evokes changes in the material microstructure and thermal-hydral-mechanical properties, depending on the heating rate, moisture, boundary conditions, geometry and size of the heated member, loading type, chemical-physical interactions, etc. Extensive experimental material research has led to the development of mathematical models and numerical procedures that could, to a certain degree, capture adequately the behaviour of structures under fire conditions. Advanced modelling guidelines have been proposed in standards, such as Eurocode. Based on these recommendations and previous efforts conducted by other researchers, a numerical model is developed in finite element (FE) software ANSYS. The model incorporates temperature dependent physical, thermal and mechanical properties of constituting materials and conducts nonlinear heat transfer and structural analysis, simulating the response of RC frame structure under standard fire action. Explicit modelling of concrete and steel reinforcement allows monitoring of temperature evolution in both concrete and reinforcement elements, deformations, section forces and stresses and strains in reinforcement bars, providing a broad insight into the structural behaviour at both global and local level. Special consideration is given to the influence of fire scenario on RC frame behaviour.*

Keywords: *fire resistance, heat transfer, structural fire analysis, numerical modelling*

1. INTRODUCTION

In case of fire, structure is exposed to severe external flame temperatures causing changes in the physical and mechanical properties of the constituting materials. The heat transfer first occurs between the fluid (hot gas) and the boundary surfaces of the exposed members by means of convection and radiation. Once the heat penetrates the boundary, it is further transferred through the member by means of conduction. Since the structure is already in a state of equilibrium caused by external actions at the time the fire starts, the increase of temperature affects the stress and strain development in the members, regardless of the loading remaining constant during fire. The heat transfer process is non-stationary, with the gas temperature evolving rapidly through time. Fire severity

[1] PhD, Faculty of Technical Sciences, University of Novi Sad, Serbia, e – mail: idzolev@uns.ac.rs
[2] PhD, Faculty of Technical Sciences, University of Novi Sad, Serbia, e – mail: ladjin@uns.ac.rs

depends on a number of factors, including the availability of combustible materials, ventilation conditions and physical characteristics of the space in which fire is initiated [1]. In order to establish a standardized comparable method for determining the fire resistance of members, the fire action is commonly determined using standard ISO 834 [2] temperature-time fire curve. The function is characterised by temperature-time formulation, providing hot gas temperatures in the compartment dependent only on one variable - time. Temperature is assumed uniform within the fire sector, which obviously presents a crude approximation, but is deprived of the uncertainties of various factors affecting real fire development. Also, as monotonically increasing function, standard fire curve does not take into account the cooling phase of the real fire development.
Depending on the dominant structural material, design of structures for fire action is to-date mostly based on simplified methods, which are, to a more or less extent, reliable. Performance based approach is also permitted, providing a high level of requirements regarding numerical modelling. Only a relatively small number of computer codes are currently capable of simulating complex processes of structural fire behaviour. These codes are generally divided into two major categories: specialized software, such as SAFIR [3], OpenSEES [4], VULCAN [5] and commercial software, such as ANSYS [6], ABAQUS [7], etc. While specialized software are being developed at the universities and research centres, and are optimized in terms of computational performance, they are often limited by the complexity of geometry and the type of fire exposure that can be analysed, unlike generalized commercial codes, which are more robust, but also, computationally more demanding. When analysing structures in fire, numerical models should be able to provide realistic analysis, leading to a reliable approximation of the expected behaviour. In general, nonlinear numerical analysis has to be conducted, taking into account changes in material properties at elevated temperatures, both thermal and mechanical, as well as the nonlinear temperature distribution at the element cross-section level.
In the present study, numerical model is developed in ANSYS for the purpose on analysing the behaviour of reinforced concrete (RC) frame structure subjected to standard fire. Modelling procedure, as well as assumptions and restrictions for further application are discussed. Special consideration is given to the influence of fire scenario, in terms of the position of the fire in the building, and the response is monitored at both global and local level.

2. NUMERICAL MODELLING

In the proposed model in ANSYS, thermal and stress analyses are sequentially coupled. First, thermal calculation is carried out for the entire duration of the fire, after which the structural analysis is performed, taking into account the temperature variation in time, through the imported internal body temperature of the members.

2.1. Thermal model

Fire analysis should be conducted based on fire risk assessment, aiming at identifying and prioritising potential risks for fire ignition and development. Generally, during the design stage, buildings are divided into fire compartments, physically separated by members (slabs, walls), whose roles are to contain the fire from spreading from one part of the building to another. These members are not always load bearing. Sometimes, their only purpose is to assure compartmentation. Depending on the member's function, different fire resistance criteria need to be fulfilled (R - load bearing, E - integrity, I - insulation [8]), resulting in a fire resistance classification of members, which is based on standard fire exposure. The classification should clearly indicate the type of criteria the analysed member meets and the minimum time this type of resistance should be guaranteed. This is only valid for standardised fire exposure (Figure 1). Using performance based design approach, fire resistance should be determined by monitoring performance criteria and assuring fire resistance for the whole duration of fire, calculated based on real properties of the analysed fire sector.

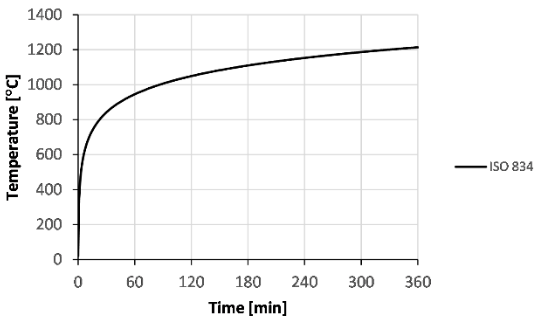


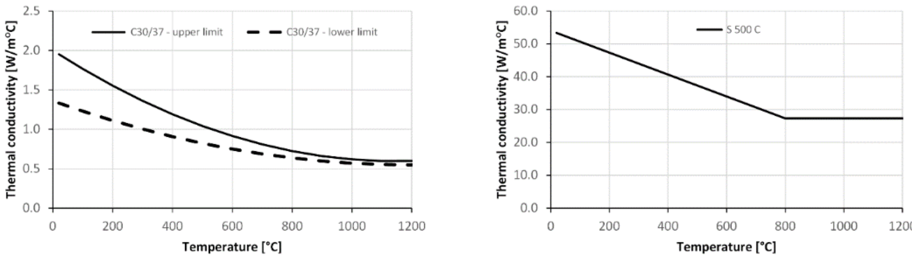
Fig. 1. Temperature-time evolution of the standard ISO 834 fire curve [9]

In order to account for a temperature distribution in the exposed members, transient thermal analysis needs to be conducted. Heat transfer should be calculated taking into account conduction, convection and radiation.

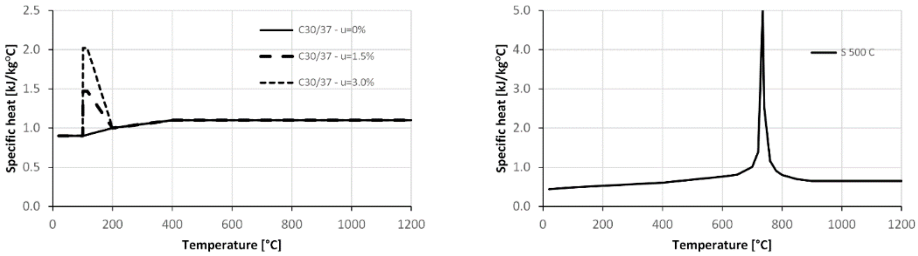
Complex behaviour that occurs in the material at elevated temperatures can implicitely be modeled by implementing temperature-dependent properties of concrete and steel, namely thermal conductivity, specific heat and density, which can be obtained using experimental testings [10] or recommendations provided in codes for advanced numerical procedures, such as the Eurocodes dealing with structural fire design (Part 1-2 series). For the case of reinforced concrete structures, [8] provides these relations for the temperature range expected during fire (Figure 2). Generally good fire resistance of RC members, in comparison with, for example steel members, is mainly due to concrete constituent materials, which form an essentially inert material with low thermal conductivity and high heat capacity. Comparing the thermal properties of concrete and steel, it can be seen that concrete has relatively low thermal conductivity, acting as an insulation for the reinforcement, and protecting it from a direct fire exposure, by the thickness of the concrete cover (assuming full cover integrity during fire). The rate of heat transfer from the exposed surface to inner layers is measured by the thermal diffusivity. Lower thermal diffusivity of concrete results in slower temperature rise at a certain depth in the material.

To consider heat transfer due to convection and radiation, a convective heat transfer coefficient and emissivity of the exposed materials should be determined, respectively. For the standard fire exposure and concrete material, these can be obtained from [8] and [9] as $\alpha_c = 25$ W/m^{2}K and $\varepsilon = 0.7$ for the exposed surfaces and $\alpha_c = 4$ W/m^{2}K and $\varepsilon = 0.7$

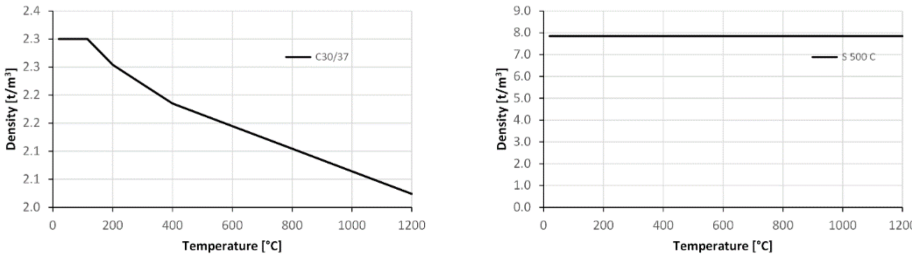
for the unexposed. According to [9], radiation on the unexposed side can be omitted, but in that case, the convective heat transfer coefficient should be adopted as $\alpha_c = 9$ W/m^{2}K.



a1) Thermal conductivity of concrete *a2) Thermal conductivity of steel*



b1) Specific heat of concrete *b2) Specific heat of steel*



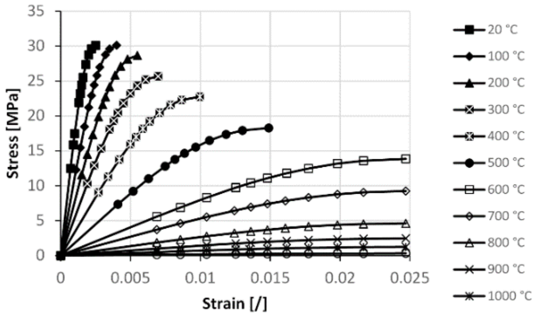
c1) Density of concrete *c2) Density of steel*

Fig. 2. Thermal properties of concrete and steel at elevated temperatures [8]

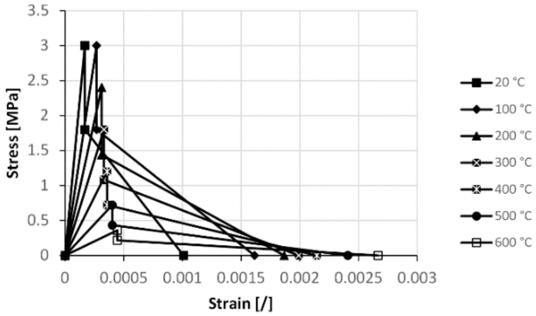
In both thermal and structural analysis, concrete geometry is modelled using solid bodies, while reinforcement is modelled using line bodies. For the thermal analysis, geometry is further discretized using 8-node solid70 brick element and a uniaxial 2-node link33 line element, for concrete and reinforcing steel, respectively, with a single degree of freedom at each node, temperature. The time step for the iterative calculation procedure should be chosen adequately, to avoid the appearance of space oscillations of the solution [11], depending on the material thermal properties and the size of the finite element in the dominant direction of the heat transfer. Taking this into account, the mesh element size is adopted as 1.25 cm. A tie constraint is used to apply temperatures from concrete to reinforcing bars at the coinciding node locations. Convection and radiation are applied on both exposed and unexposed surfaces of the members using surf152 planar elements.

2.2. Structural model

The constitutive material models for concrete and steel are adopted according to [8]. Proposed stress-strain relations for concrete implicitly account for thermal creep effects and are applicable for heating rates between 2 and 50 K/min. Concrete is modelled using nonlinear constitutive concrete material model of William and Warnke [12], combined with multilinear isotropic hardening plasticity, following stress-strain curves provided in [8]. The model is constructed based on an elastic perfectly plastic formulation which is augmented by a brittle failure condition in tension. In both cases it is assumed that the normality principle determines the direction of the inelastic deformation rates for ductile as well as brittle post failure behavior. Cracking is permitted in three orthogonal directions through modification of the stress-strain relations by introducing the plane of weakness in a direction normal to the crack face. The isotropic stiffness matrix is replaced by an orthotropic matrix upon crack formation. Stress normal to the crack direction is softened until zero value is reached. The open and close crack shear transfer coefficients, β_t and β_c, representing the amount of shear force transferred through opened and closed crack are adopted as 0.4 and 0.8, respectively (0 indicating smooth crack with a total loss of shear transfer and 1 indicating rough crack without any loss of shear transfer). Tensile behaviour of concrete before cracking is assumed to be linear elastic. When the tensile strength is exceeded, the stress is suddenly reduced to 60% of the tensile strength, after which it linearly descends to zero at six times the strain value at maximum tensile strength (Figure 3). Once cracks form, they can close and reopen, but their directions remain fixed. If the material at an integration point fails in uniaxial, biaxial, or triaxial compression, it is assumed to crush at that point, leading to a complete deterioration of the structural integrity of the material, with a neglected contribution to the stiffness of an element at the integration point. Although this kind of material modeling represents the concrete behaviour more realisticly, it introduces severe numerical convergence problems which could be very difficult to overcome, often acquiring crushing of the elements to be omitted in the analysis. The nonlinear behavior of concrete can be accounted for with the addition of plasticity model, such as multilinear isotropic hardening plasticity (MISO), which could be used to define stress-strain relations prior to crushing. The stress-strain uniaxial response is defined through a series of points, where the slope of each subsequent line consisting of two consecutive points, needs to be smaller than the previous, but has to remain positive. If crushing capability of the material model is omitted, after reaching its maximum, the compressive strength is kept constant with further straining, providing perfectly plastic response. The nonlinear definition, thus, can be defined up to the maximum compressive strength, without the capability of modelling the descending branch of the stress-strain response. The material reference of this model [13] suggests that if cracking or crushing capabilities are present, geometric nonlinearities, such as large strain, large deflection and stress-stiffening effects, should not be considered in the analysis. For many reinforced concrete applications, the absence of these effects would not influence the structural response, but in some cases, where deformations and stresses can lead to second order, *P-D* effects, the usability of this model would be questionable.



a) Stress-strain relations for concrete in compression



b) Stress-strain relations for concrete in tension

Fig. 3. Temperature-dependent stress-strain relations for concrete

This concrete model is only applicable if the geometry is discretised using 8-node solid65 brick elements. These elements can also be used to account for reinforcement, in an implicit way, by determining the real constant, which smears the stiffness of reinforcement over the concrete element. This approach could be more favourable if the aim of the analysis is directed towards determining the global response of the structure. If, however, the aim is to determine cloesely the development of temperatures, stresses and strains in reinforcement bars, steel elements should be modeled explicitly. This discrete approach implies that each reinforcement bar should be modelled as a separate body. Reinforcement bars, both longitudinal and transverse, are modelled as a uniaxial 2-node link180 line elements, with 3 translational degrees of freedom at each node. The nodes of the reinforcement and the surrounding concrete elements coincide and are coupled. Multilinear isotropic hardening plasticity model is assumed, since large strains are expected to develop, following stress-strain definitions according to [8] (Figure 4).

Mechanical properties of concrete and steel used to generate nominal stress-strain-temperature relations are converted into true stress-strain curves, to account for dimensional changes and to provide more realistic representation of the material behaviour [14], using following relations:

$$\sigma_{true} = \sigma_{nom}(1+\varepsilon_{nom}) \qquad \varepsilon_{true} = \ln(1+\varepsilon_{nom}) \qquad (1)$$

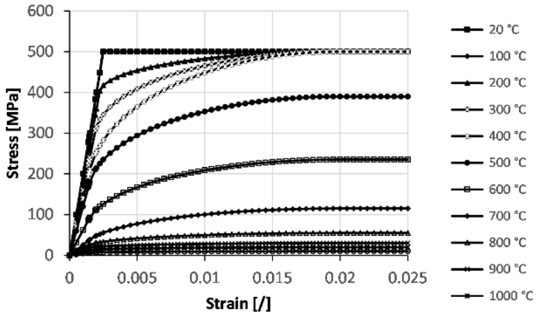
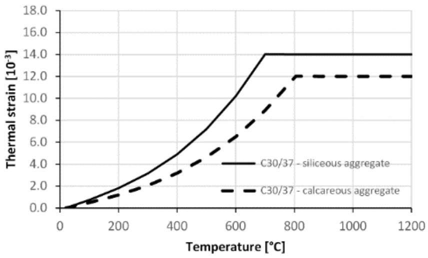
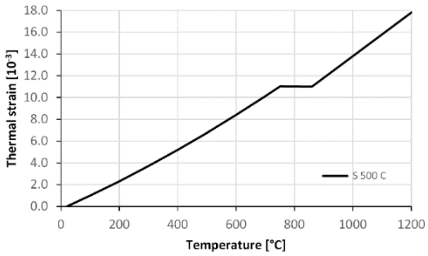


Fig. 4. Temperature-dependent stress-strain relations for steel

Thermal strains of both concrete and steel are nonlinear with regards to the temperature [8] (Figure 5) and are significantly affecting the magnitude of deformations of members during fire.

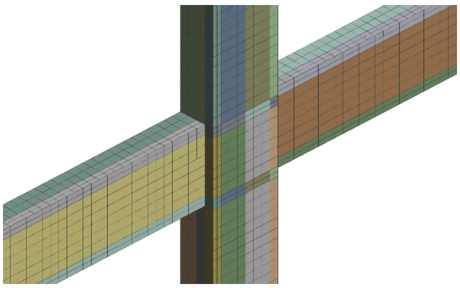


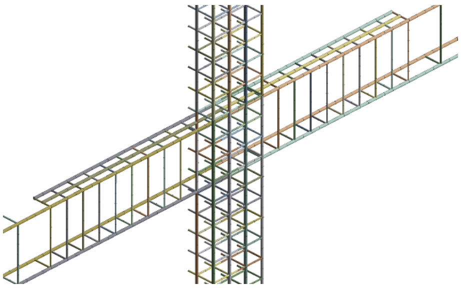


a) Thermal strain of concrete *b) Thermal strain of steel*

Fig. 5. Thermal properties of concrete and steel at elevated temperatures [8]

In general, mesh size does not have to be the same for thermal and structural model, and for structural model is adopted as 5.0 cm, except in the regions of severe thermal gradients (concrete cover), where the element size is reduced to 2.5 cm (Figure 6).





a) Concrete mesh *b) Reinforcement mesh*

Fig. 6. Structural model FE mesh of the beam-column joint

2.3. Restrictions and limitations

The thermal-structural analysis is not fully coupled. Thermal response is calculated on undeformed geometry, independent on the mechanical response during heating. The main drawback of this method is the inability to predict concrete spalling, which is especially significant in the first 30 minutes of fire, when high temperature gradients and increase in pore pressures could lead to chunks of concrete being detached from the members, exposing the reinforcement directly to fire. While spalling can have significant effect on fire resistance, it is less pronounced for ordinary performance concrete with low moisture content assumed [15].
Bond-slip between steel reinforcement and concrete is not taken into account. Perfect bond is assumed, resulting in equal total strain in the reinforcement and concrete in the contact region.
Transient creep strain in concrete is modelled implicitly, according to [8]. Since cooling phase is not considered, this would not affect the overall behaviour of the structure.

3. RC FRAME STRUCTURE

A three-storey two-bay RC frame structure, designed for ductility class M and a peak ground acceleration of 0.2 g, according to [16] is subjected to standard ISO 834 fire, covering the entire floor in a certain fire scenario. Given the structural regularity and the symmetries in terms of geometry and loading conditions, only 1/4 of the central plane frame is analysed. Fire analysis, as accidental load combination, accounts for total permanent load and 50% of the imposed load [17]. Fire limit state load, model geometry and adopted reinforcement for a central frame are presented in Figure 7. Concrete grade C30/37 mixed with siliceous aggregate and reinforcement S500 type C, are considered in the design.

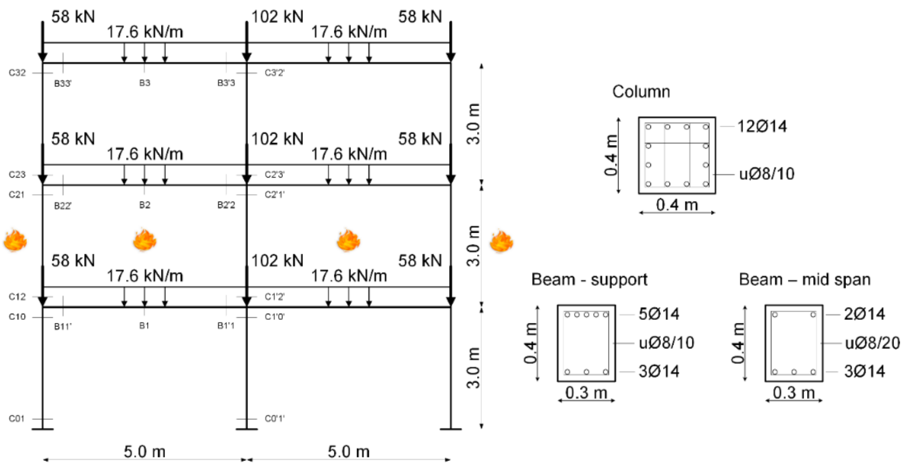
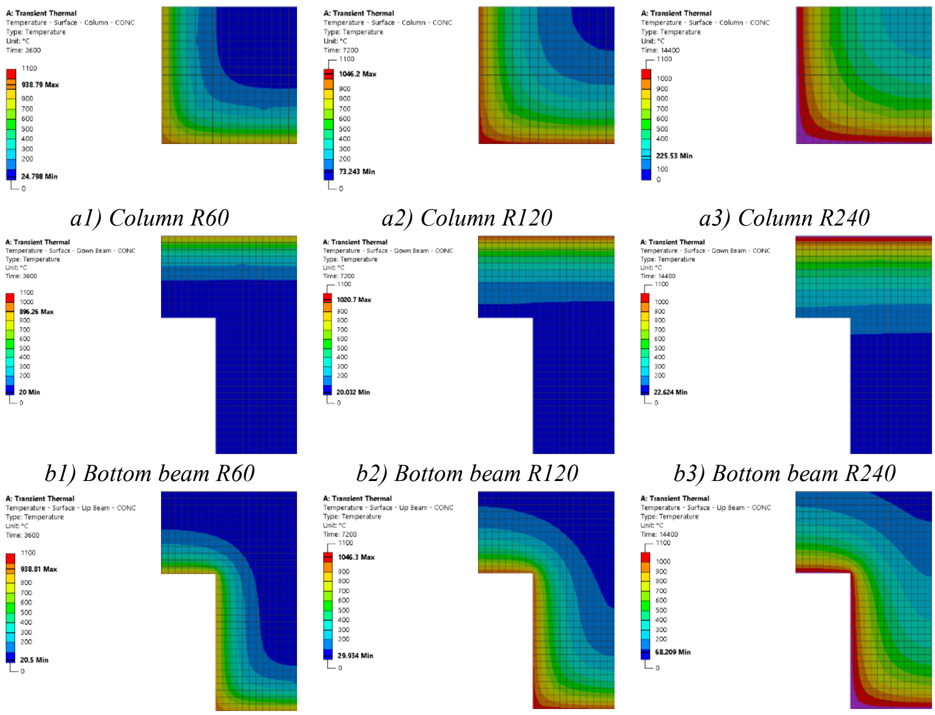


Fig. 7. Geometry, boundary and loading conditions

To account for a more realistic heat transfer, a part of the RC slab with a thickness of 15 cm is modelled in the thermal analysis. Even if neglected in the subsequent structural analysis, the slab presents physical barrier and prevents vertical fire spread in the building, as well as protects the beams in case of side exposures by its thickness. As a surface element, the insulation criteria [8], assuring that fire will not spread to the surrounding sector by heat transfer, can be determined by monitoring the temperature rise on the unexposed slab surface.

4. THERMAL AND STRUCTURAL RESPONSE

Temperature profiles in concrete members at the cross-section level and temperature histories of main reinforcement bars are presented in Figures 8-9. The insulating property of concrete cover is evident (assuming spalling does not occur) and can be highlighted by comparing the concrete surface temperatures and reinforcement temperatures. In the first 30 minutes of fire exposure, thermal gradient in the concrete cover is the highest, resulting in large tensile stresses induced by thermal expansion that lead to severe cracking of concrete. Temperature of the main reinforcement bars positioned in the unexposed zone of the beam members gradually increases, but remains relatively low, assuring full load bearing capacity of reinforcement.

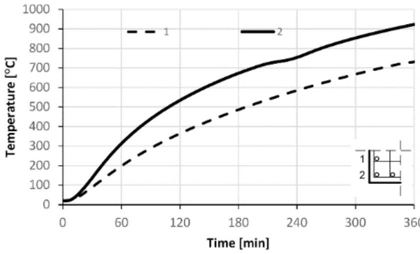


a1) Column R60 *a2) Column R120* *a3) Column R240*

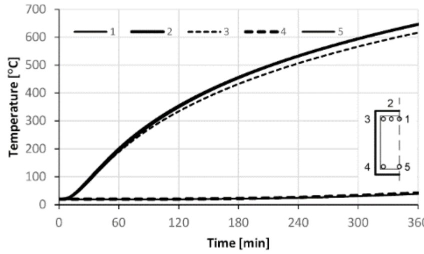
b1) Bottom beam R60 *b2) Bottom beam R120* *b3) Bottom beam R240*

c1) Top beam R60 *c2) Top beam R120* *c3) Top beam R240*

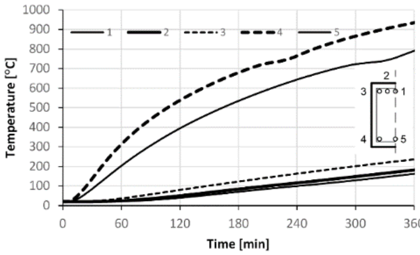
Fig. 8. Temperature profiles of exposed columns and beams



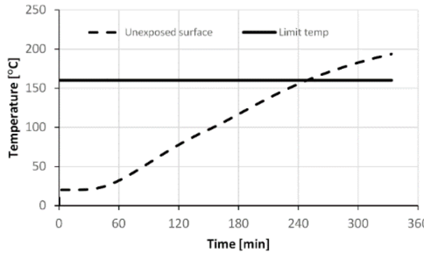
a) Column



b) Bottom beam



c) Top beam

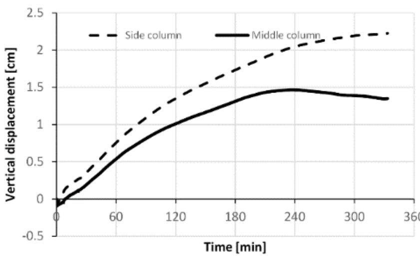


d) Unexposed slab surface

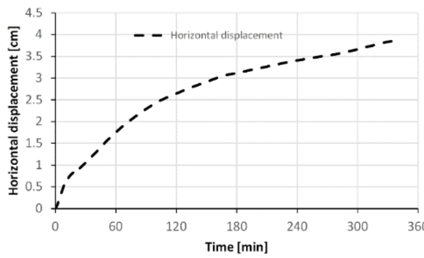
Fig. 9. Temperature evolution in reinforcement and on unexposed slab surface

Temperature rise at the unexposed slab surface assures 247 minutes of insulation criterion, requiring average temperature rise on the unexposed size to be limited to 140°C [8].

Increase in deformations of RC members during fire is governed by the thermal expansion and degradation of material strength and stiffness. Columns initially extend upwards due to thermal expansion, but as the strength and stiffness are reduced with higher temperatures, after 240 minutes, middle column vertical thermal expansion becomes less dominant than the deformation from the applied external load, resulting in the total vertical displacement shifting downwards. Evolution of maximum horizontal displacement, governed by the thermal expansion of heated beams and shear resistance of columns, presents initially steep rise, with the rate of displacement reduced in time as the beam softens due to heat penetration (Figure 10).



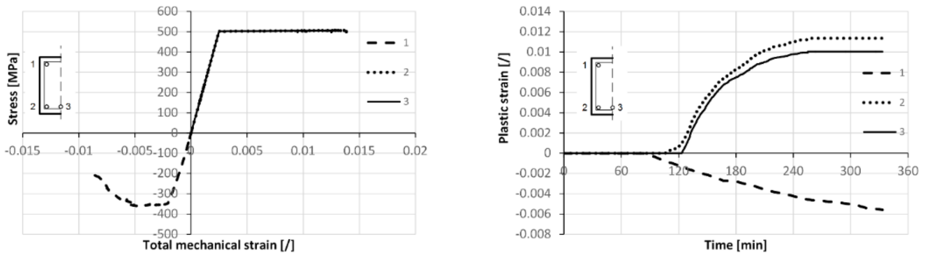
a) Vertical displacement of columns



b) Maximum horizontal displacement

Fig. 10. Vertical and horizontal displacement of exposed columns

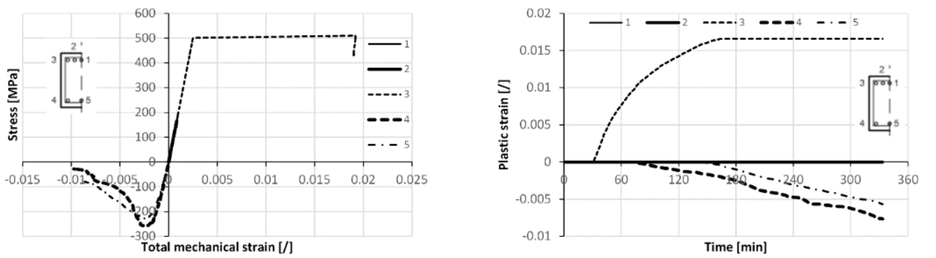
Stress-strain curves and plastic strain evolution in time of reinforcing bars in the section B1 (see Figure 7) are presented in Figure 11. Plastic strains start to develop after 80 minutes, first in the heated top corner bars, followed by yielding of bottom corner and middle bars at 111 and 125 minutes, respectively, whose temperatures remain slightly above ambient.



a) Stress-strain curves - Section B1 *b) Plastic strain - Section B1*

Fig. 11. Stress-strain and plastic strain development in reinforcement bars in section B1

Critical cross-section in the Beam 2 is located at a section where additional top reinforcement bars are no longer required. Suspension of additional reinforcement results in a redistribution of stresses to corner bars, eventually exceeding their load bearing capacity and causing them to yield (Figure 12).



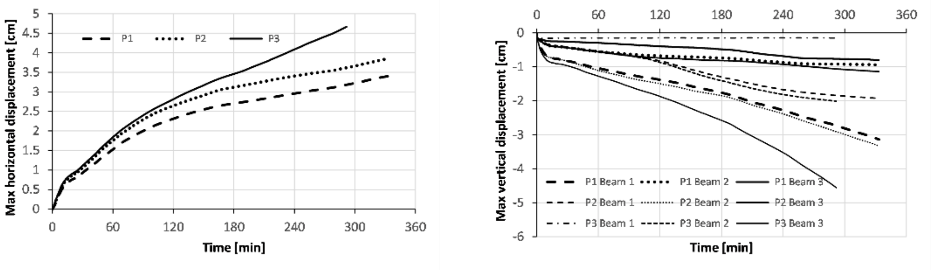
a) Stress-strain curves - Beam 2 critical section *b) Plastic strain - Beam 2 critical section*

Fig. 12. Stress-strain and plastic strain development in reinforcement bars in Beam 2 critical section

Although exposed columns are heated from all four sides, total mechanical strains in time are below the arbitrary 2%. Plastic strains start to develop in corner bars after 73 minutes, and in side bars after 202 and 268 minutes, but yielding of bars does not occur.

5. INFLUENCE OF FIRE SCENARIO ON STRUCTURAL RESPONSE

Analysis is further expanded to account for different fire scenarios, by assuming that fire could develop on each floor, representing separate fire compartments. Since thermal response is dependent on the type of fire, material and geometrical properties of exposed members, different fire scenarios result in the same temperature distribution translated to the elements exposed to fire. Maximum horizontal displacements occur when the fire is located on the top floor of the frame due to larger rotation of the beam-column connection. Vertical deformations of beams are larger in case of beams exposed to fire from bottom and side surfaces, enabling faster penetration of heat, compared to beams exposed only from the top (Figure 13).



a) Maximum horizontal displacements *b) Maximum relative vertical displacement*

Fig. 13. Horizontal and vertical deflections depending on the fire scenario

6. CONCLUSION

Structural fire behaviour can be analysed using numerical simulations that can adequately capture both thermal and structural response, taking into account changes in material properties at elevated temperatures. Proposed model can determine the evolution of deformations, stresses and strains at global and local level in RC frame structure, which can identify critical regions that need special consideration when structural integrity might be at risk in the events of fire.

In order to secure a favourable behaviour of RC structure during fire, integrity of concrete cover needs to be assured, to provide thermal protection of reinforcement and concrete core of the exposed member's cross-section. Concrete slab, as a boundary element of the fire compartment, with a thickness of 15 cm, can provide 4 hours of insulation criterion for standard fire exposure. When analysing the global effect of fire, it is important to consider possible fire scenarios. While fire scenario anticipating fire development on higher floors might be more favourable for building evacuation, it can cause more severe structural deformations and damages that could result in local failures, eventually leading to a progressive global collapse of the building. Although irreversible deformations develop in both concrete and reinforcement during fire, global structural integrity can be secured if proper design methods are applied. Analysed RC frame structure, designed for seismic action according to Eurocode standards, also possesses adequate fire resistance, mainly due to load bearing and deformation capacity reserves, demanded by the design procedures for seismically resistant structures.

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REFERENCES

[1] CEB-FIP: *Fire Design of Concrete Structures - Materials, Structures and Modelling* (T. Bulletin 38), International Federation for Structural Concrete (fib), **2007**
[2] ISO 834: *Fire Resistance Test - Elements of Building Construction.* International Standard 834, **1975**
[3] Franssen, J.-M.: SAFIR: A Thermal/Structural Program for Modeling Structures Under Fire. *Engineering Journal - AISC, 42*(3), **2005**, pp. 143-158
[4] Huang, Z.: The Behaviour of Reinforced Concrete Slabs in Fire. *Fire Safety Journal, 45*, **2010**, pp. 271-282
[5] Usmani, A., Zhang, J., Jiang, J., Jiang Y., Kotsovinos, P., Zhang, J., May, I. (2010). Using OpenSees for Structures in Fire. *Structures in Fire (Proceedings of the Sixth International Conference)*. Lancaster, Pennsylvania, **2010**, pp. 919-926
[6] Hawileh, R., Naser, M.: Thermal-Stress Analysis of RC Beams Reinforced with GFRP Bars. *Composites: Part B, 43*, **2012**, pp. 2135-2142
[7] Gao, W., Dai, J.-G., Teng, J., Chen, G.: Finite Element Modeling of Reinforced Concrete Beams Exposed to Fire. *Engineering Structures, 52*, **2013**, pp. 488-501
[8] EN 1992-1-2.: *Design of Concrete Structures, General Rules, Structural Fire Design*. European Committee for Standardization, **2014**
[9] EN 1991-1-2: *Actions on Structures, General Actions, Actions on Structures Exposed to Fire*, Brussels, European Committee for Standardization, **2002**
[10] Kodur, V.: Properties of Concrete at Elevated Temperatures. *ISRN Civil Engineering*, **2014**, pp. 1-15. doi:http://dx.doi.org/10.1155/2014/468510
[11] Bergheau, J.-M., Fortunier, R.: *Finite Element Simulation of Heat Transfer.* John Wiley & Sons, Inc., **2008**
[12] William, K., Warnke, E.: Constitutive model for the triaxial behaviour of concrete, *Concrete structures subjected to triaxial stresses*, **1974**, pp. 1-30
[13] ANSYS® Academic Teaching Mechanical: *ANSYS Help Documentation, Release 16.0.* Canonsburg: ANSYS, Inc., **2015**
[14] Pakala, P., Kodur, V.: Effect of concrete slab on the behavior of fire exposed subframe assemblies with bolted double angle connections, *Engineering Structures*, *107*, **2016**, pp. 101-115
[15] Klingsch, E.: *Explosive spalling of concrete in fire.* Zürich: Institut für Baustatik und Konstruktion, ETH Zürich, **2014**
[16] EN 1998-1-1: *Design of Structures for Earthquake Resistance: General Rules, Seismic Actions and Rules for Buildings,* Brussels, European Committee for Standardization, **2004**
[17] EN 1990: *Basis of Structural Design,* European Committee for Standardization, **2002**

НУМЕРИЧКА СИМУЛАЦИЈА ОДГОВОРА АРМИРАНОБЕТОНСКОГ ОКВИРА У ПОЖАРУ

Резиме: *Армиранобетонске (АБ) конструкције показују сложено понашање када су изложене пожару. Значајно термичко дејство изазива промене у микроструктури материјала и термичко-хидрално-механичким својствима, у зависности од брзине загревања, влажности, граничних услова, геометрије и димензија загреваног елемента, типа оптерећења, хемијско-физичких интеракција итд. Опсежна експериментална истраживања материјала омогућила су развој математичких модела и нумеричких процедура, који, у одређеној мери, могу адекватно да обухвате понашање конструкција у условима пожара. Смернице за напредно моделирање предложене су у стандардима, попут Еврокода. На основу ових претпоставки и претходних напора других истраживача, развијен је нумерички модел у програму на бази методе коначних елемената (МКЕ) ANSYS. Моделом се обухватају температурно зависна физичка, термичка и механичка својства материјала и спроводи се нелинеарни пренос топлоте и структурална анализа, чиме се симулира одговор АБ оквира изложеног стандардном пожарном дејству. Експлицитним моделирањем бетона и челичне арматуре омогућено је праћење развоја температуре посебно у бетонским и челичним елементима, затим деформација, пресечних сила и напона и дилатација у арматурним шипкама, пружајући опсежан увид у структурално понашање и на глобалном и на локалном нивоу. Посебан акценат је дат на утицај пожарног сценарија на понашање АБ оквира.*

Кључне речи: *пожарна отпорност, пренос топлоте, пожарна анализа конструкција, нумеричко моделирање*