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Load-Bearing Capacity Analysis for Reinforced Concrete Columns in Fire Conditions

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Abstract - **Depending on the risk of fire associated with a building, they must meet some minimum requirements associated with a level of fire stability. Usually, a tabular method is used to check concrete elements, but this method does not give information about the load-bearing capacity in case of fire. Determining the load-bearing capacity is a laborious process that requires knowledge of the temperature distribution in the section. Starting from the observation that the isotherms provided by Annex A of SREN 1992 are defined only for a limited number of cases and do not explicitly take into account the concrete cover, an analysis of the temperature distribution a was performed for a section of reinforced concrete column in the absence of reinforcement but also for two variants of the concrete cover. Using the results of the thermal analysis, it was analyzed how to modify the load-bearing capacity of a reinforced concrete column under the conditions of exposure to standard fire, for 90 minutes.**

Keywords – column's load-bearing capacity, standard calculation fire, strength and deformation properties according to the temperature, thermal analysis

1. INTRODUCTION

An accidental action with a major impact on the users of a building is fire. In this situation it is very important that the load-bearing elements maintain their stability and resistance for a period of time that allows the evacuation of users and ensures the safety of the intervention teams. The essential safety requirement in case of fire is ensured by active and passive fire protection measures.

In Romania P118/1 is the normative act that aims to ensure fire safety through compliance rules. The materials, installations and construction elements must meet the criteria and performance levels stipulated in this norm. For the load-bearing elements of the structures, their ability to keep for a given period of time, must be checked, at least equal to the level set in the normative, depending on the level of fire stability of the building. Performance is given by the duration for which the respective criterion is met, expressed in minutes, within the following standardized modules: 10, 15, 20, 30, 45, 60, 90, 120, 180, 240, 360. Eurocodes provide information that allows estimating if and how long a structure is able to withstand the existing loads in case of fire action.

Load-bearing capacity assessment for construction elements for the fire situation at a specified duration of time, t, and the comparison with the design effect of the actions in the accidental situation for fire exposure represents one of the methods of checking the fire resistance.

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A common practice for checking the fire resistance for reinforced concrete elements is the tabulated date method, which can only be applied for exposure to standard fire. The values in these tables have been established by an empirical basis and confirmed by tests and theoretical evaluation. For example, for column member, in these tables are proposed the minimum column width (b_{min}) and the minimum distance between the axis of the reinforcement and the surface of the concrete (a_{min}), depending on the mechanical reinforcement ratio (ω) and the load level at normal temperature condition (n). This method does not require the temperature distribution assessment for the member section and does not give information about the load-bearing capacity.

In order to determine the load-bearing capacity in the case of fire, the following must be solved in stages:

- gas temperature estimation in the fire compartment for time t, according to the duration specified by the criterion R, using nominal or parametric time-curves;
- temperature distribution in the cross section of the studied member at time t, depending on the specific rules for material and the gas temperature in the vicinity of the member;
- bearing capacity estimation according the modified properties of the materials corresponding to the temperature in the section.

Determination of the performance of the structure or for members would take into account the behavior of the structure at high temperatures, the potential fire risk and the beneficial effects of active and passive fire protection systems, together with the uncertainties associated with these three features and the importance of the structure. Even in the case of more simplistically procedures, based on standard fire curve, the classification system that provides specific durations of fire resistance takes into account these aspects.

2. CHECKS OF THE MECHANICAL RESISTANCE CRITERION FOR REINFORCED CONCRETE MEMBER EXPOSED TO FIRE

The description of the action in the fire situation is regulated by SREN1991-1-2 and involves the estimation of a fire design in a fire compartment, by a curve that shows the gas temperature evolution by time. There are the following alternatives:

- a standardized (nominal) curve that allows the analysis of structural elements for a specified duration, without considering the regression phase; standard temperature-time curve is constant for all material and all fire load densities;
- a fire model (simpler or more elaborate), based on physical parameters, which allows analysis throughout the fire, including the regression phase; for the parametric temperature-curve the growth stage varies with the opening factor and thermal properties of the boundaries; the max temperature is determined by the fire load density, opening factor and the type of occupancy and ventilation control.

In reference [1] the nominal curve and parametric curves that describe natural fire models are compared: “the standard temperature–time curve according to ISO 834 was developed summarizing data from fires in residential, office and commercial buildings. The curve should cover most of the potential courses of fires in common buildings. As fire tests have shown, the maximum temperature of a natural fires can exceed the ISO-curve, but after the maximum it decreases again, whereas the ISO curve rises continuously. More is described by taking into account the relevant boundary conditions (fire load, ventilation, geometry of fire compartment)”.

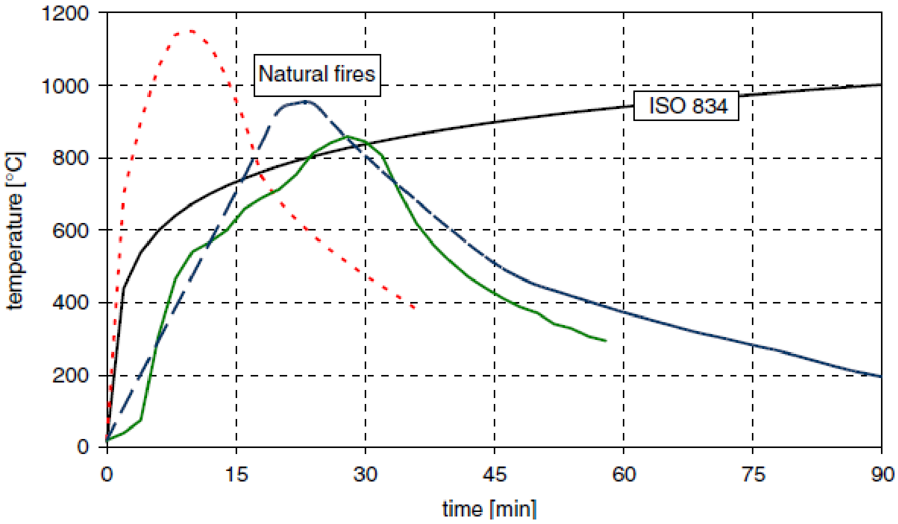


Fig. 1 Natural fire and ISO 834 standar fire [1]

Figure 2 shows the parametric fire curves plotted for a range of ventilation factors, fuel loads and materials according to the Eurocode. Fire curves were produced for three fire loads and two types of construction, showing the significant dependence of fire temperature on the thermal properties of the bounding materials [2].

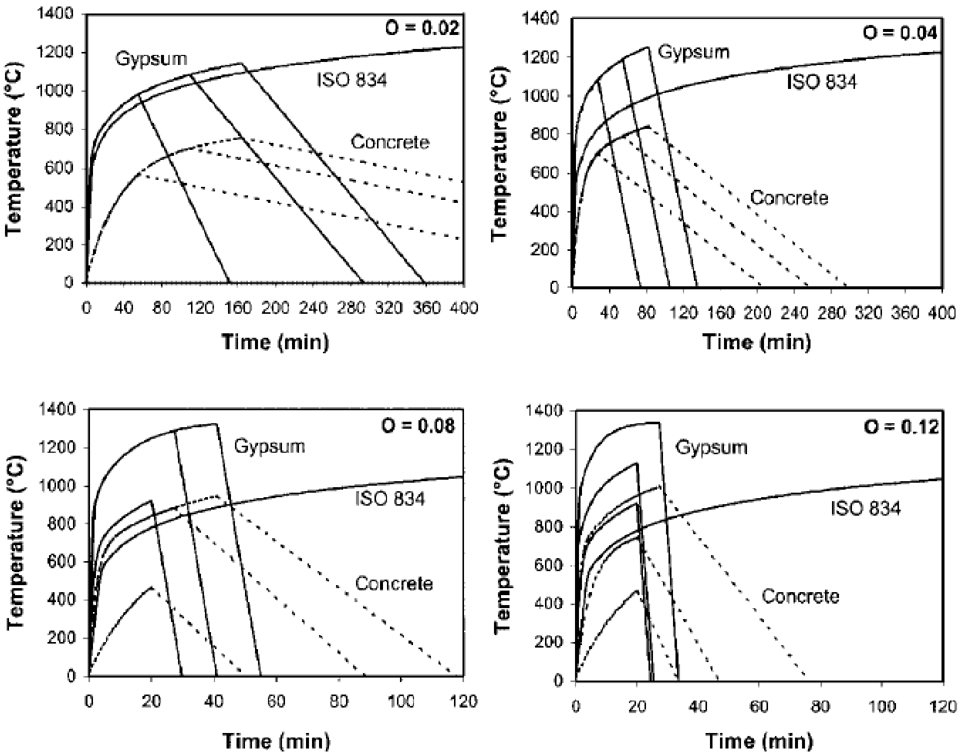


Fig. 2 Parametric time curve depending on the type of construction and opening factor [2]

In the design situation of exposure to fire will be considered the actions determined as for a calculation at normal temperature and without taking into account the decrease of the load as a result of combusting. According to the design codes, the simultaneity with other accidental situations is not considered, and actions resulting from industrial operations need not be taken into account. In order to obtain the effects of the actions in the case of fire, the mechanical actions are combined as specified in the relationship from CR0-2013, for accidental design situation.

The behaviour of concrete at high temperatures is relatively good, and at temperature below 300^{o}C, the fire effect is reduced. Concrete elements heat up unevenly, resulting large temperature differences between layers and there is a tendency to detach by separating the warmer layers from the colder ones [3].

The verification of the R criterion (mechanical resistance) for the case of exposure to standard fire is done by checking whether the load-bearing capacity is maintained during the required time of exposure to fire, while for exposure to parametric fire it is checked whether the collapse is avoided throughout the fire, including the period of decrease [5].

For the analysis of the load-bearing capacity in fire exposure conditions, a mandatory stage is the study of the temperature evolution during the fire. These can be determined by testing or by calculation. In support of this stage, reference [5] provides in Annex A temperature distributions in case of exposure to standard fire for a number of cases, more specifically for concrete column width/diameter of 30cm, for exposures to standard fire 30, 60, 90 and 120 minutes.

One of the simplified methods provided by SREN for checking the fire resistance is the 500^{o}C isotherm method. This proposes a general reduction of the section on the depth of the damaged concrete. It is considered damaged concrete, which no longer contributes to the bearing capacity of the element, the one with temperatures above 500^{o}C. For the reduced section, the method considers that the initial values of the resistance and of the elasticity module are preserved. The thermal analysis performed by a thermal transfer module with EF allows accurate assessment for the reinforcement's temperature and the conventional method can be used to determine the bearing capacity of the reduced section of the column, considering the modification of the resistance characteristics for reinforcements depending on the temperature. SREN1992-1-2 shows in section 3 the strength and deformation properties depending on the temperature, both for concrete and for reinforcement bars.

3. TEMPERATURE DISTRIBUTION IN THE SECTION BY THERMAL ANALYSIS

Starting from the observation that the isotherms provided by Annex A in reference [5] are defined for a limited number of sections and do not take into account the width of concrete cover, the temperature distribution analysis was performed for a section of concrete column without and with the rebar section and also for two case for concrete cover width. In a first stage for heat transfer model validation for a column width 30cm was analyzed, similar to the one in Annex A SREN1992-1-2. After validating the model by comparing the obtained isotherms, was made the analysis for a 55cm column, exposed 90 minutes to standard fire. We opted for a transient analysis that takes into account the variation of some parameters depending on the temperature, both for concrete and steel. Such an analysis can also be done to determine the evolution and temperature distribution in the concrete section for any other type of fire curve.

In order to establish the temperature distribution in the reinforced concrete section, attention was paid to the thermal properties of the two materials. For concrete, section 3.3 of SREN1992 was analyzed and it was considered in a simplified way a constant value of the calorific capacity 1100J/kgK (Figure 3.6a from SREN 1992-1-2) and for the thermal conductivity it was defined the dependence on the temperature according to the curve 2 of Figure 3.7. For steel, section 3.4 of SREN1993 was analyzed and both the dependence on the temperature of the specific heat and of the thermal conductivity (Figure 3.4 and Figure 3.5 of SREN1993-1-2) was considered in the model.

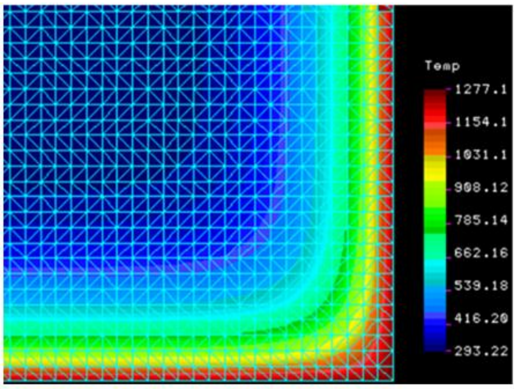
The thermal analysis also aimed at identifying the differences that occur if one takes into account the presence of steel bars of the longitudinal reinforcement with the afferent thermal properties, as well as the effect induced by the thickness of the concrete cover. Consequently, the following cases were analyzed:

case a - simple concrete section;

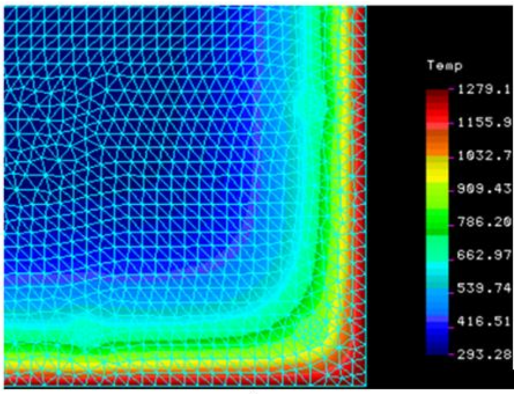
case b - reinforced section with 4 bars with 16 mm diameter of per side, with axis distance of steel bar from the nearest exposed surface 4 cm;

case c - axis distance of steel bar from the nearest exposed surface 5 cm.

The distribution of temperature per section, after 90 minutes, for *case a* and *case b* is shown in Figure 3. The temperature evolution for a corner rebar and intermediate rebar, for two values of the concrete cover can be traced in Figure 4.



case a



case b

Fig. 3 Distribution of temperatures per column section (55x55 cm) for a 90-minute exposure to the standard fire curve (temperature in K, for a quarter of section)

In order to assess the load-bearing capacity of the analyzed column, the temperatures in the center rebar are necessary; they are presented comparatively for the 3 cases studied in Table 1. In the Table 1 the position of the 500^{o}C isotherm for *case a* and *case* b was also presented.

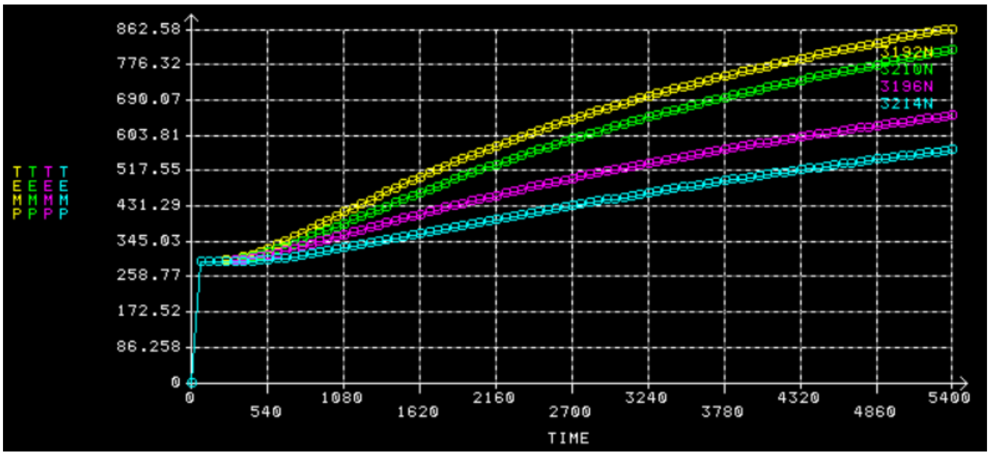
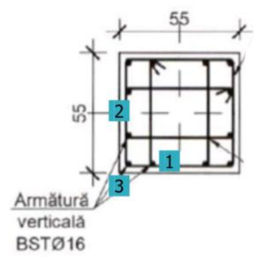


Fig. 4 Temperature evolution (time in seconds, temperature in K) in the nodes corresponding to the reinforcements: node 3192-case b corner rebar; node 3210-case c corner rebar; node 3196-case b middle rebar; node 3214-case c middle rebar

Table 1 Results of heat transfer analysis

	Case a	Case b	Case c	
Temperature for rebar 1	347 ^{o}C	377 ^{o}C	294 ^{o}C	
Temperature for rebar 2	347 ^{o}C	380 ^{o}C	294 ^{o}C	
Temperature for rebar 3 (corner)	557 ^{o}C	588 ^{o}C	534 ^{o}C	
Distance from the column's edge to the 500^{o}C isotherm	2.6 cm	3 cm		
Projection of the distance from the column's corner to the 500^{o}C isotherm	4.4 cm	5 cm		

It is easy to see that for a covering layer of about 3 cm (*case b*), the reinforced temperatures increase by less than 50^{o}C compared to the situation in which, as in Annex A of SREN1992-1-2, this aspect is not taken into account (*case a*). On the other hand, the insulating role of the cover concrete is illustrated by the results obtained in *case c*, where the temperature values are lower than in the case of the analysis in which the contribution of rebar steel was not considered. The results obtained illustrate that the corner reinforcements reach sensitive higher temperatures, with about 200^{o}C, which leads to a lower contribution to the bearing capacity of the section.

4. LOAD-BEARING CAPACITY ANALYSIS

The analysis performed by the author has the objective to determine how much becomes the moment capable for the above column, stressed by biaxial bending and compression, taking into account the modification of the strength and deformation properties at high temperatures. For numerical calculation it was considered the unfavorable situation of those studied at thermal analysis, the temperatures distribution in *case b*.

Section 3 of SREN 1992-1-2 indicates the temperature dependence of the strength and deformation properties for concrete and reinforcement.

The bending moments capable on the two directions were considered equal, due to the symmetry of the section and were determined in the hypothesis of the monoaxial eccentric compression, with design value of the applied axial force N_{Ed} [3, 4] and for verification the simplified criterion by SREN1992-1-1 (5.39) was applied [4].

Design effect of actions in accidental situation, for normal temperature, for the analyzed column are: $N_{Ed}=1223kN$, $M_{Edy}=25kNm$, $M_{Edz}=6kNm$.

For the same values of design value of the applied axial force and bending moments, after 90 minutes of exposure to standard fire, a conventional calculation of the load-bearing capacity was applied, but with the following changes from the normal temperature:

- The concrete section has been reduced, eliminating the thickness of the concrete layer with temperatures higher than 500°C.
- For the reduced cross-section of reinforced concrete a set of constant mechanical properties were considered: in one hypothesis they were considered mechanical properties identical to those at normal temperature (*case d*); another hypothesis, unfavorable, considered for the entire section of concrete reduced the properties of concrete from temperature of 500°C (*case e*).
- For reinforcement bars were considered strength and deformation properties dependent on both temperature and type of stress (tension or compression), as recommended in section 4.2.4.3 of SREN 1992-1-2.

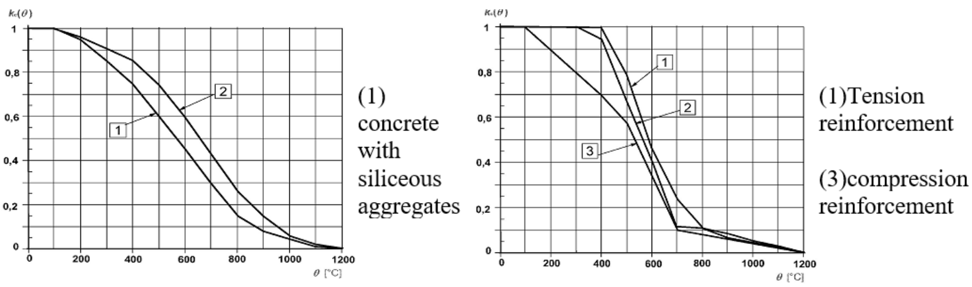


Fig. 5 Coefficients for reducing concrete and reinforcement strength [5]

In Table 2 are presented comparatively the results obtained for the accidental situation at normal temperature (*case b*), and after exposure to fire standard (*case d and case e*).

Table 2 Results of load-bearing capacity analysis

	case b	*case d*	*case e*
N_{Ed} [kN] design value of applied axial force	1223	1223	1223
M_{Edy} [kN m] design value of bending moment around y axix	25	25	25
M_{Edz}[kN m] design value of bending moment around z axix	6	6	6
M_{Rd} [kN m] desingn moment resistance	405	278.8	201.7
N_{Rd} [kN] design axial resistance of section	5765	4521	2920
x [cm] neutral axis depth	18.5	20.5	30

5. CONCLUSIONS

The thermal analysis performed proves that the isotherms in Annex A SREN1992 are conservative for a concrete cover higher than the one corresponding to the normative requirements for exposure classes XC2 XC3. It was found that the temperature increase in the reinforcement axis obtained for the model that considers a concrete cover of 32 mm is small enough (about 50^{o}C), which means that it does not significantly influence the reduction coefficients corresponding to the high temperatures. For an increase in the thickness of the coating layer by 1 cm, a decrease of about 50^{o}C of the temperatures in the axis of the reinforcements is obtained compared to the much simpler model, which considers only the concrete section.

Considering that the strength properties of steel diminish rapidly at temperature rise, the analysis of the bearing capacity emphasized the exact evaluation of the contribution of the reinforcements to the resistance of the reinforced concrete section in conditions of exposure to fire; has been applied a conventional calculation methods to determine the moment capacity considering for each bar the modified strength and deformation properties according to the temperature resulted from the thermal analysis. It has been found that the capable moment under standard fire exposure conditions for 90 minutes represents between 50% and 68% of that determined under normal temperature conditions. Even in the conditions of this diminution, the design load-bearing capacity (axial compression and bearing moment) in the fire situation is greater than design effect of actions in the fire situation.

6. REFERENCES

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