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FIRE SAFETY ENGINEERING –FROM THEORY TO PRACTICE

Abstract: Understanding the performance and the response of the structures in fire is of a particular importance for structural fire design. According to Eurocodes, fire is treated as accidental load on structures and structural elements have to be designed to withstand fire during the prescribed period of time. Fire induced temperatures cause reduction of the load-bearing characteristics of the constitutive materials of the structural elements and this effect directly reflects on the reduction of their bearing capacity. Stresses and strains caused by temperature additionally reduce the fire resistance of structures. This lecture is focused on the fire resistance and the behavior of different types of reinforced concrete elements and frame structures. By several case studies the influence of the fire on temperature distribution within the structural elements, the fire resistance of the whole structure, the bending moments and deformations of elements are presented and discussed.

Key words: fire, heat transfer, stress-strain analysis, RC structures, energy efficient facades

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1. INTRODUCTION

Fires are possible catastrophic events in buildings with serious consequences on building`s operation, stability, bearing capacity, etc. In the past, the design of structures was made with respect to the: self weight, imposed loads, wind, snow and, in seismic regions, the seismic action. Fire as an action on structures was not considered. As a consequence of many fire incidents, where parts or whole buildings collapsed, the engineer's way of thinking and sense of logic was changed. Treating fire only through architectural and urban design recommendations and fire protecting elements with isolation materials was not enough. There was a necessity for understanding the behavior of: construction materials, structural elements, assemblies and whole structures, under fire exposure.

According to Construction Products Regulation 305 from 2011 the structures have to fulfil not only the basic requirements regarding the bearing capacity, stability and serviceability, but also the requirements concerning the fire safety of structures. Other requirements are: hygiene, health and the environment; safety and accessibility in use; Protection against noise; energy economy and heat retention and sustainable use of natural resources.

According to the fire safety requirements, the construction works must be designed and built in such a way that in the event of an outbreak of fire: the load-bearing capacity of the construction can be assumed for a specific period of time; the generation and spread of fire and smoke within the construction works are limited; the spread of fire to nearby construction works is limited; occupants can leave the construction works or be rescued by other means; the safety of rescue teams is taken into consideration [1,2,3].

Achievment of all these requirements is possible by a**ctive fire protection measures,** by using alarm systems, sprinklers, installations for destinguishing the fire etc, by **passive fire protection measures,** that means by using appropriate structural sysems, appropriate materials, etc. and by **risk management in case of fire.**

The Eurocodes, as most up to date standards in the field of design of structures, have separate parts that deal with the design of concrete, steel, timber, composite, masonary and aluminium structures for accidental situation of fire exposure. This specialized EC parts define the fire as a load, give the material characteristics and their behaviour at elevated temperatures, prescribe the calculation methods and procedures for analysis of structures exposed to fire [1,2]. The Eurocodes cover only the passive fire protection measures.

Fire resistance of an element, of a part, or of a whole structure is ability to fulfil the requirements for a specified load level, for a specified fire exposure and for a specified period of time. According to Eurocodes, three criteria are given to define the fire resistance of structures or structural elements. These are: Load bearing function (R); Integrity (E) and Insulation (I). The criterion Load bearing function (R) expresses the ability of the structure or the member to sustain specified actions during the relevant fire, according to defined criteria. The criterion Integrity (E) expresses the ability of the separating element of the building construction, when exposed to fire on one side, to prevent the passage of flames and hot gases through it and to prevent the occurrence of flames on the unexposed side. The criterion Insulation (I) expresses the ability of the separating element of the building construction when exposed to fire on one side, to restrict the temperature rise of the unexposed face below specified levels. Criterion "I" may be assumed to be satisfied where the average temperature rise over the whole non-exposed surface is limited to 140 K, and the maximum temperature rise at any point of that surface does not exceed 180 K. Not always all these trhree criteria have to be fulfiled. In depends on the type of the element.

The behavior of a member, as well as of the entire structure, in case of fire depends on many factors, such as: the fire exposure, it means the intensity and duration of the fire action, the position of the elements in relation to the fire compartment, it means the fire scenario, the structural system and support conditions, the distribution and intensity of the existing loads, the size and shape of the elements, as well as the temperature dependence of the thermal and mechanical characteristics of the constitutive materials.

In fire conditions, reinforced concrete structures with concrete as dominant material, have an advantage over steel, timber or composite structures. This is due to the good performance of concrete at elevated temperatures. A contribution to this good performance is the concrete cover over the steel reinforcement, especially when it has sufficient thickness. The above stated makes reinforced concrete structures better fire resistant than the other structures [4-6].

However, different RC element types have different fire resistance. There are several factors that influence the element behaviour in fire, as:

- The type and the dimensions of the cross-section. The cross sections with larger dimensions have higher fire resistance (slower heat transfer into the section);
- Compressed elements calculated with lower percentage of reinforcement have higher fire resistance (smaller bearing contribution of the reinforcement that reaches high temperatures very soon after fire starts and looses the bearing capacity);
- Concrete cover thickness that protects the reinforcement of high temperatures and improves the fire resistance of elements subjected to bending;
- Different concrete mixtures have different behavior during heating i.e. calcareous concretes behave better than siliceous concretes [2].

The oldest procedure for defining the fire resistance of structural elemnts is conducting a standard fire tests in specially constructed furnaces. The other, more practical option for defining the fire resistance of structural elemnts, is by using calculation methods, and there are two options, simplified or numerical methods. Both options are offered in all Eurocodes that treats different materials.

Nowadays, in practice, only single structural elements are analyzed by using simplified calculation methods, while worldwide, the global structural response of buildings in fire conditions is accurately determined with proper use of advanced calculation methods.

This paper presents several case studies on fire resistance of RC elements and frame structures, by using advanced calculation methods.

2. CASE STUDY: FIRE RESISTANCE OF RC COLUMNS

The columns as structural elements have an important role in preventing loss of global stability of structures under fire. If these elements do not suffer failure, damages shall be of a local character, which shall enable evacuation and efficient extinguishing of the fire.

The fire resistance of centrically loaded RC columns depends on: shape and dimensions of the cross section; concrete cover thickness; type of aggregate; steel ratio, intensity of the axial force, as well as the fire scenario [4], but the dominant effects have the shape and dimensions of the cross section and the intensity of the axial force.

The set of columns with height h=3m, with same cross-sectional area of 900cm^2, same percentage of reinforcement of 1% and different column's side ratios: 30x30cm, 25x36cm, 20x45cm and 60x15cm (RC wall) were analyzed. The columns were exposed to ISO834 standard fire from all sides and all over the height. All columns were fixed at the bottom side and pin-ended on the other side and such support conditions were chosen to enable free expansion in longitudinal direction. As a result of the column's height and the dimensions of the cross sections, small slenderness ratios were achieved. In such case the support conditions had a negligible impact on the fire resistance of the columns and were not varied in this study. Siliceous aggregate concrete was used and the compressive strength at ambient temperature was $f_c(20°C)=30MPa$. The yield strength of the reinforcing bars at ambient temperature was taken to be $f_y(20°C)=400MPa$.

In order to define the influence of the load level on the fire resistance of the columns, the intensity of the axial compressive force was varied. The load coefficient α was defined as ratio between the applied axial force and the ultimate axial force of the column at ambient temperature. The load coefficient α was varied between α=0.1 and α=0.4.

Numerically achieved results for the temperature distribution in the cross sections of two types of columns, from the set of analyzed columns, for two characteristic moments, are presented on Figure 1 and Figure 2.

By comparison of the isotherms in the cross sections it could be found out that in case when the ratio between the two sides of the cross section is higher (for example 15x60cm), the temperature penetrates dipper in a shorter time period and the average temperature of the cross section is higher than in case when the ratio is less or equal to one (case of column 30x30cm). This results with higher reduction of material properties and reduction of the bearing capacity of the column. Consequently, the column has lower fire resistance.

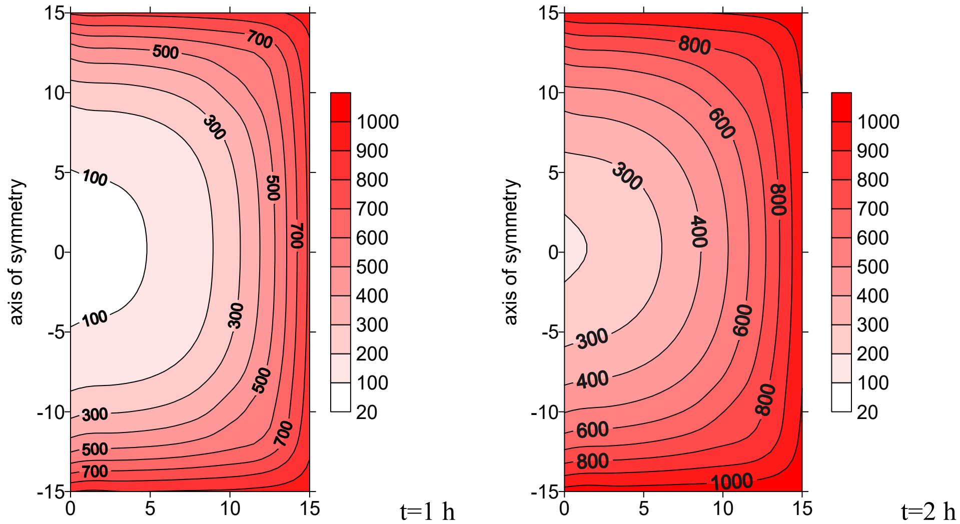


Figure 1 - Temperature distribution in the cross section of the column with dimensions 30x30 cm, for two characteristic moments

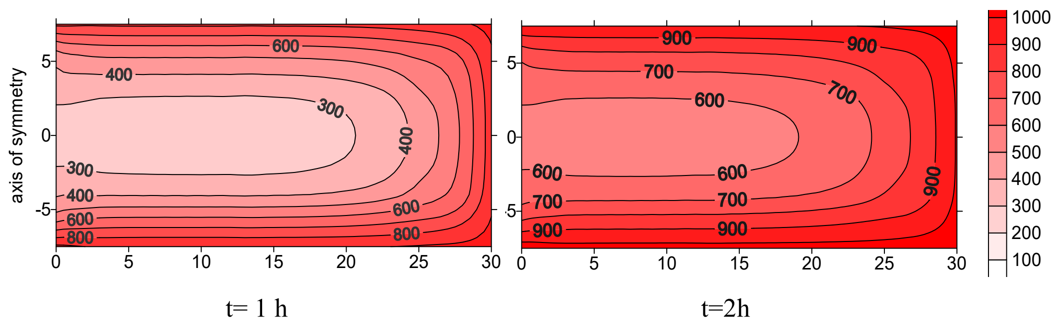


Figure 2 - Temperature distribution in the cross section of the column with dimensions 15x60 cm (RC wall), for two characteristic moments

As part of the analysis, besides the shape and the dimensions of the cross sections of the fire exposed columns, the intensity of the axial force was varied too. The load coefficient α was involved as ratio between the applied axial force and the ultimate axial force of the column at ambient temperature. The load coefficient α was varied between α=0.1 and α=0.5. This enabled comparison of the fire resistance of the analyzed set of columns with different shapes and sizes, for different load levels.

Numerically achieved results for the stresses in the concrete part of the cross sections of the two types of analyzed columns, for load coefficient α=0.35 and for two characteristic moments, are presented on Figure 3 and Figure 4.

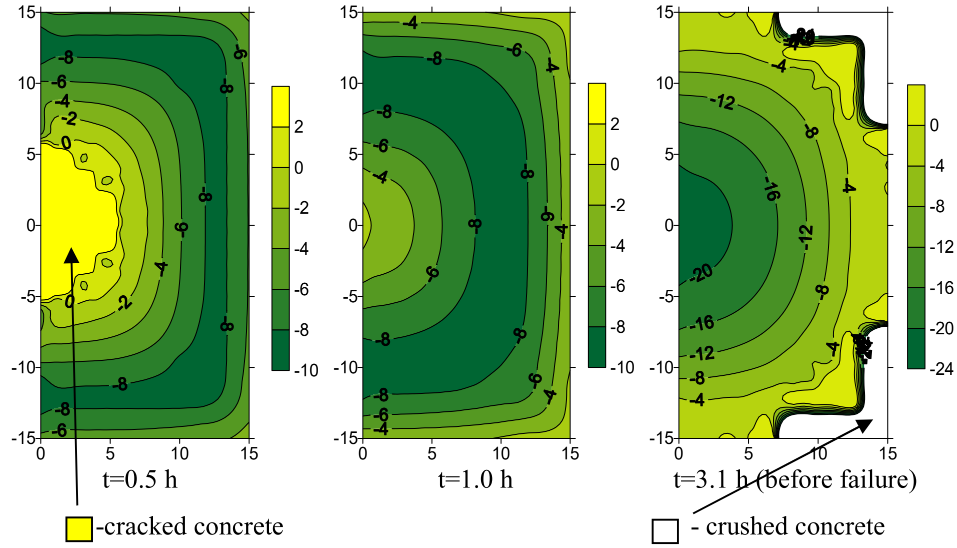


Figure 3 - Time redistribution of stresses in the cross section of centrically loaded RC column with dimensions 30x30 cm, exposed to fire from all sides, (load ratio $\alpha = 0.25$)

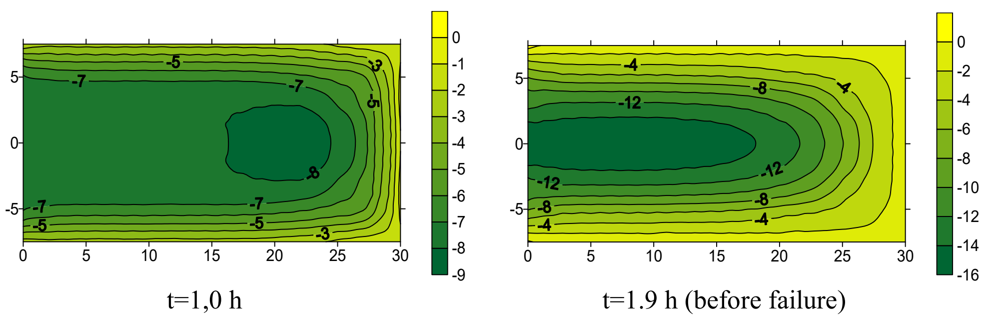


Figure 4 - Time redistribution of stresses in the cross section of centrically loaded RC column with dimensions 15x60 cm (RC wall), exposed to fire from all sides, ($\alpha = 0.25$)

At first moments of fire expose, because of the high temperature differences between the surface layers and inner layers and because free thermal expansion is not allowed, the concrete core cracks. This effect happens when load coefficient is less than 0.3 and is not so much expressed when the ratio between the two sids of the cross section is higher. For example, in case of column 15x60cm (Figure 4) this effect doesn't appear for load coefficient higher than 0.2. After a time high temperatures penetrates dipper into the cross section, mechanical properties of steel and concrete are reduced and the effect of the axial force becomes dominant, so cracks are closed.

For the analyzed set of columns, the fire resistance curves as function of the shape of the cross-section and the intensity of the axial compressive force were constructed (Figure 5). These curves indicate that the highest fire resistance achieves the column with lowest ratio between the two sides of the cross section.

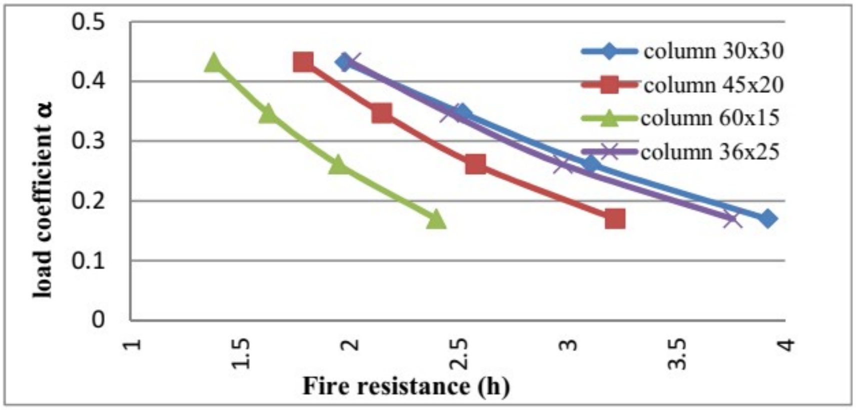


Figure 5 – Fire resistance curves for centrically loaded RC columns exposed to fire from all sides, as function of the shape of the cross section and the load coefficient α

The total axial dilatation of the columns as result of the total thermal elongation and the negative axial dilatation caused by the compressive axial force, for all four RC columns exposed to fire from all four sides and for the load coefficient α=0.35, are presented on Figure 6.

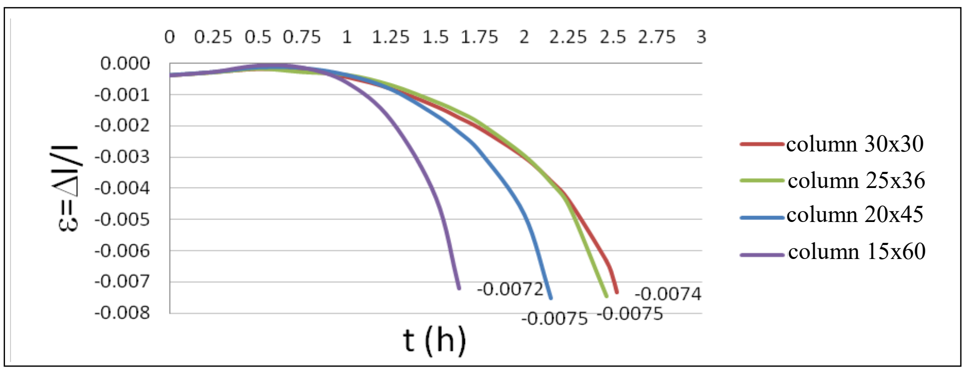


Figure 6 – Axial dilatation of the centrically loaded RC columns exposed to fire from all sides, for load coefficient α=0.35, as function of the shape of the cross section

If we analyze the total axial dilatation of the columns as function of the intensity of the axial force and the time of fire exposure, it can be noticed that for all four shapes of cross sections at the beginning of the heating process the positive dilatations caused by temperature are dominant. The negative dilatations caused by the axial force decrease with tendency to become positive, which is case for load coefficients less than 0.3. After a time the average temperatures of the column's cross sections become higher, the mechanical characteristics of the steel and concrete are reduced, the negative dilatations are dominant again and significantly larger than the positive dilatations caused by temperature. As a result of that, the columns fail in compression. The most sensitive is the cross section with dimensions 15x60 cm (due to a higher average temperature), and the best behavior shows the cross section with dimensions 30x30cm.

3. CASE STUDY: FIRE RESISTANCE OF ONE-WAY CONTINUOUS RC SLAB

In fire conditions, as a result of a large number of real fire tests and corresponding numerical analyzes, it was found out that the moment of failure of the beam and floor structures is always followed by significant deformations (deflections) [7-10]. If the structure is close to the limit state, after the cooling phase the residual deflections are so great that it cannot be used without significant rehabilitation. For these reasons, during the fire action, the deformation (deflection) of the slab is limited to prescribed value. According to the ISO standard, this limit value is L/30 (L is the span of the slab).

This case study presents the numerically achieved results for the fire resistance of one way continuous three span RC slab (Figure 7) in case of different fire scenarios (Figure 8).

The slabs are exposed to ISO 834 Standard fire. Separate stripes wide b =0.125m are analysed. The temperature dependent physical and mechanical properties of the siliceous aggregate concrete (compressive strength fck=30Mpa) and the reinforcement (yield strength fyk=400Mpa) are assumed according to EC2, part 1-2. The dead load is G1=1.5 kN/m2 (excluding self weight) and live load is Q=4.0 kN/m2, with y2,1=0.6 (for category C). The concrete cover thickness is c0=25mm.

The slabs are continuous over two supports. Only one suport is horizontally restrained, with the other supports are free to move lontitudinally. In all cases the bottom reinforcement is continuous through the slab. The length of the top reinforcement is presented in Figure 7.

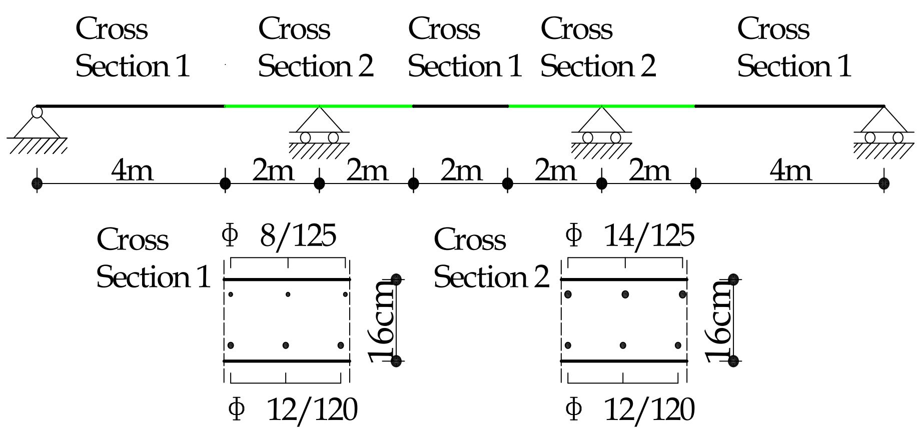
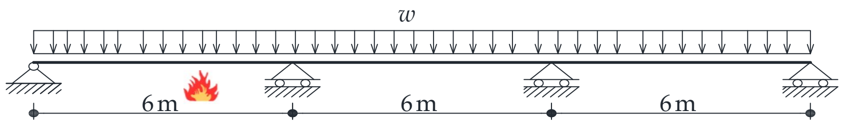


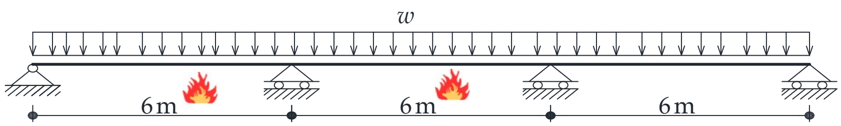
Figure 7 – Slab geometry and provided reinforcement

The analyses were conducted by using the software SAFIR [3] (2D Analysis). The fire resistance of the RC slab was defined by 'ultimate strength design' criteria, according to all recommendations given in EC2, part 1-2 [2]. The RC slab was constructed without thermal insulation. Fire was located in all three bays from top or bottom of the slab, or the position was varied as fire in the first exterior bay, exterior and middle bay and only in the middle bay (Figure 8).

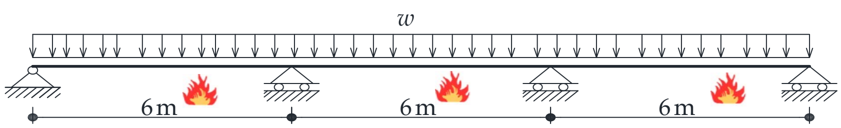
- Case I



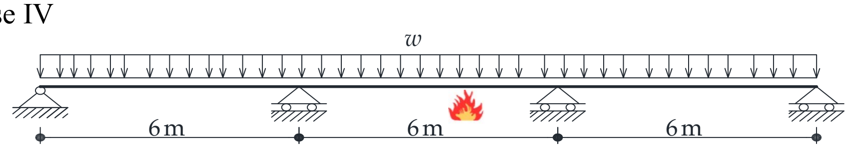
- Case II



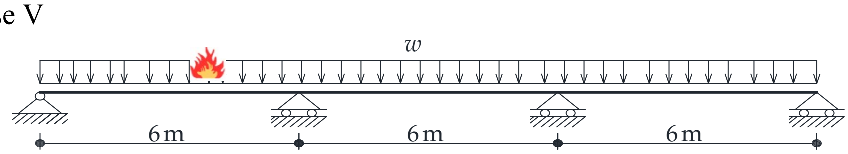
- Case III



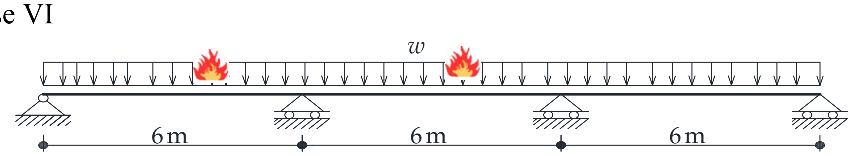
- Case IV



- Case V



- Case VI



- Case VII

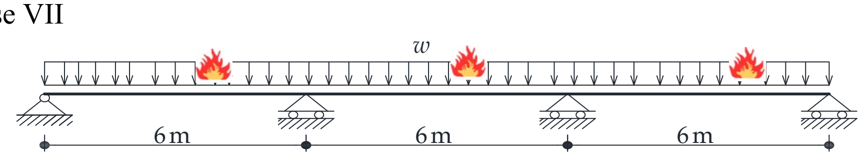


Figure 8 – Fire scenarios

Temperature distribution in the cross section of the analysed slab, at characteristic moments of fire exposure, are presented in Figure 9.

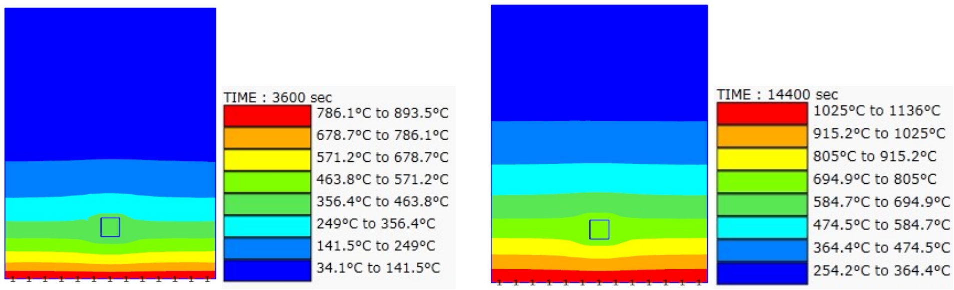


Figure 9 – Temperature profiles in the cross-section after 60 min and 240 min fire exposure from bottom side

In case when the fire is from the bottom side of the slab the bottom part of the cross section becomes hotter than the top and tends to expand more than the top (Figure 9). This differential heating causes the ends of the slab to tend to lift from the supports thus increasing the reactions. This action results in a redistribution of moments. The negative moments increase while the positive moments decrease and tend to become negative (Figure 10, case III). After a time, when the temperature penetrates dipper in the cross section and the temperature difference is reduced, the negative moments begin to decrease again, but slowly and never reach the initial values. In case of fire from the top side the effect is opposite (Figure 10, case VII).

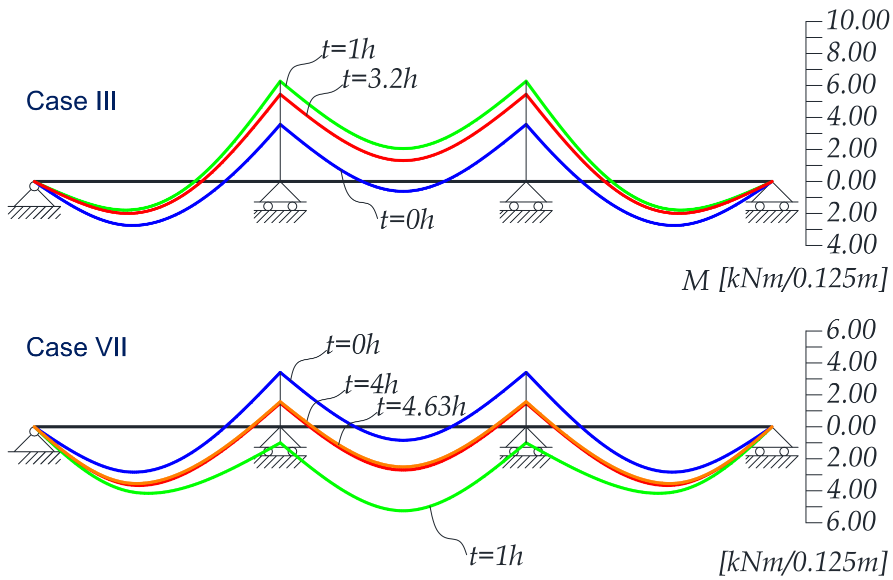
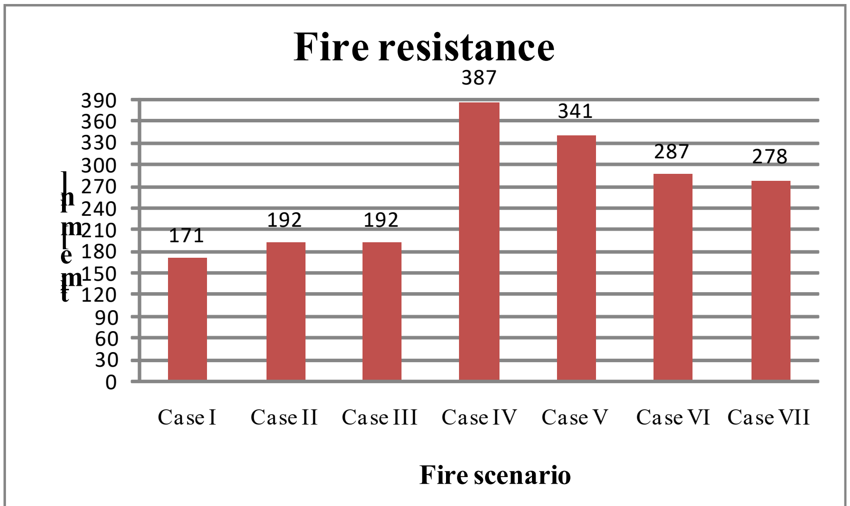


Figure 10 – Time redistribution of bending moments for two characteristic fire scenarios

When the fire is in the middle bay from the bottom side (case IV), the lateral displacements of the joints are restrained by the cold and stiff slabs away from the fire zone, so significant compression force develops in the slab above the fire. This axial force acts as a prestressing force and delays the moment when yielding of the reinforcement will occur, so it has a positive effect on the fire resistance of the slab (Figure 11). This is not a case when fire is in the exterior bay and free expansion is partly allowed.



Case	I	II	III	IV	V	VI	VII
Time	171	192	192	387	341	287	278

Figure 11 - Fire resistance of slabs for various fire scenarios

The concrete directly exposed to fire develops large compression stresses due to thermal gradients. At the supports of the slab fire has the same effect as the uniform load does, so concrete at the bottom of the cross section crushes, while concrete at the top of the cross section cracks. At the middle of the span the effect is opposite.

The temperature in the reinforcement directly influences the fire endurance of reinforced concrete elements. The concrete protects reinforcement from high temperatures, so bars which are close to the fire are hotter, and opposite. Thus the increase in negative moment at the supports can be accommodated, but the redistribution that occurs is sufficient to cause yielding of the top reinforcement. The resulting decrease in positive moment at the mid-span means that the bottom reinforcement can be heated to higher temperature before failure would occur. Deflections at the mid span at the moment of failure are presented in Figure 12, while the deformed shapes of the slab for two fire scenarios are presented in Figure 13.

The analysis presented in this paper show that fire scenario has important role in the fire resistance of RC slabs. The slabs exposed to fire from top side, have better fire resistance than they exposed from bottom side. The reason for this is that tension reinforcement at spans is not exposed to fire, and there is no decrease of its resistance. In the case IV, where the slab is exposed to fire from bottom side only at middle span, higher resistance is achieved than in all other fire scenarios. This happens because the lateral spans cause axial restrain to the middle span, thus causing a prestressing force to the slab.

The compressive axial force has positive effect on vertical displacements of the slabs too, they are much smaller than in case of unrestrained slabs (Figure 12).

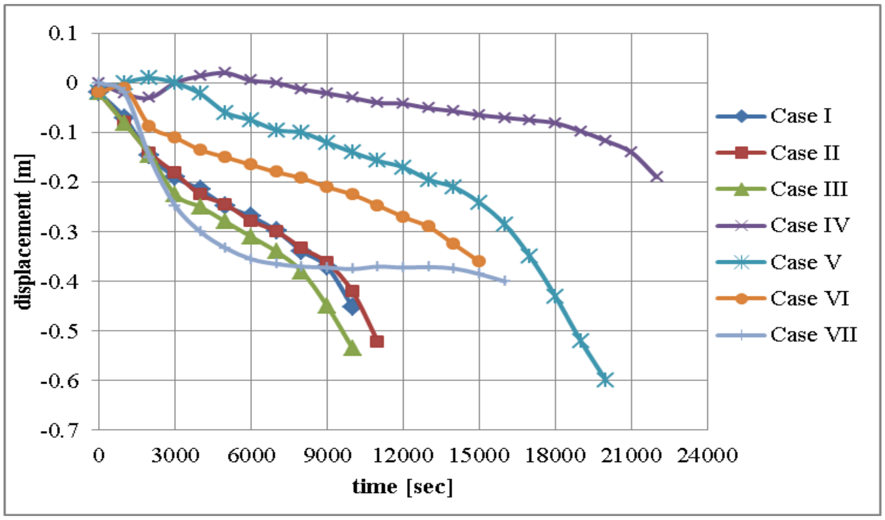


Figure 12 –Vertical displacements at location of maximum positive moment

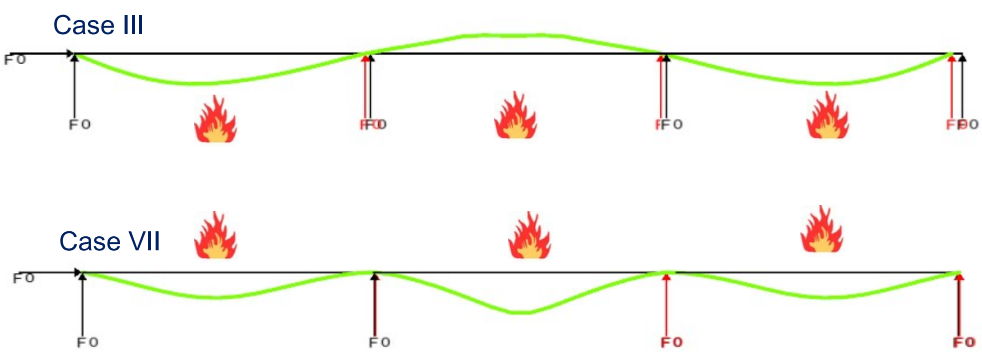


Figure 13 –Deflected shapes of slab at moment of failure for two specific fire scenarios

4. CASE STUDY: FIRE RESISTANCE OF RC FRAME STRUCTURE

This research was focused on the fire resistance and the behavior of fire exposed reinforced concrete frame structure [11]. In general, fire location is hard to be predicted and therefore the determination of the worst fire scenario, which has to be considered in the structural fire design, is very challenging. Different fire scenarios mean different locations of the fire, aiming to create worst but still realistic fire conditions. For this research the fire compartment was assumed to be one or two spans of same floor, or the whole floor.

For the purpose of this research twelve different fire scenarios were considered and the heating regime was defined with the standard temperature-time curve ISO-834. All fire scenarios were analyzed for two different load ratios ($q/q_u \approx 0.6$ and $q/q_u \approx 0.8$). The internal forces, displacements and failure modes were defined and comparison of results was made.

The three-bay three-storey RC frame, analyzed in this paper, is shown in Figure 14. The structure is made of concrete with siliceous aggregate, with a compressive strength f_c=30 MPa and reinforcing bars with a yield strength f_y=400 MPa. The RC structure is designed for load combination that includes seismic action, according to the national standards. The cross sections of all beams are 0.35x0.45 m^2 and the column sections are 0.40x0.40 m^2. The concrete cover thickness of all cross-sections is a=2 cm. Uniformly distributed load of 50 kN/m (for $q/q_u \approx 0.6$) and 67 kN/m (for $q/q_u \approx 0.8$), including self weight, is applied. Geometry, support conditions, reinforcement details and numeration of: bays, beams, columns and joints, are shown in Fig. 14.

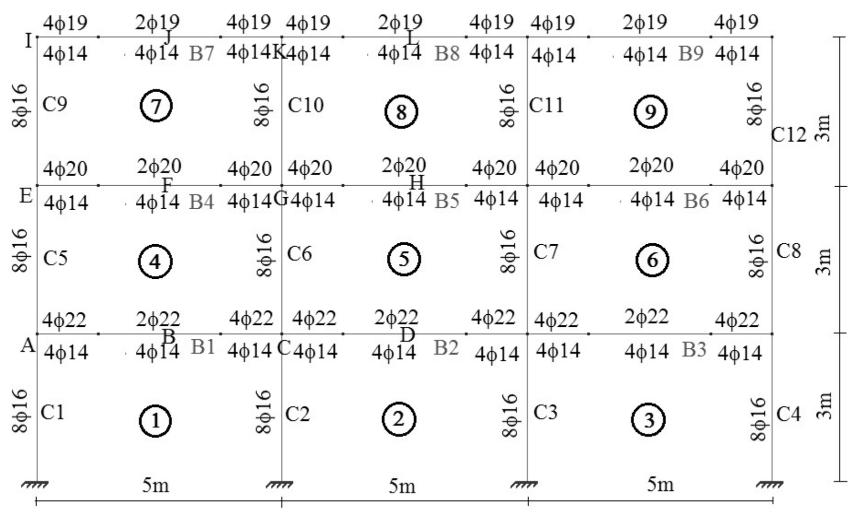


Figure 14 - Frame geometry, support conditions, reinforcement, numeration of: beams, columns, joints and bays

The material models for concrete and steel are the constitutive models given in Eurocode 1992-1-2 and Eurocode 1993-1-2, respectively. The 2D numerical analyses were conducted for temperature raise according to Standard fire curve ISO 834. Top compartment beams were assumed to be fire exposed on three sides (bottom, left and right side) and the bottom compartment beams were assumed to be fire exposed only on the top side.

The fire resistance for each analyzed fire scenario was calculated for two different load ratios and the results are presented in Table 1. In programme SAFIR there is no deflection criterion for defining the failure point, therefore the calculation goes on until the failure of the whole structure happens.

Table 1- Fire scenarios and corresponding fire resistances of the frame

Fire scenario	Spans involved in fire scenario	Fire resistance for q/q_u=0.6	Fire resistance for q/q_u=0.8
I	1, 2, 3	215.97 min = 3.60 hours	138.43 min = 2.31 hours
II	4, 5, 6	178.01 min = 2.97 hours	116.4 min = 1.93 hours
III	7, 8, 9	126.92 min = 2.12 hours	75.21 min = 1.25 hours
IV	1, 2	215.92 min = 3.6 hours	138.45 min = 2.31 hours
V	1	215.68 min = 3.59 hours	137.23 min = 2.29 hours
VI	2	No failure before 5 hours	200.08 min = 3.33 hours
VII	4, 5	177.27 min = 2.95 hours	116.22 min = 1.94 hours
VIII	4	179.29 min = 2.99 hours	116.97 min = 1.95 hours
IX	5	249.24 min = 4.15 hours	151.20 min = 2.52 hours
X	7, 8	127.84 min = 2.13 hours	75.97 min = 1.27 hours
XI	7	125.87 min = 2.10 hour	75.10 min = 1.25 hours
XII	8	149.49 min = 2.49 hours	92.86 min = 1.55 hours

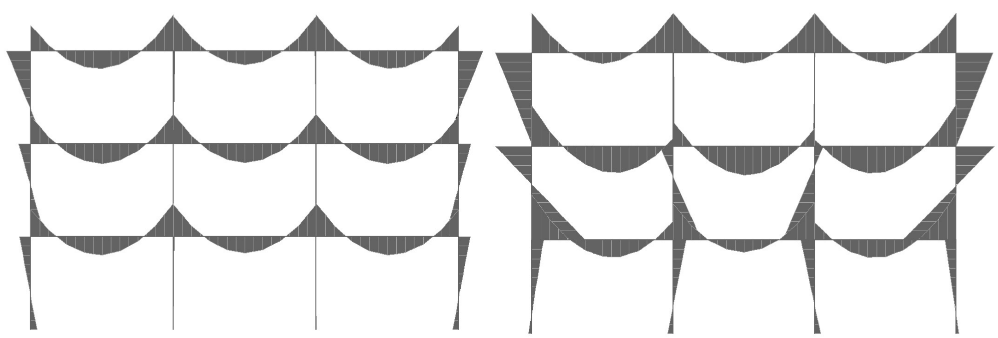
As expected, the frame with higher value for the load ratio reaches lower fire resistance. The higher the position of the fire compartment is the lower the fire resistance of the frame is. All fire scenarios involving some or all spans of the top floor result with lower fire resistance in comparison to the corresponding fire scenario at the lower storeys.

There is a negligible difference in the fire resistance of the frame in the case of a whole floor fire compartment and when the fire compartment involves only the outer span or both, the outer and the central span of the same floor. Exception to this conclusion is the case when the fire compartment comprises only the middle span of the floor. The fire resistance in this case is significantly higher than in case of other possible fire scenarios for the same floor, for reasons being explained in the sections below.

During the first minutes of the fire action, large thermal expansions of the fire exposed beams occur. Depending on the fire scenario these thermal elongations are more or less restrained by the adjacent bays. In the top beams of the fire compartment, in the heated parts of the cross-sections, these restraints result in high compressive forces that act as prestressing forces. The compressive axial force reduces the positive mid-span bending moment and delays the moment when yielding of reinforcement will occur, so it has a positive effect on the fire resistance of the frame [8]. When fire occurs only in the outer span of a floor, free outward expansion is allowed and the induced axial force is smaller and has a bit smaller effect. After the initial rapid increase of the compressive axial force, later in the ongoing fire process, the top beams experience reduction of compression due to heat penetration deeper in the cross sections, thus resulting in lower temperature differences but higher thermal elongations in comparison to the bottom beams. Exceptions are the top beams in the cases when the fire compartments comprise only the middle span of a floor. In such cases the compression force increases in time. This is the main reason why in the cases of fire only in the middle span (fire scenarios VI, IX and XII) the fire resistances are much higher compared to the cases when the fire is located in the outer span or spread through the whole floor.

All top beams of a fire compartment (fire exposed on the bottom and on both sides) tend to lift up during the first minutes of the fire exposure because of the high temperature differences in the cross-sections and the pressure in the lower heated parts of the beam cross sections. The negative bending moments at the ends of these beams decrease and the positive mid-span bending moments tend to become negative i.e. redistribution of moments occurs [4, 11]. The time when major redistribution occurs depends on the fire scenario and the dimensions of the cross sections. Later, the temperature penetrates deeper in the cross sections, the temperature differences between the bottom and the top sides become smaller and the bending moment diagrams tend to return to the shape as for the moment t=0 sec, but never reach the initial shapes (Figure 15).

The lateral displacements at the top levels of the fire compartments and the deflections of characteristic beams at the moment of failure, for some of the analysed fire scenarios, are presented in Table 2.

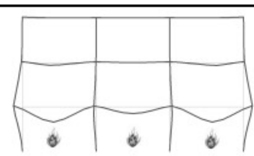
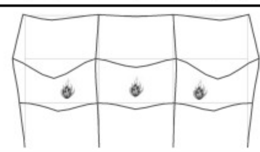
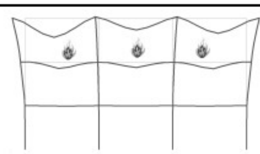
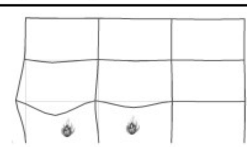
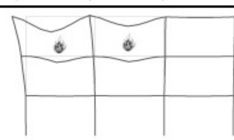
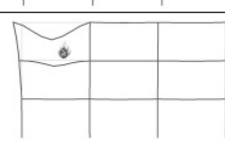
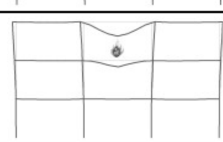


a) b)

Figure 15 - Bending moment diagrams for different time of the fire analysis, in fire scenario III a) t=24 sec b) t=126.92 min

As expected, the comparison of the lateral displacements at the top levels of the fire compartments in fire scenarios I, II and III, which represent whole floor fire compartments, lead to conclusion that the higher the floor is the bigger the lateral displacements are. When the fire compartment involves the outer and the middle span, the lateral displacement has its largest value and almost always the outer beam deflection has its smallest value. When the fire compartment involves the outer span only, the lateral displacement has its smallest value and almost always the outer beam deflection has its highest value. Where the fire compartment comprises only the middle span of a particular floor, the lateral displacements at the level of the top beams are smallest because of the cold and stiff outer spans of the frame.

Table 2. Deformations and displacements in characteristic joints of the frame

Fire scenario	Deformation at the moment of failure	Horizontal displacement of joints	Vertical displacement of joints
I		Δx_A=-5.98 cm	Δy_B = -10.37 cm Δy_D = -4.65 cm
II		Δx_E= -8 cm	Δy_F = -13.04 cm Δy_B = -5.47 cm Δy_H = -5.45 cm Δy_D = -4.09 cm
III		Δx_I= -8.58 cm	Δy_J = -13.01 cm Δy_F = -4.41 cm Δy_L = -7.85 cm Δy_H = -3.82 cm
IV		Δx_A= -6.72 cm	Δy_B = -10.15 cm Δy_D = -4.39 cm
V		Δx_A=-4.44 cm	Δy_B=-10.62 cm
X		Δx_I=-9.86 cm	Δy_J=-11.51 cm Δy_F=-4.61 cm Δy_L=-7.11 cm Δy_H=-4.09 cm
XI		Δx_I=-6.20 cm	Δy_J=-13.60 cm Δy_F=-4.31 cm
XII		Δx_I=-2.5 cm	Δy_L=-11.13 cm Δy_H=-4.89 cm

5. CASE STUDY: FIRE RESISTANCE OF ENERGY EFFICIENT FLOOR STRUCTURES

Floor structures, as horizontal elements, have a very important role in providing bearing capacity, usability and stability of the building as a whole. Their proper selection and design, when they are exposed to different types of loads (mainly: permanent and variable), should provide stable and safe structure during the exploitation period.

In case of fire, floor structures do not have only load bearing function. In most cases they are used as elements for separating the fire compartment. Where compartmentation is required, the elements forming the boundaries of the fire compartment, including joints, shall be designed and constructed in such a way that they maintain their separating function during the relevant fire exposure (EN 1991-1-2, 2004). This shall ensure, where relevant, that integrity failure does not occur, insulation failure does not occur, thermal radiation from the unexposed side is limited.

Does the floor structure meet the required fire resistance criteria mainly depends on: mechanical and thermal characteristics of the materials used for the construction; initial loading level; support conditions; dimensions of the cross section and the fire scenario [12].

Mainly, the massive simply supported or continuous reinforced concrete slabs with different thickness are used for the multistory residential buildings. For the individual housing construction semi-prefabricated reinforced concrete slabs system FERT and STIRODOM (with infill of extruded polystyrene-XPS) are usually used. These types of slabs have load bearing capacity only in one direction and from that reason only simply supported slabs are analyzed in this paper. The RC slabs and the slabs system STIRODOM were constructed with and without thermal insulation at the bottom side of the slabs. In fire conditions, as a result of a large number of real fire tests and corresponding numerical analyzes, it was found out that the moment of failure of the floor structure is always followed by significant deformation (deflection). If the structure is close to the limit state, after the cooling phase the residual deflections are so great that it cannot be used without significant rehabilitation. For these reasons, during the fire action, the deformation (deflection) of the slab is limited to prescribed value. According to the ISO standard, this limit value is L/30 (L is the span of the slab).

For defining the fire resistance of the floor structures, the proper thermal characteristics of the applied materials should be taken into account. Comparing with the traditional one, the new contemporary materials are lightweight and have better thermal and acoustic properties, but it doesn’t mean that in case of fire the higher fire resistance should be achieved. Some of these materials (Styrodur, Styrofoam, etc) are thermally unstable when exposed to high temperatures.

The cross sections and the dimensions of the three different types of simply supported floor structures were defined according to the current standards and the technical characteristics of the producers [12]. (Figure 16). The following parameters, characteristics and conditions were assumed:

- The slabs were exposed to ISO 834 Standard fire only from the bottom side,
- A separate stripes with dimensions: b = 1 m, L = 6m (typical section) were analyzed,
- The temperature dependent physical and mechanical properties of the siliceous aggregate concrete (compressive strength f_c=30Mpa) and the reinforcement (yield strength f_y=400Mpa) were assumed according to EC2, part 1-2,
- Physical properties of other materials at ambient temperature were taken according to the values provided by the producers and are given in Table 3.

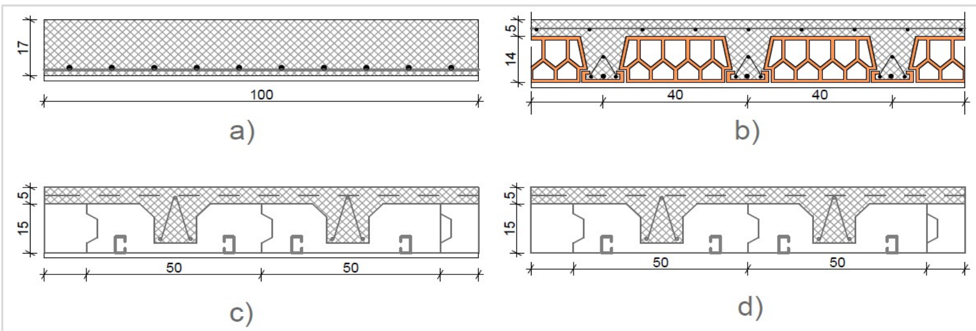


Figure 16 - Different types of floor structures: a) RC slab; b) slab system FERT; c) slab system STIRODOM with plasterboard as thermal insulation; d) slab system STIRODOM

Table 3 Thermal properties of insulation materials

Properties/material		brick	Plasterboard	EPS	Concrete	Reinforcement
Density	kg/m^3	1500	1000	30	2400	7800
thermal conductivity	W/mK	0.80	0.21	0.035	2.0	54
specific heat	J/kgK	920	1090	1450	960	440
Surface emissivity		0.93	0.85	0.90	0.92	0.69

For all types of floor structures the design loads (permanent and variable) at ambient temperatures were assumed to be the loads that cause vertical deformation equal to L/250 (EN 1992-1, 2004). These loads were considerably lower than the ultimate loads. For the selected types of floor structures the fire resistance in time domain is presented in Figure 17. The design loads that, at ambient temperatures, cause vertical deformation equal to L/250 are taken as 100%. All other loads are given as a percentage of these limited design loads (84%, 67% and 50%). Differences in the fire resistance of the certain types of floor structures are not significant except for the slab system STYRODOM with ceiling of plasterboard. This type of floor structure is more resistant to the effects of temperature and the fire resistance is much higher than for the other types of floor structures. The same structure, but without plasterboard at the bottom (ceiling) side (only thin plaster layer), has the lowest fire resistance. The reason for that is the melting of the infill of extruded polystyrene-XPS caused by temperatures over T=300°C. At temperatures T=450-500ºC the infill is completely burned and the temperatures in the cross section of the slab are much higher than in other three cases. Consequently, the deflection rapidly increases much more over the limited value L/30 .

Time dependent bending deformations (deflections) of the analyzed floor structures are presented in Figure 3. As a result of the excellent insulation properties of the plasterboard lining, the slab system STYRODOM with plasterboard at the bottom side has the highest fire resistance and smallest vertical deflection. During the heating period t = 60 min, as a result of the initial stiffness, the STYRODOM slab without plasterboard has small initial deflection, but after this time period, as a result of melting of the infill, the bending deformation (deflection) rapidly increases and after t=70 min reaches the limited value L/30. When fire exposure is less than 40 min the RC slab has higher stiffness and lower deflection then the slab system FERT. The thermal conductivity of concrete is twice higher than of the brick, respectively the temperatures in the cross section of the RC slab are higher then of the slab system FERT, respectively the stiffness of the RC slab decreases faster and after t=90 min both structures simultaneously reach the limited deflection L/30 (Figure 18).

The analysis presented in this paper show that from all three types of floor structures the RC slabs have the best performance at ambient temperature, as well as in case of fire. The performance of the slab system FERT when exposed to fire is satisfactory too, but we should not neglect its lower stiffness and greater deflections at ambient temperatures. The fire resistance of the contemporary floor structures (STYRODOM, ITONG, etc.) depends on the thermal insulation of the slab. The infill of extruded polystyrene-XPS is sensitive on temperatures over 300oC, therefore we should no avoid these structures, but it is necessary to provide protective measures.

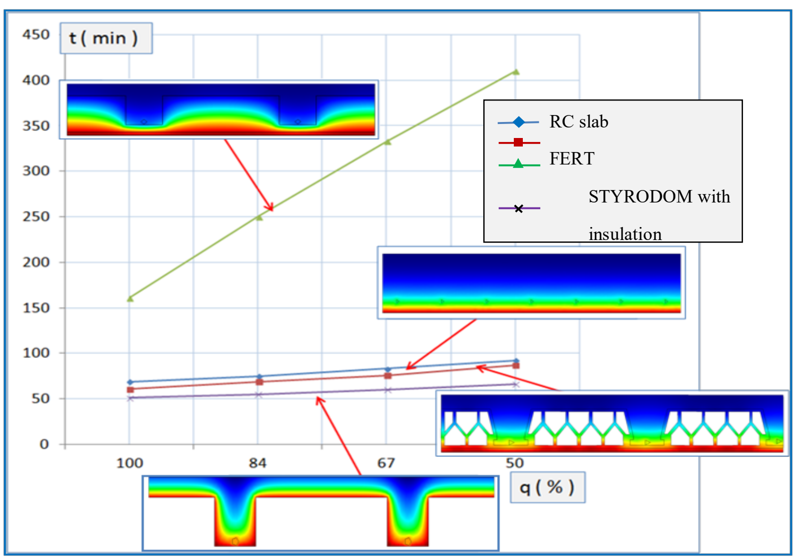


Figure 17 - Fire resistance of different types of simply supported floor structures, as function of the applied loads expressed as percentage of the design loads that cause deflections L/250

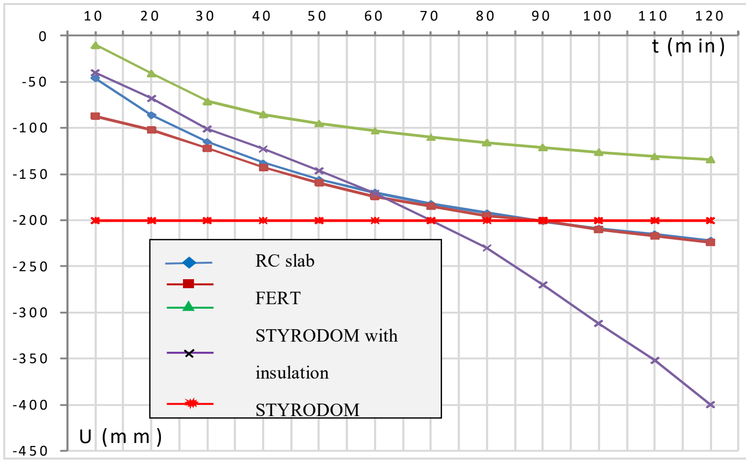


Figure 18 - Time dependent vertical deflections of different types of simply supported floor structures exposed to fire from the bottom side

6. CONCLUSIONS

The general conclusions are:

- Treating fire only through architectural and urban design recommendations and fire protecting elements with isolation materials is not enough.
- There is a necessity of understanding the behavior of fire exposed:

  construction materials,

  structural elements,

  assemblies and whole structures.

7. REFERENCES

[1] EN 1991-1-2 (2002). Eurocode 1: Actions on structures - Part 1-2: General actions - Actions on structures exposed to fire.

[2] EN 1992-1-2 (2004). Eurocode 2: Design of concrete structures - Part 1-2: General rules. Structural fire design

[3] SAFIR (2014). Computer program, University of Liege, Belgium

[4] Cvetkovska M. (2002). *Nonlinear stress strain behavior of RC elements and RC frames exposed to fire*. PhD. Thesis, Ss. Cyril and Methodius – Skopje, R. Macedonia.

[5] Huang, Z., Burgess, I. and Plank, R. (2006), "Behaviour of reinforced concrete structures in fire", *Structures in Fire, Fourth International Workshop*, Aveiro, May, 561-572.

[6] Buchanan A., 2001, "Structural design for fire safety", Wiley, New York, USA

[7] Ellobody, E., Bailey, C. (2009), “Modelling of unbonded post-tensioned concrete slabs under fire conditions”, *Fire Safety Journal*, **44**, 159-167.

[8] Lim, L. (2003): "Membrane Action in Fire Exposed Concrete Floor Systems", Doctoral dissertation, Departament of Civil Engineering, University of Canterbury, Christchurch, New Zealand

[9] Wang, G. (2006): "Performance of Reinforced Concrete Flat Slabs Exposed to Fire", Fire Engineering Report 06/2, University of Canterbury, Christchurch, New Zealand.

[10] Adam Levesque (2006). *Fire Performance of Reinforced Concrete Slabs*. Master Thesis, Worcester Polytechnic Institute

[11] Meri Cvetkovska, Milos Knezevic, Qiang Xu, Cvetanka Chifliganec, Marijana Lazarevska, Ana Trombeva Gavriloska (2018), *Fire scenario influence on fire resistance of reinforced concrete frame structure*, Procedia Engineering-ELSEVIER, Vol. 211, pp 28-35

[12] Meri Cvetkovska, Milivoje Milanovic, Marijana Lazarevska, Ana Trombeva Gavriloska (2015): *Fire resistance of energy efficient floor structures*, International Conference: Applications of Structural Fire Engineering-ASFE, Dubrovnik, Croatia

According to the lectures on **Fire safety engineering - from theory to practice**, anwer the following questions:

1. What does Safety in case of fire mean?
2. Stages of room fire. Which stage is important for the materials and which one for the structural behavior.
3. Criteria for Fire resistance according to Eurocodes for structures.
4. Which criterion the columns and beams have to fulfill?
5. Which criteria the nonbearing walls have to fulfill?
6. Parameters that influence the fire resistance of centrically loaded RC columns.
7. Parameters that influence the fire resistance of eccentrically loaded RC columns, beams and slabs.
8. Parameter that mostly influence the fire resistance of unprotected steel elements.
9. Why the thermal insulation helps in increasing the fire resistance of structural elements?
10. Compare the fire safety of facades made of EPS or XPS and stone wool.