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POST EARTHQUAKE FIRE RESISTANCE OF RC FRAME

M. Cvetkovska(1), R. Pejovic(2), A. Trombeva Gavriloska(3), M. Lazarevska(4), B. Stevanovic(5)

(1) Professor, University Ss. Cyril and Methodius in Skopje, Macedonia, Faculty of Civil Engineering, cvetkovska@gf.ukim.edu.mk
(2) Professor, University of Montenegro, Faculty of Civil Engineering, radenko@ac.me
(3) Assoc. Prof., University Ss. Cyril and Methodius in Skopje, Macedonia, Faculty of Architecture, agavriloska@arh.ukim.edu.mk
(4) Assist. Prof., University Ss. Cyril and Methodius in Skopje, Macedonia, Faculty of Civil Engineering, marijana@gf.ukim.edu.mk
(5) Professor, University of Belgrade, Serbia, Faculty of Civil Engineering, bole@imk.grf.bg.ac.rs

Abstract

Fire following an earthquake is an important factor causing damage to buildings and life-line structures. Therefore, besides satisfying structural design requirements for normal loads, such as dead and live loads, including the seismic hazard, buildings should also be designed to withstand the fire following earthquake for a certain minimum duration as required for a desired level of performance. Calculating structural response to fire after earthquake is a few step process: modeling the structure including nonlinear analysis options; choice for earthquake analysis scenario; seismic nonlinear analysis: pushover or dynamic time history; fire hazards analysis to identify all possible fire scenarios; thermal analysis to calculate temperature history in each member; structural analysis to determine forces, stresses and deformations to estimate whether local or global collapse would occur during any of the fire hazard scenarios.

To evaluate the seismic damage in a structure, first, the seismic hazard level is determined from the seismic hazard spectrum for the given site, followed by the selection of appropriate ground motion records and structural analysis. The seismic excitation induces damage and lateral deformation provoking additional stresses in the frame due to the moment caused by the P-D effect. Structural members and joints are also weakened by the cyclic inelastic deformation, causing stiffness and strength degradation. Once the earthquake-induced damage in the structure is determined, the damaged structure is subjected to a post earthquake fire (PEF) scenario, which involves fire hazard analysis to determine the time history of fire growth and spread and stress and collapse analysis of the structure but also to analyze no-collapse conditions and cooling after fire.

The behavior of a particular reinforced concrete structure that was fire exposed after seismic action is presented in this paper. The seismic response of the structure is evaluated using a pushover analysis, while the displacement demand under the corresponding seismic event is determined using the recommendations implemented in Eurocode 8. The earthquake-induced damage in the structure is determined and, as next step, the structure is exposed to Standard fire ISO 834. For that purpose the program FIRE, based on FEM, is used. The program FIRE carries out the nonlinear transient heat flow analysis and nonlinear stress-strain response associated with fire. The solution technique used in FIRE is a finite element method coupled with time step integration. The computer modulus FIRE-T solves the governing differential equation of heat transfer in conduction. The response of a reinforced concrete elements and plane frame structures exposed to fire is predicted by modulus FIRE-S. This modulus accounts for: dimensional changes caused by temperature differences, changes in mechanical properties of materials with changes in temperature, degradation of sections by cracking and/or crushing and acceleration of shrinkage and creep with an increase of temperature. Pushover analysis, representing dynamic effects of an earthquake via static nonlinear procedure, is incorporated in the program as an option that precedes the post-earthquake fire analysis.

Keywords: fire, earthquake, fire resistance, stress-strain analysis, pushover analysis

1. INTRODUCTION

Earthquakes represent an extreme event causing enormous damage to buildings, infrastructure, not to mention a loss of human lives. The occurrence of a fire following an earthquake can understandably produce a disastrous effect. Although ground shaking is the major concern in most earthquakes, subsequent fire could be even more dangerous to urban infrastructure. The post earthquake fire if ignited could grow, intensify, and spread out of control in the neighborhood. Therefore, besides satisfying structural design requirements for normal loads, such as dead and live loads including the seismic hazard, buildings should also be designed to withstand the fire following earthquakes for a certain minimum duration as required for a desired level of performance. Fire resistance requirements for specific building members and structures are provided in building codes. However, much of these criteria are developed for fire exposure under normal conditions without a cumulative damage from preceding earthquake [1,2].

The behavior of a particular reinforced concrete structure exposed to fire after surviving strong earthquake is presented. The seismic response of the structure was evaluated using a nonlinear static pushover analysis.

2. BASIC THEORY

A computational procedure for nonlinear analysis of reinforced concrete frame subjected to fire loading exposed to different fire models is analyzed. For that purpose the program FIRE [3] is used. A coupled thermal - structural analysis approach is implemented in the program. In each time step, the fire behavior of a structural member is estimated using a complex, coupled heat transfer - strain equilibrium analysis, based on theoretical heat transfer and structural mechanics principles. The analysis is performed in three sub steps within each time step: namely, calculation of fire temperatures to which the structural members are exposed, calculation of temperatures in the structural members, and calculation of resulting deflections and internal forces including an analysis of the stress and strain distribution. The program FIRE carries out the nonlinear transient heat flow analysis (modulus FIRE-T) and nonlinear stress-strain response associated with fire (modulus FIRE-S) [3].

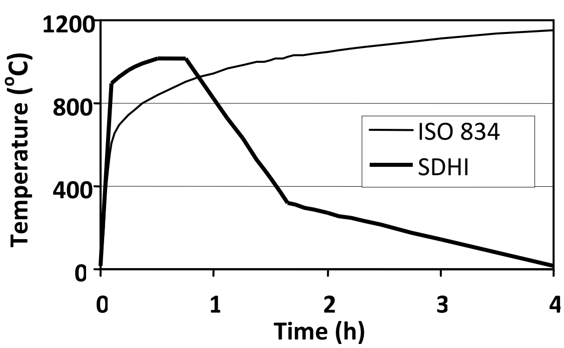

Fig. 1 - ISO 834 and SDHI fire models

The solution technique used in FIRE is a finite element method coupled with time step integration. The computer modulus FIRE-T solves the governing differential equation of heat transfer in conduction and in that purpose the following assumptions are made: a fire can be modeled by a single valued gas temperature history: ASTM E119, ISO 834 or SDHI (short duration, high intensity) fire model, Fig. 1; no contact resistance to heat transmission at the interface between the reinforcing steel and concrete occurs; the fire boundary conditions can be modeled in terms of both convective and radiating heat transfer mechanisms; the temperature dependant material properties are known (recommended in Eurocode 2, part 1.2) [4]; while cracks appear, or same parts of the element crush, the heat penetrates in the cross section easier, but in this study it is neglected. It has been assumed that the heat flow is separable from the structural analysis. The response of a reinforced concrete elements and plane frame structures exposed to fire is predicted by modulus FIRE-S. This modulus accounts for: dimensional changes caused by temperature differences, changes in mechanical properties of materials with changes in temperature, degradation of sections by cracking and/or crushing and acceleration of shrinkage and creep with an increase of temperature. To define the fire response of reinforced concrete structure is thus a complex nonlinear analysis problem in which the strength and stiffness of a structure as well as internal forces continually change due to restraints imposed by the structural system on free thermal expansion, shrinkage, or creep. Pushover analysis, representing dynamic effects of an earthquake via static nonlinear procedure, is incorporated in the program as an option that precedes the post earthquake fire analysis.

3. NUMERICAL EXAMPLE – CASE STUDY

3.1 Structural Geometry and Material Characteristics

The object of the numerical analysis is a two-story three bay planar reinforced concrete frame structure. Frame geometry, element cross-sections and reinforcement of all cross sections are schematically presented in Fig. 2. Concrete compressive strength is f=30MPa, reinforcement yield strength is f_y=400MPa. Structure self weight is included in the permanent and live loads, applied on beams as cumulate uniformly distributed, q=45kN/m'. Total weight of the structure is W=2x(45.0x15.0)=1350kN. The reinforcement of beam cross sections is taken in such a way that the stresses in steel bars due to nominal load q are approximately 60% of the yield strength. The percentage of column reinforcement is taken to be 1%.

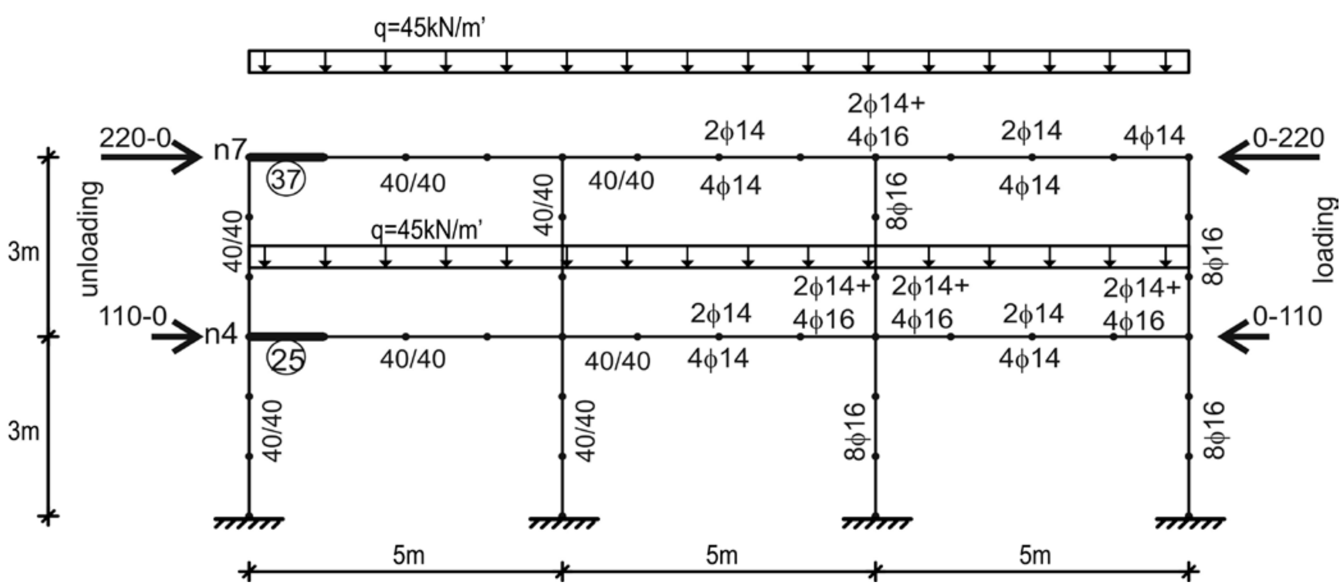

Fig. 2 - Geometry and loading

3.2 Description of Analysis Cases

Thirteen different loading cases have been analyzed:

1. Gravity load (q=45kN/m'), (abbreviated as "g" in figures and tables)
2. Gravity load + ISO fire scenario 1
3. Gravity load + SDHI fire scenario 1
4. Gravity load + ISO fire scenario 2
5. Gravity load + SDHI fire scenario 2
6. Gravity load + Pushover (loading + unloading) + ISO fire scenario 1
7. Gravity load + Pushover (loading + unloading) + SDHI fire scenario 1
8. Gravity load + Pushover (loading + unloading) + ISO fire scenario 2
9. Gravity load + Pushover (loading + unloading) + SDHI fire scenario 2

10. Gravity+Pushover (loading+ unloading +opposite loading+ unloading) + ISO fire scenario 1
11. Gravity+Pushover (loading+unloading+opposite loading+ unloading)+ SDHI fire scenario 1
12. Gravity+Pushover (loading+unloading +opposite loading+ unloading)+ ISO fire scenario 2
13. Gravity+Pushover (loading+unloading +opposite loading+ unloading)+SDHI fire scenario 2

Fire scenario 1 assumes fire in the first story left compartment and fire scenario 2 assumes fire in the second story left compartment of the frame. In the pushover analysis a triangular load distribution in horizontal direction is applied, Fig. 2. In loading case 6 up to loading case 9, the horizontal forces are incrementally increased from 0 to 220kN on the second floor and from 0 to 110kN on the first floor pushing the rightmost floor nodes of the structure (applied in direction from right to left).

3.3 Results of Analysis Cases

The value for the base shear of 330kN (220kN + 110kN) corresponds to 0.244W and is chosen in such a way that the structure is pushed into nonlinear range and rather large number of plastic hinges are formed. Total base shear has been reached corresponding to top story horizontal displacement of 4.23cm, Fig. 3a. Once the loading phase up to corresponding base shear of 330kN is completed, unloading phase takes place. After unloading the corresponding residual plastic disp9lacement at node 7 was 1.09cm, Fig. 3b.

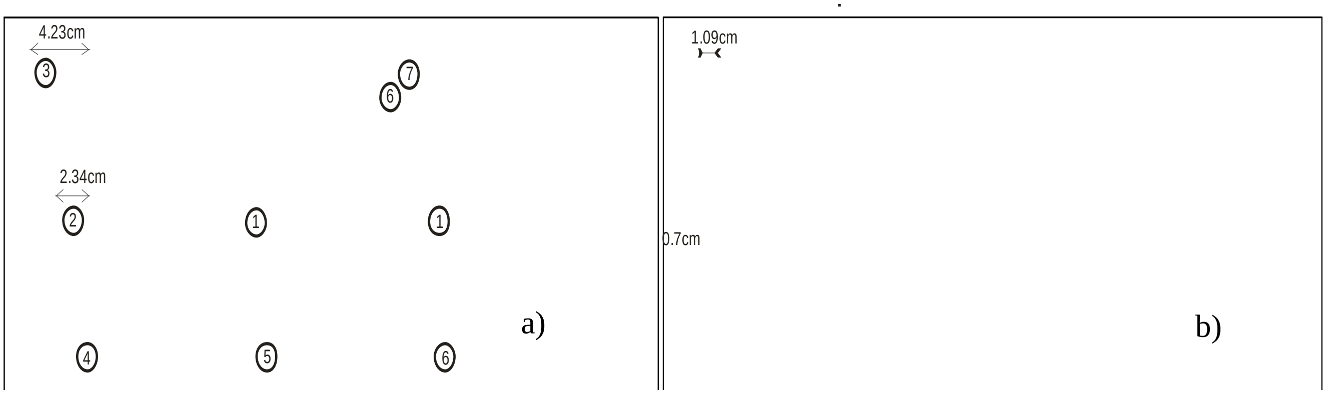

Fig. 3 – a) Sequence of plastic hinges formation and horizontal displacements due to pushover, b) Residual displacements after pushover

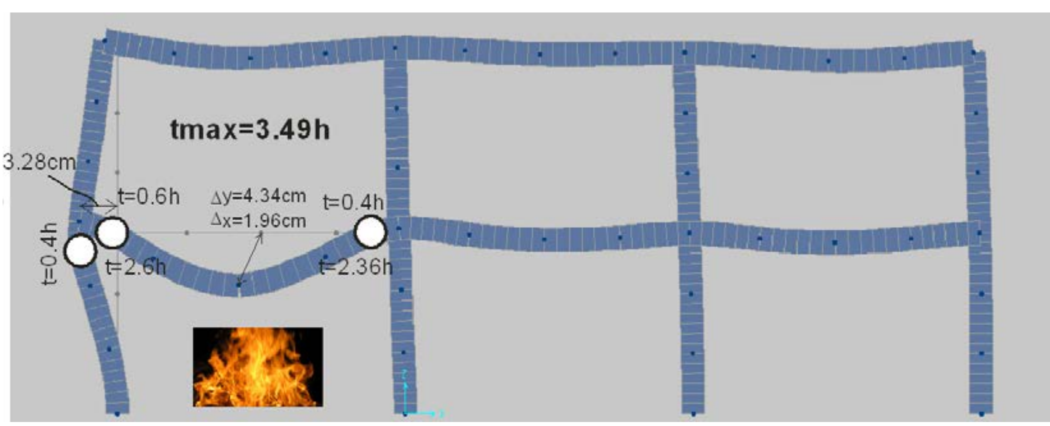

Fig. 4 - Load case 6, sequence of hinges due to fire

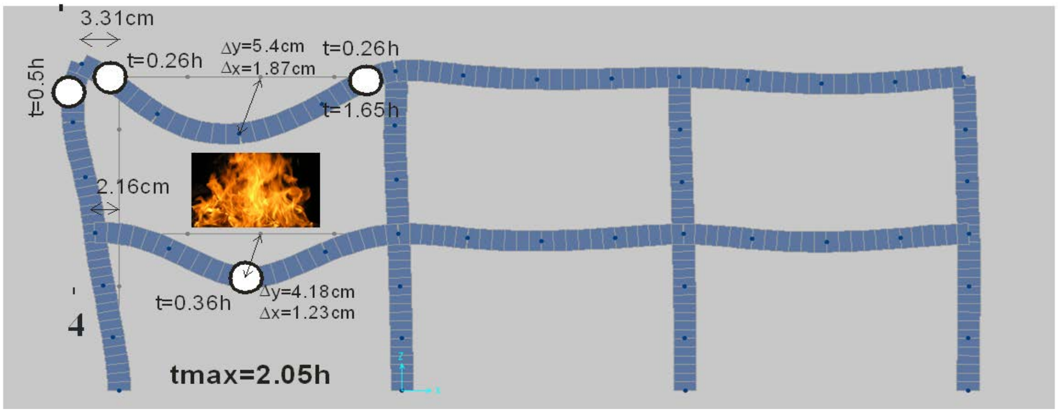

Fig. 5 - Load case 8, sequence of hinges due to fire

To accomplish loading cases 6 to 9, after unloading the structure is exposed to two different fire scenarios for two different fire load models (Fig.4 and Fig.5). In loading case 10 up to loading case 13, the horizontal forces are applied to the structure that is already plastically deformed in the previous cycle of pushover analysis and are incrementally increased from 0 to 220kN on the second floor and from 0 to 110kN on the first floor, pushing the leftmost floor nodes (node 7 and node 4) of the structure (applied in direction from left to right). Again, to accomplish loading cases 10 to 13, after unloading the structure from total base shear of 330kN in the opposite direction, the frame is exposed to two different fire scenarios for two different fire load models. Some of the obtained results are graphically presented. Base shear-displacement relations from loading case 10 for nodes 7 and 4, due to ISO fire scenario 1, are presented in Fig. 6. Loading, unloading, loading in opposite direction and unloading from opposite direction are assigned as: L1PO, UL1PO, L2PO and UL2PO consequently. The sequence of formation of plastic hinges on Fig. 3 are shown on the L1PO branches of Fig. 6. Additional results for displacements of nodes: 4, 7, 9 and 12 for characteristic moments of the analysis in fire scenario 1 or 2 and for fire models ISO and SDHI, are presented in Table 1, Table 2 and Table 3.

As expected, residual horizontal displacements of nodes 4 and 7 as well as the residual vertical displacements of nodes 9 and 12, are slightly higher when fire action is applied after pushover, then in the case when no seismic action was applied. This trend is observed in both fire scenarios and both fire models. Interesting results are obtained for the capacity of this reinforced concrete structure to sustain fire load. Namely, the duration of time that the structure survived the ISO fire after a pushover episode was higher then the duration of time that the structure survived the ISO fire without a seismic action in fire scenario 1, 3.49 hours against 3.3 hours (loading case 6). In fire scenario 2 the duration of time, for both cases, was almost equal, t=2.0 hours (loading case 8). That was even more emphasized in loading cases 10 and 12, when a full cycle (loading, unloading, loading in opposite direction and unloading from opposite direction) was completed, Table 3.

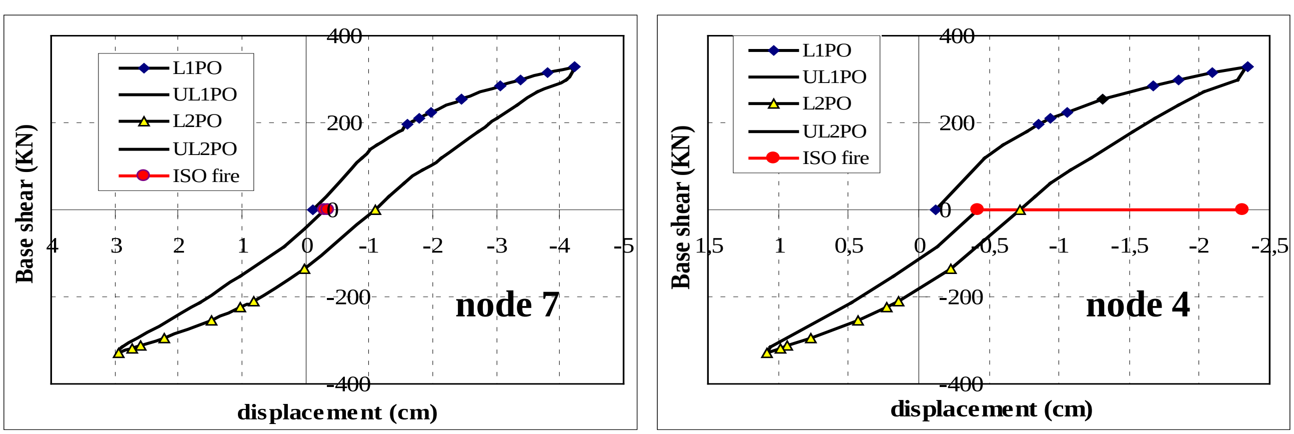

Fig. 6 - Base shear-displacement relations for nodes 7 and 4, due to ISO fire scenario 1

Table 1 - Fire scenario 1

Node	Displac. (cm)	only "g" and fire action				"g"+ seismic action + fire action					
		g	g+ISO t=3.3h	g+SDHI t=1.1h	g+SDHI t=5h	g	g+LPO	residual displac.	residual + ISO t=3.49h	resid.+SDHI t=1.1h	resid.+SDHI t=5h
4	Δx	-0.12	-2.85	-1.83	-1.08	-0.12	-2.34	-0.72	-3.28	-2.23	-1.49
7	Δx	-0.13	-0.27	-0.20	-0.19	-0.13	-4.23	-1.09	-1.12	-1.03	-1.01
9	Δy	-0.43	-4.14	-1.27	-1.61	-0.43	-0.64	-0.78	-4.34	-1.45	-1.78

Table 2 - Fire scenario 2

Node	Displac. (cm)	only "g" and fire action				"g"+ seismic action + fire action					
		g	g+ISO t=2.0h	g+SDHI t=1.1h	g+SDHI t=5h	g	g+LPO	residual displac.	residual + ISO t=2.0h	resid.+SDHI t=1.1h	resid.+SDHI t=5h
4	Δx	-0.12	-1.62	-1.22	-0.70	-0.12	-2.34	-0.72	-2.16	-1.79	-1.27
7	Δx	-0.13	-2.62	-1.98	-1.89	-0.13	-4.23	-1.09	-3.31	-2.67	-2.25
9	Δy	-0.43	-3.67	-2.68	-1.53	-0.43	-0.64	-0.78	-4.18	-3.23	-2.46
12	Δy	-0.56	-5.34	-3.29	-4.26	-0.56	-0.59	-0.73	-5.41	-3.27	-4.29

Table 3- Fire scenario 1 and 2

node	Displac. (cm)	"g"+ seismic action in two directions+ fire action						
		g	g+L1PO	residual displac. 1	g+L2PO	residual displac.2	residual + **fire 1** t=3.54h	residual + **fire 2** t=2.43h
4	Δx	-0.12	-2.34	-0.72	+1.12	-0.42	-2.31	-1.60
7	Δx	-0.13	-4.23	-1.09	+3.05	-0.30	-0.36	-1.55
9	Δy	-0.43	-0.64	-0.78	-0.87	-0.96	-3.70	-4.20

The reinforcement of the RC frame cross sections is defined from criteria that do not take into account seismic provisions. In order to see what is the seismic capacity of the structure and to what level of seismic demand corresponds the assumed base shear of 330kN, the N2 method was implemented [5]. Final results of capacity-demand calculations are graphicaly presented in Fig. 7. Base shear of 330kN and the obtained displacement of 4.23cm at node 7 corresponds to elastic demand spectrum for PGA=0.129g. By inverse procedure, it was found that this RC frame has capacity (base shear of 420kN and target displacement of 9.88cm) to sustain elastic demand spectrum for PGA=0.3g. All loading cases were reapplied such that the horizontal forces were increased up to a base shear of 393kN (corresponding approximatelly to 94% of frame's capacity). Due to limited space only few results for displacements for nodes 4 and 7 are listed. For base shear=393kN, Δ_{x4}=4.19cm, Δ_{x7}=7.31cm. For base shear=0kN (unloading), Δ_{x4}=1.98cm, Δ_{x7}=3.04cm (residual displacements). It is worth mentioning also, that increased resistance in case of fire scenario 1 was observed. The structure has sustained fire load after earthquake in duration t=3.76 hours.

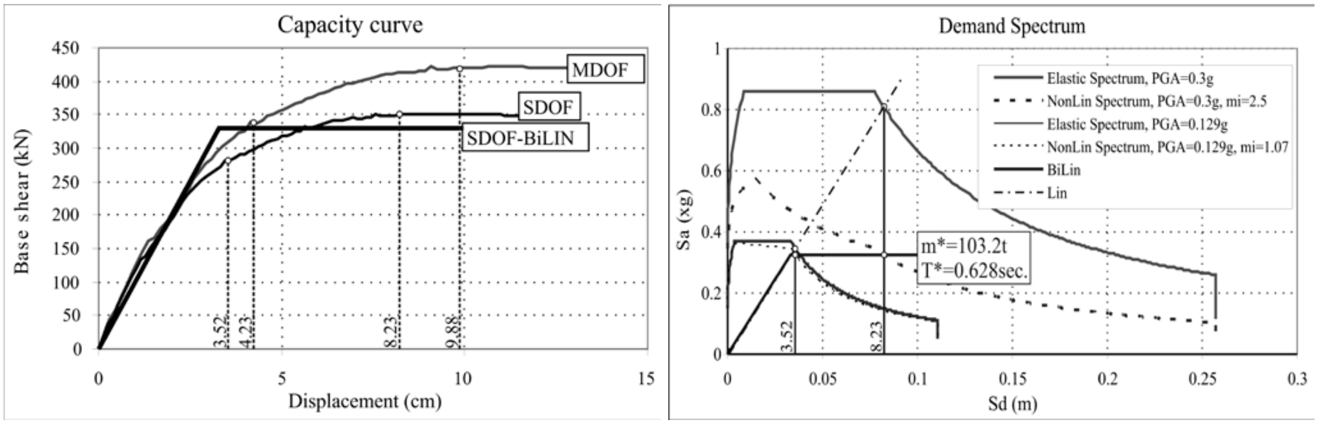

Fig. 7 - Pushover curve – capacity curve;   Elastic and demand spectra

Stress-strain hystories for reinforsing bars in beam cross sections of nodes 4 and 7 for applied loading cases are presented in Fig. 8 thru Fig. 11. The interpretation of the numbers next to particular stress-strain curve is such that, for example on Fig. 8 the stress-strain hystory for ISO fire-3 starts at point 3, continues thru point 3' and finishes at point3''. Also, the stress-strain relation for some reinforcement bars is doubly presented, as for example bottom bar, Node 4, fire scenario 1 on Fig. 9. The left graph presents real stress-strain relations during time history of loading and on the right graph stresses are normalized as percentage values of the reduced yield stress due to elevated temperature, $f_y(T)$.

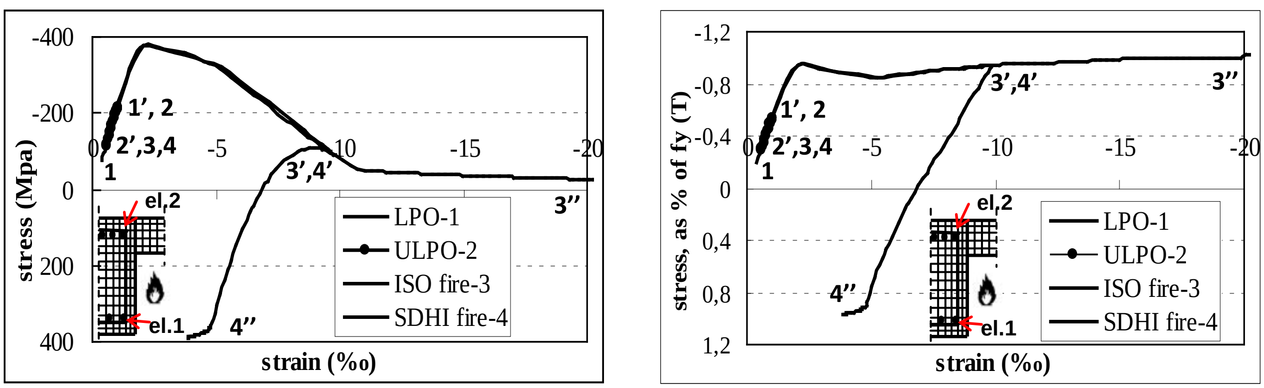

Fig. 8 - Bottom bar, Node 4, fire scenario 1

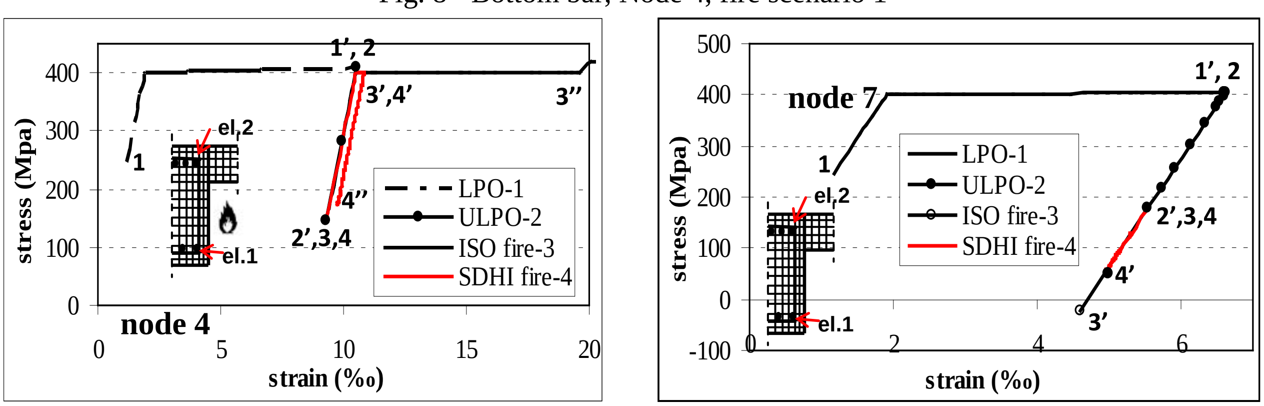

Fig. 9 - Top bar of node 4 and top bar of node 7, fire scenario 1

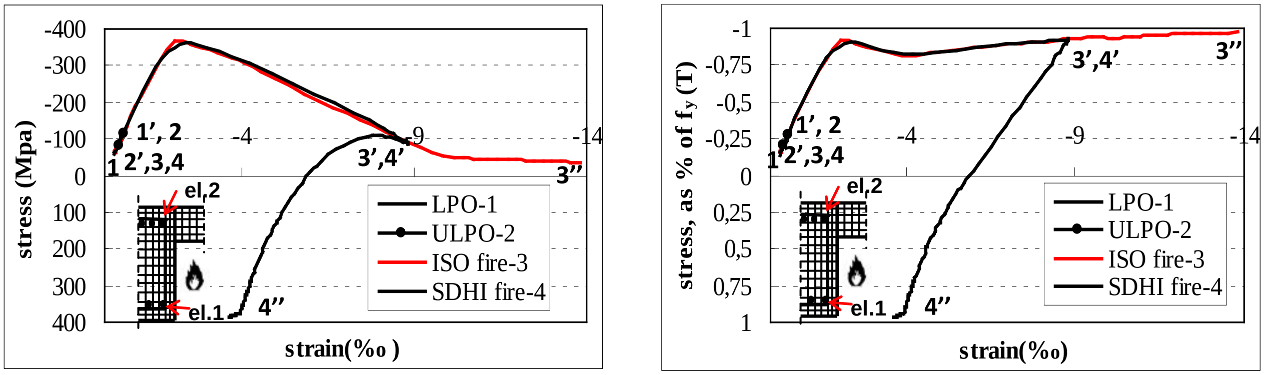

Fig. 10 - Bottom bar, Node 7, fire scenario 2

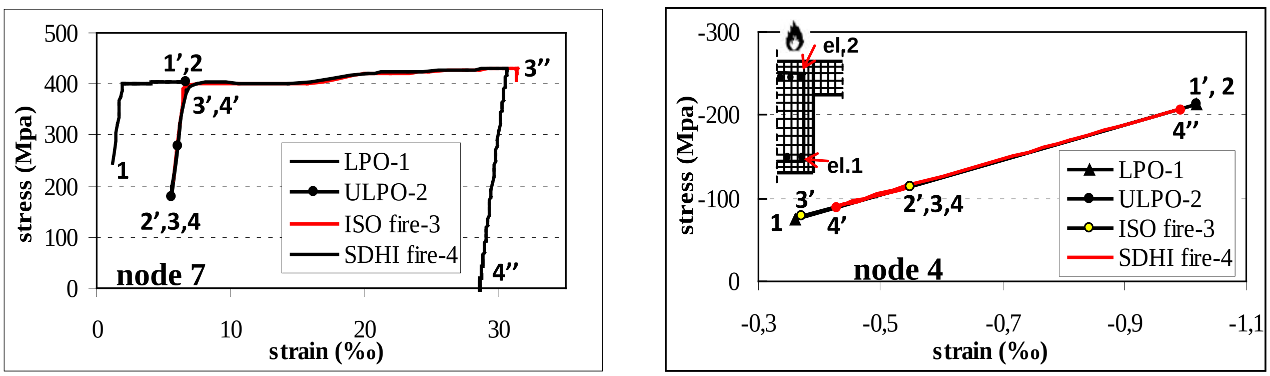

Fig. 11 - Top bar of node 7 and bottom bar of node 4, fire scenario 2

4. CONCLUSIONS

The numerical analyzes on the fire resistance of a reinforced concrete frame, designed according to the EUROCODE 2 and EUROCODE 8 are presented in this paper. Based on the analysis results it is found out that the safety factors for materials and loads, as well as special seismic design requirements for structural elements directly affect the capacity of reinforced concrete structures under fire conditions. Structures designed to satisfy only the strong seismic requirements (fire safety design is not considered) show higher fire resistance in comparison with structures designed without seismic and fire safety criteria. The reason for that is the fact that in fire conditions the strains in the constitutive materials (steel and concrete) could reach much higher values than in case of room temperatures. This fact leads to a conclusion that further investigations have to be conducted for the opposite case, when fired structures are exposed to seismic action without previous repair.

5. References

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