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ROBUSTNESS CAPACITY OF MULTISTOREY STEEL STRUCTURES IN CASE OF FIRE AFTER EARTHQUAKE EVENTS

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Abstract. The study summarised in the present paper has investigated the post-earthquake robustness of multistorey steel framed structure prone to fire action. Building Frames, i.e., homogeneous system with moment resisting frames (MRFs) on one direction and centrically braced frames (CBFs) with inverted V braces on the other direction, of 4 and 8 stories, as case study structures, are numerically analysed. The structures were designed according to the relevant codes for persistent and seismic design situations in 2 locations with different seismicity. The structures were assessed and optimized for seismic loading using a push-over analysis. Afterwards, the robustness capacity was checked against thermal action, through a nonlinear dynamic analysis, using Extreme Loading for Structures (ELS) software [8].

Key words: robustness, earthquakes, fire scenarios, multi-storey steel frames, nonlinear analyses.

1. INTRODUCTION

The *robustness* capacity of a structural system may be defined as the ability to survive (intact or with an acceptable level of damage) to a given set of threats, and generally characterizes the entire system rather than its individual components, according to Eurocode 1 [1]. A target-oriented definition, more complex, has been recently suggested by the Working Group WG6, "Robustness", of CEN/TC250 [2], and refers to the capability of the system to avoid disproportionate collapse: "*Structural robustness is an attribute of a structural concept, which characterizes its ability to limit the follow-up indirect consequences caused by the direct damages (component damages and failures) associated with identifiable or unspecified hazard events (which include deviations from original design assumptions and human errors), to a level that is not disproportionate when compared to the direct consequences these events cause in isolation*". Accordingly, robustness may be regarded as an indicator of the ratio between direct and indirect consequences due to certain hazards [3].

A robust structure is generally linked to a high degree of redundancy in terms of seismic risk of built environment. Consequently, the design should lead to a good balance between stiffness, strength, and plastic deformation capacity of members and connections. As a result, when some members are loaded excessively and damaged, it is expected that effective alternative routes for redistributing the loads and prevent the propagation of damage and eventually the collapse to be available. Current seismic design codes aim to fulfill these objectives by applying the capacity design method and relevant admissibility criteria. However, in case of cascading events like fire or explosion after earthquake, which are not covered in the current design codes, the situation may prove to be critical for the structure [4].

Steel structures usually possess good seismic performance and are preferred since a substantial reduction in construction time may be obtained and they are more sustainable. However, steel is vulnerable to fire action since its mechanical properties are drastically reduced at high temperatures. Even though the elements may be protected, ensuring appropriate period of protection, the second order effects that appear due to residual displacements post-earthquake can induce progressive collapse Consequently, adequate concern should be raised on post-earthquake behaviour of multistorey building steel frames, already damaged, and with residual lateral drifts under potential fire scenarios [5–7].

The current paper investigates the situations in which a seismic resistant structure, after an earthquake, is prone to subsequent fire. Building Frames, i.e., homogeneous system with moment resisting frames (MRFs) on one direction and centrically braced frames (CBFs) with inverted V braces on the other direction, of 4 and 8 stories are numerically analysed. The structures have been Code based designed for persistent and seismic design situations in 2 locations with different seismicity. The structures were optimized for seismic loading via a push-over analysis. Afterwards, the robustness capacity was checked against progressive thermal action due to fire, through a nonlinear dynamic analysis, using Extreme Loading for Structures (ELS) software [8].

2. DESIGN OF REFERENCE STRUCTURES

The structures selected for the study have 4 and 8 stories of 4.0 m, 3 bays of 6.0 m on transversal direction (*X* direction) and 3 spans of 6.0 m on longitudinal direction (*Y* direction) – see Fig. 1 for the structural configuration of the 4 stories DBF.

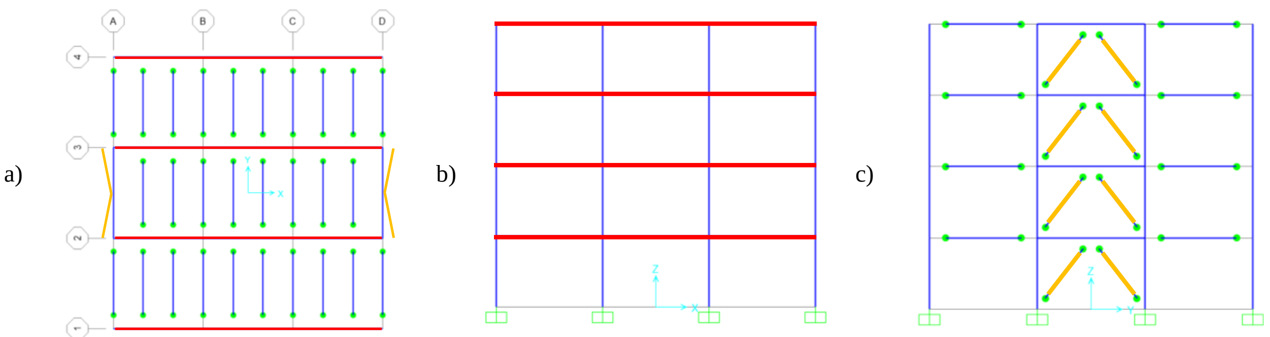


Fig. 1 – a) Plan view; b) transversal MRF*; c) longitudinal CBF** of the case study structure.

Legend: *Beams in MRF are red; **Braces in CBF are orange; All joints with green are considered pinned.

Table 1

Cross sections for the members

Location of the structure	Story	GLRS*	LLRS**			
		Secondary beams	MRF beams	Braces	Non-dissipative beams	Columns
Bucharest	4	IPE300	IPE360	152.4×7.1	HEA360	HEB400
	3			177.8×8.0	HEA360	
	1-2			219.1×8.0	HEB400	
	8	IPE300	IPE500	152.4×7.1	HEA360	HEM360
	7			177.8×8.0	HEA360	
	6			219.1×8.0	HEA400	
	5			244.5×8.0	HEA400	
	4			244.5×8.0	HEA400	
	1-3			273×8.0	HEB400	
Cluj	4	IPE300	IPE300	108.0×5.6	HEA260	HEB300
	3			127.0×5.6	HEA280	
	1-2			139.7×5.6	HEA280	
	8	IPE300	IPE400	101.6×5.6	HEA280	HEB400
	7			108.0×5.6	HEA280	
	6			114.3×6.0	HEA280	
	5			127.0×5.6	HEA280	
	4			139.7×5.6	HEA300	
	1-3			141.3×6.3	HEA300	

Legend:
*Gravity load resisting system (GLRS); **Lateral load resisting system (LLRS)

Three steel grades were used for the elements, i.e., S275 for the dissipative braces and S355 for the rest of the elements, including the beams and the columns. The last steel grade, S460 was used just for the columns in case of the 8 levels structure located in the area with higher seismicity. The structures were designed for the persistent and seismic design situation, with a dead load was 4.3 kN/m^2 on all levels and additional 0.5 kN/m^2 load on the façade walls and the live load of 3.0 kN/m^2. The climatic loads, i.e., wind load and snow load, were considered independently according to the location for each structure.

The structure was designed in two separate locations in Romania, with high and low seismicity to study the influence of the seismic design on the robustness capacity, i.e., Bucharest and Cluj, with the peak ground acceleration of 0.30 g and 0.10 g. The seismic design was performed according to the Romanian seismic design norm [9], and ductility factors q = 6.5 (for MRFs) and q = 2.5 (for CBFs) were considered for ductility class high DCH. The output of the design for the case study structures in both locations, detailed in Table 1 and Table 2, presents results of the modal analysis with the first two modes in all cases as translations, first on X direction, second on Y direction, respectively, while the third mode is torsional for all structures.

Table 2

Modal properties for the case study structures

Location of the structure	Stories	Mode	Period [s]	Location of the structure	Stories	Mode	Period [s]
Cluj	4	1	1.45	Bucharest	4	1	0.99
		2	0.74			2	0.54
		3	0.62			3	0.44
	8	1	1.93		8	1	1.38
		2	1.61			2	1.14
		3	1.2			3	0.85

2.1. Seismic Performance Based Evaluation (SPBE)

The SPBE of the structures was done via a push-over analysis performed in SAP2000 software [10], and the degradation state according to N2 method [11]. Figure 2 shows the capacity curves for the four structures, as well as the target points for Damage limitation state (DL), Significant Damage state (SD) and Near Collapse state (NC). For this study, only the capacity of the MRFs was studied, while the subsequent thermal loading will be assigned on the elements relevant for this structural system. From the results presented in Fig. 3 and Fig. 4 (deformation state shown only for SD and NC since for DL no plastic hinges occurred) it may be inferred that the structures have proper plastic mechanism and the rotations in the beams are below the limits.

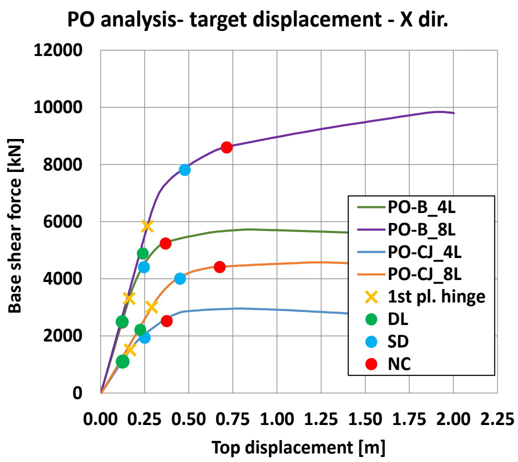


Fig. 2 – Force-displacement capacity curves and target points.

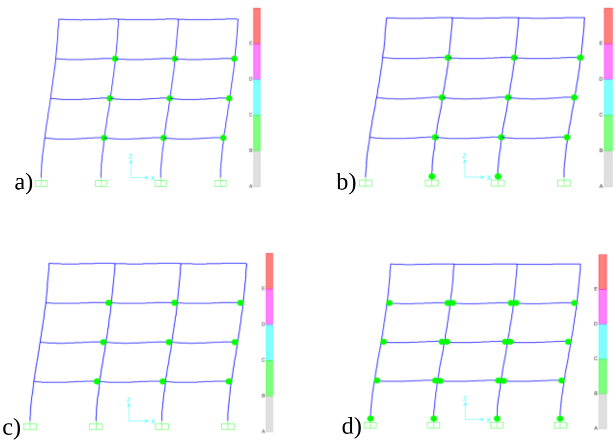


Fig. 3 – Plastic mechanism for the 4 levels structures: a) at SD and b) at NC in Cluj; c) at SD and d) at NC in Bucharest.

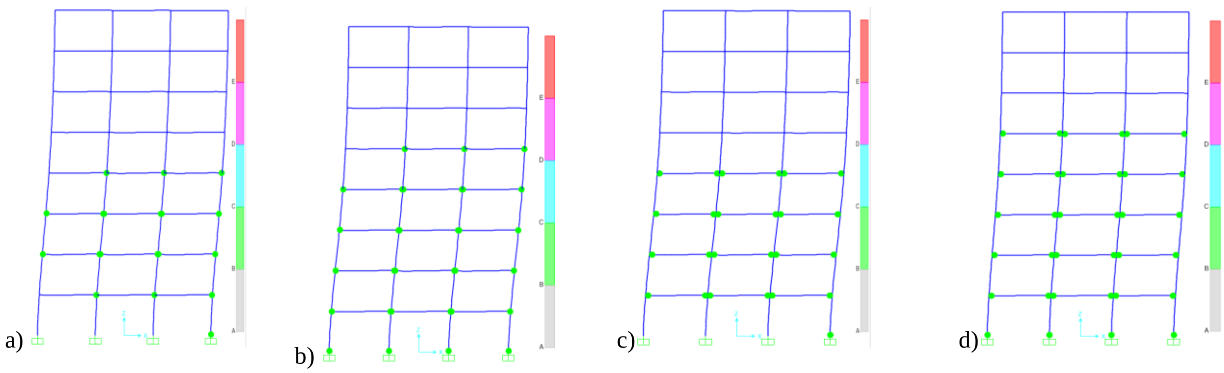


Fig. 4 – Plastic mechanism for the 8 levels structure: a) and b) in Cluj, at SD and NC, respectively; c) and d) in Bucharest at SD and NC, respectively.

3. ROBUSTNESS ASSESSMENT FOR POST-EARTHQUAKE FIRE SCENARIOS

3.1. Fire scenarios

There are several alternatives to study the influence of fire action after an earthquake. In [12] is presented the approach in which the fire acts on one element. Consequently, a thermal analysis may be performed with localised fire models, or a notional column removal may be used in ALPM method [12]. Alternatively, in [13] the thermal action was applied on both columns and beams, but the analysis was performed on plane frames. However, in these cases, the effect of the temperature developing in a compartment and affecting other elements is neglected. In [14] a method in which a compartment is considered to be influenced by the temperature effect, is presented. Consequently, for the performed parametric study, two compartments at the ground floor were considered (marginal and central with black lines as seen Fig. 5).

The thermal action for the beams and columns was considered independently. Table 3 shows the peak values of the temperature reached by the members, while in Fig. 6 the elements on which uniformly thermal action is applied are highlighted with orange colour.

Table 3

Temperatures assumed for thermal analysis

Element type	Peak value of the temperature [°C]
Column	800
Main beam	700
Secondary beam	500

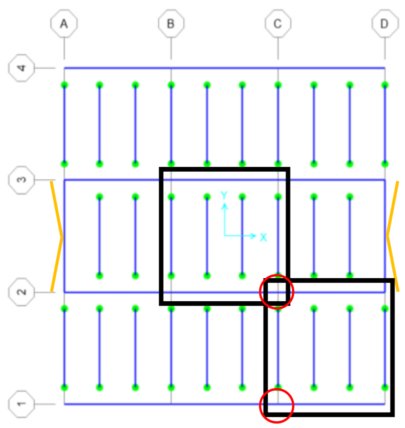


Fig. 5 – Scenarios considered to be affected by the temperature effect.

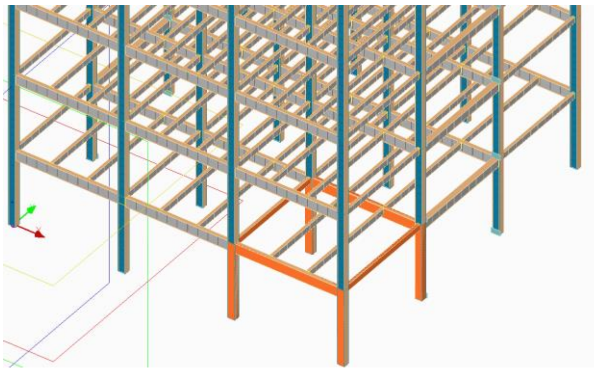


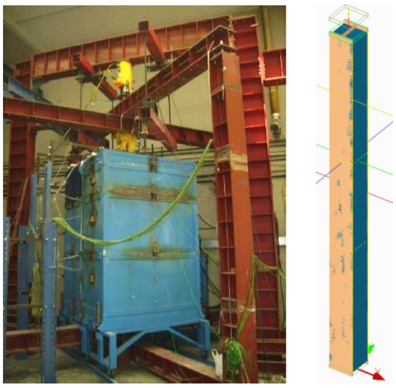
Fig. 6 – 3D from a numerical model.

3.2. Developing a simplified numerical model

The thermal analyses were performed in Extreme Loading for Structure (ELS) software that uses Applied Element Method (AEM) [8]. A numerical model on a single column was developed and calibrated against relevant experimental results before creating a 3D model of the structure. Thus, from [15] a single steel column was considered from the extensive experimental campaign conducted. Figure 7a presents the experimental setup used for testing. Different steel sections (HEA160 and HEA200) were tested at the thermal action, with different loading and stiffness conditions. For the numerical study presented in this paper, an HEA200 steel section was chosen, which was loaded axially with 1000 kN. In Fig. 7b is depicted the numerical model, for which solid elements were preferred. The column has a length of 3m, and the relevant material properties were assigned. In terms of discretization, 5 elements were used for the flange and web and 50 elements on the length of the column, respectively.

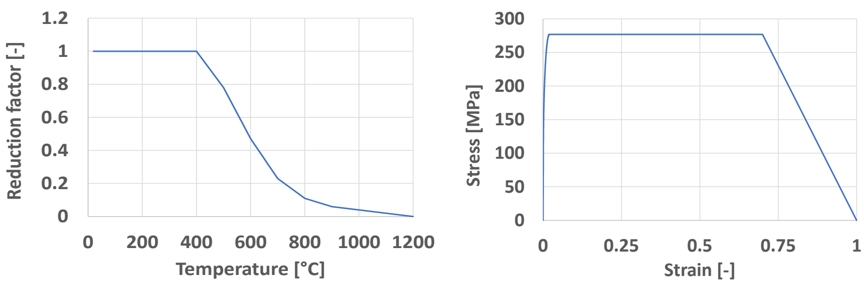
The stiffness of the surrounding structure (for the current model was 13 kN/mm) was considered using an equivalent spring placed at the top part of the column. To avoid numerical instabilities, the thermal action was considered the same on the entire length of the column, even though in the experimental setup the furnace heated the column on a length of 1m and it was placed at the mid-height of the column.

According to [16] the material properties are influenced by the temperature, resulting in a gradual decrease. In the numerical model, the material degradation was considered in a simplified way, using a reduction factor for the yield strength as presented in Fig. 8a and the stress-strain curve will be modified as shown in Fig. 8b.



a) b)

Fig. 7 – a) Experimental testing set-up for the columns [15]; b) numerical model of the column.



a) b)

Fig. 8 – a) Reduction factor used for material degradation; b) stress-strain curve for S355 computed for 500°C used in the ELS software.

The analysis was performed by applying first the vertical load in a nonlinear static analysis. Afterwards, the thermal action was applied on the column in a nonlinear dynamic analysis. Fig. 9 shows the comparative results in terms of normalized force (with respect to the force applied), and Fig. 10, in terms of vertical displacement, respectively. The analysis was considered in the strength domain, so the time may not be the most representative variable to be used to plot the results. However, both peak values are very close compared to the results registered from the experimental test.

a) b)

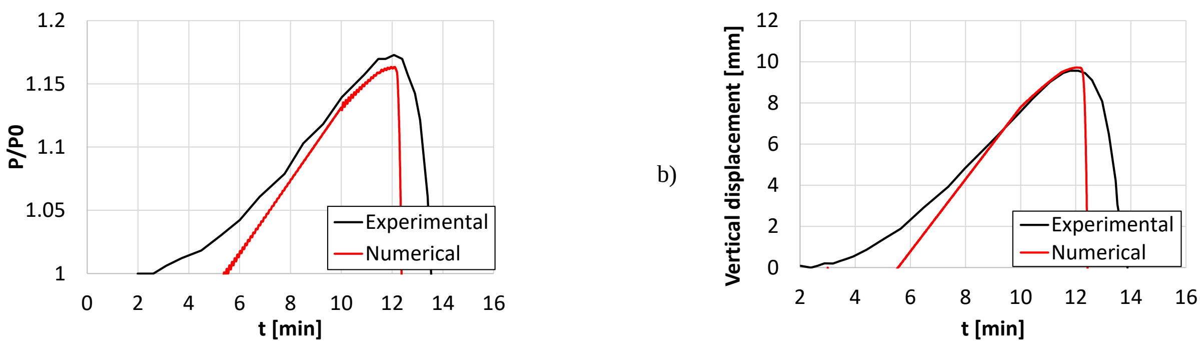


Fig. 9 – Experimental *vs*. numerical results: a) normalized force *vs*. time; b) vertical displacement *vs*. time.

Moreover, Fig. 10 shows the failure mechanism for the experimental test and the numerical model. Even though the magnitude of the out-of-plane deformation may be higher, the mechanism is the same resulting in a good overall agreement in terms of results.

a) b)

Fig. 10 – Experimental *vs*. numerical failure mode: a) experimental; b) numerical, showing also out-of-plane deformation.

3.3. Numerical model

The 3D numerical models for each case study structure were developed in ELS taking into account the assumptions and the results obtained in the previous chapters. As mentioned in the introductory part, the thermal action will act after the earthquake action ended. Consequently, after applying the gravitational loading on all levels, the structure is pushed up to the target displacement for SD limit state in a nonlinear static analysis conducted in force control. Then, the structure is unloaded using the same type of analysis and the post-earthquake state is obtained. Table 4 presents the top residual displacement for each case study structure.

Table 4

Residual displacements for the structures at SD limit state

Location	Levels [–]	Residual displacement in *X* dir. [m]
Cluj	4	0.021
	8	0.059
Bucharest	4	0.023
	8	0.119

Afterwards, a second nonlinear dynamic analysis is performed in which the thermal action is applied on the elements from compartment function of the considered scenario. Apart from the 8 cases for which the fire action is considered on the compartment, another 8 cases were considered, in which the columns surrounded by red circles in Fig. 5 are removed in case of column loss scenarios. The process is considered dynamic, hence a very short removal time, i.e., 0.001s is used in a nonlinear dynamic analysis. Thereby, a comparison between the thermal action developing in a compartment versus on a single element can be studied.

3.4. Results and comments

Table 5 presents the results of the parametric study in terms of the outcome of the analysis, whether it is progressive collapse, significant damage or insignificant damage. Also, Fig. 11 and Fig. 12 present the history of lateral deformation for the relevant columns in case of thermal analysis and history of vertical displacement in case of column removal scenarios, respectively.

It may be inferred that progressive collapse occurs only in case of thermal action for the structure located in Cluj, area with low seismic demand, for both structures with 4 and 8 levels. Fig. 13 presents the case of progressive collapse for 4 levels structure in case of central compartment subjected to thermal action. In case of the structure with 8 levels, severe out-of-plane deflection of the columns occurs, even if it does not lead to progressive collapse immediately. In case of the structure with 4 levels, plastic hinges occur in the beams in case of a single column being removed using ALPM.

Table 5

Overview of the results obtained from the parametric study

Nb.	Location of the structure	Levels	Compartment [thermal action]	Column loss scenario [ALPM]	Outcome
1	Cluj	4	Corner	–	Progressive collapse
2	Cluj	4	Central	–	Progressive collapse
3	Cluj	4	–	Marginal	Significant damage (beams)
4	Cluj	4	–	Central	Significant damage (beams)
5	Cluj	8	Corner	–	Progressive collapse
6	Cluj	8	Central	–	Significant damage (columns)
7	Cluj	8	–	Marginal	No damage
8	Cluj	8	–	Central	No damage
9	Bucharest	4	Corner	–	Significant damage (columns)
10	Bucharest	4	Central	–	Significant damage (columns)
11	Bucharest	4	–	Marginal	No damage
12	Bucharest	4	–	Central	No damage
13	Bucharest	8	Corner	–	Significant damage (columns)
14	Bucharest	8	Central	–	Significant damage (columns)
15	Bucharest	8	–	Marginal	No damage
16	Bucharest	8	–	Central	No damage

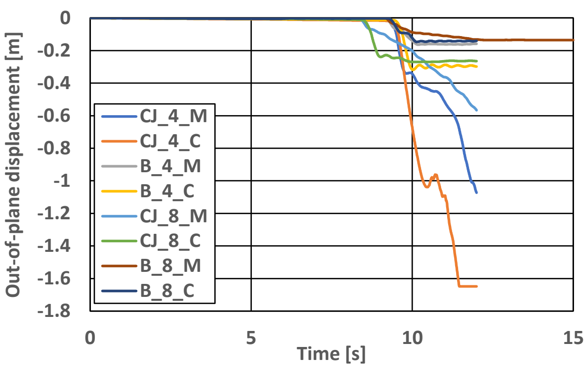


Fig. 11 – History of lateral displacement for the relevant columns in case of thermal action.

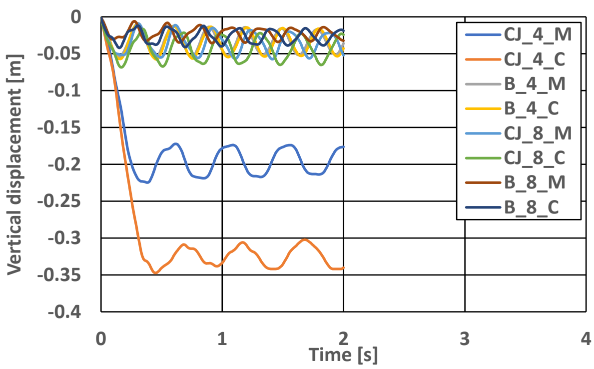


Fig. 12 – History of vertical displacement in case of column loss scenarios.

In case of the second location, Bucharest with high seismicity, the structures exhibit a better overall behaviour. The thermal action in both 4 and 8 levels structures do not lead to progressive collapse, only to significant out-of-plane deformation, as depicted in Fig. 14. Moreover, no damage for the beams was recorded in case of column removal scenarios.

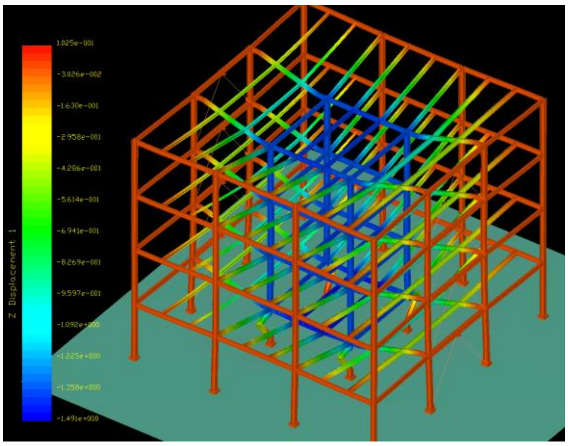


Fig. 13 – Vertical displacement in case of progressive collapse for the 4 levels structure located in Cluj, with central compartment subjected to thermal action

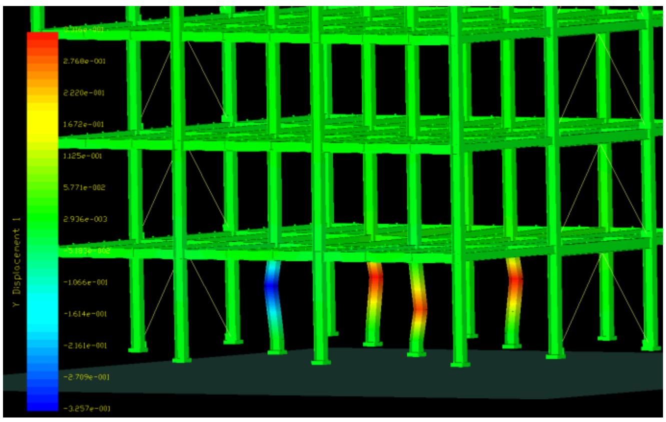


Fig. 14 – Out-of-plane displacement in case of severe damage for the columns for the 4 levels structure located in Bucharest, with central compartment subjected to thermal action.

4. CONCLUDING REMARKS

The results obtained through the case studies, convincingly show that the seismic intensity dictates a more robust and safer design, as no structure located in Bucharest developed progressive collapse.

The structural design considering the common particularities like capacity design, overstrength, ductility and redundancy provided an appropriate design output, as provided by the performance based seismic assessment. However, in case of the subsequent fire action on the compartments, these design considerations are not enough in case of structures located in low seismic areas. As observed also in [14], the columns that carry only gravitational loading influence a lot the structural performance. Compartment scenarios significantly influence the results; even though the gravitational loading is smaller on the marginal compartment compared to the central one, the lack of additional boundary conditions leaded to increased damage and progressive collapse, as it was shown by the results of the analyses from the structure in low seismic area.

One key hypothesis that was considered for all analysis is that the diaphragm action corresponding to the slab was neglected for the thermal and column loss analyses. The main reason was to study only the strength and robustness capacity of the steel structure in this phase.

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