Trakya University Journal of Engineering Sciences
http://dergipark.gov.tr/tujes
ISSN 2147–0308

Research Article / Araştırma Makalesi

BOUNDARY FIRE LOADS OF REINFORCED CONCRETE COLUMNS EXPOSED TO A REAL FIRE, PART II

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Cite this article as:

Haksever, A. & Haksever, A. (2019). Boundary Fire Loads of Reinforced Concrete Columns Exposed to a Real Fire, Part II. *Trakya University Journal of Engineering Sciences*, 20(2), 35-49.

Highlights

- Prediction of fire development and structural failure in real fires. Heat balance calculations.
- Behavior of reinforced concrete columns (RFCs) under real fire action. Determination of the fire resistance time.
- Determination of boundary fire loads for (RFCs).

Article Info

Article History:

Received:
February 3, 2019
Accepted:
November 6, 2019

Keywords:

Structural fire safety, reinforced concrete columns, fire resistance, real fire, boundary fire loads

Abstract

Deformation behavior of the structural elements in fire case is affected by many additional parameters. The researches carried out so far have shown clearly, that the statically indeterminate systems may have an increased fire resistance due to redistribution of the internal forces. The calculations and the experiments have proved also that, in these structural systems, the joint forces increase during the fire and after reaching a maximum; these forces decrease until the collapse state is reached. The present paper is aimed at investigating the structural behavior of reinforced concrete columns (RFCs) in an enclosure exposed to real ventilation controlled fires, e.g. fires with a heating and a cooling phase.
In this research work max. and min. boundary fire loads of reinforced concrete columns in an enclosure for real fires are determined. Beyond these fire loads no failure or longer fire resistance time than a ISO834 fire can be expected. Beside that for RFCs, graphs and methods for determining fire behavior and resistance have been introduced by using II. order theory.

TABİİ YANGINLARDA BETONARME KOLONLARIN SINIR YANGIN YÜKLERİ, BÖLÜM II

Makale Bilgileri

Makale Tarihçesi:

Geliş:
3 Şubat 2019
Kabul:
6 Kasım 2019

Anahtar Kelimeler:

Yangın güvenliği, betonarme yapı kolonları, yangın dayanımı, tabii yangınlar, sınır yangın yükleri

Öz

Yangın durumunda yapısal elemanların deformasyon davranışının birçok ek parametreden etkilenmektedir, Şimdiye kadar yapılan araştırmalar, hiperstatik çubuk taşıyıcı sistemlerin yangın durumunda, iç kuvvetlerin yeniden dağılımı nedeniyle artan yangın direncine sahip olduğunu göstermiştir. Teorik analizler ve deneyler, bu yapısal sistemlerde, düğüm noktası kuvvetlerinin yangın sırasında maksimum bir değere ulaştıktan sonra azalma eğilimine girdiğini ve bu durumun sistemin göçme durumuna kadar devam ettiğini göstermiştir. Bu çalışma, betonarme kolonların yangın davranışlarını ve dayanımlarını tabii ventilasyon denetimli yangınlarda, diğer bir deyişle, ısınma ve soğuma fazına sahip kapalı bir ortamdaki yangınlar için incelemeyi amaçlamaktadır.
Bu araştırmada, bir kompartman içindeki betonarme kolonların sınır yangın yükleri belirlenmiştir. Bu sınır yangın yüklerinin dışında kalan yangın yükleri için, bir ISO834 yangınından daha uzun bir yangın dayanımı saptanmakta ve göçme durumuna rastlanmamaktadır. Bu çalışmada betonarme kolonların, II. Mertebe teorisini kullanarak, yangın davranışlarını ve dayanımlarını belirlemek için yöntem ve grafikler verilmiştir.

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1. Introduction

A reasonable design level for risk to life and risk to neighbouring property may be determined by the risk levels contemplated in current codes or other specific requirements. A decision level for the directly exposed property risk is based on economic considerations and should thus be the property owners' decision.

Structural fire design is concerned mainly with the prevention of fire spread through separating vertical and horizontal partitions and avoidance or limitation of structural failure or damage referring to fires when a collapse state to be controlled as an initial stage. The basic unit for structural fire design is the fire compartment or fire zone.

It follows that, for particular types of buildings and occupancies or particular projects, structural design requirements may be dispensable, because the associated risks are sufficiently small. Certain requirements may also be dispensable, because the risk-reducing effect of structural measures may be extremely low. Finally, structural design requirements may be eased to zero or a specified minimum in view of the non-structural measures used (s. Sec. 4.3).

The fire protection design and assessment of structural members is currently still carried out on the basis of standard fire tests. In this case, the structural construction to be examined is subjected to a standardized fire exposure and tested for their structural behavior and their fire resistance duration. This method primarily ensures comparability and any reproduction of the tests for the fire behavior of various structural members. For this reason in ISO 834-Fires, the temperature-time curve is thus regulated.

The fire protection regulations for buildings based on such standard fire tests ensure undoubtedly a certain level of safety. However, the ability to construct economically is limited by the current regulations. This restricting reveals itself in particular thing that the current fire protection design only knows the relatively an unfavourable temperature development as the standard fire curve. On the other hand, the fire exposure in real fires is sometimes less intense because these have lower or favourable fire room temperature developments.

The scope of this article is to investigate the structural behavior and fire resistance duration of the columns of a reinforced concrete hall building in a real fire concerning failure damage and to compare it with standard fire. This should provide practical information about the load-bearing and deformation behavior of RFCs that are exposed to a real fire.

2. Theoretical background/experimental

Concerning literature, it is rare to find fire test results carried out in big enclosures and theoretical analyses on the structural columns under the exposure of a real fire. The tests are conducted mainly for isolated steel columns and to determine the failure time t_f and derive some applicable formulas.

Due to the fast developments of large-space multi-functional architectures, large-span steel structures have been widely used in recent years. Therefore, the fire-resistance design of this kind of structures has attracted more attentions. Since traditional ISO 834 standard fire curve is not suitable for large space structures, performance-based fire resistance design method is required. These analyses presented herein will also show that common methods of defining fire intensity through equivalent fire durations do not appropriately account for the complexities of the thermal and structural response of concrete columns exposed to a travelling fire. A more realistic description of fire scenarios is still needed for a performance-based structural fire design, based on a better consideration of ventilation conditions and thermal properties of boundary enclosure Many studies of the thermal and structural behavior for large compartments in fire carried out over the past two decades show that fires in such compartments have a great deal of non-uniformity, unlike the homogeneous compartment temperature assumption in the current fire safety engineering practice (Hagen, 1987). Furthermore, some large compartment fires may burn locally and tend to move across entire floor plates over a period of time. For the actual boundary conditions the ISO 834 Temperature–Time curve overestimates the temperatures of the natural fire in the enclosures and leads to an inefficient design of structural members. Within a performance-based fire design concept the structural design should be based on a real design fire which is representative for the boundary conditions of a given building. The “traditional way” of structural fire design using the ISO 834 Temperature–Time curve in many cases results in a design on the safe side causing unsatisfactory costs for fire protection measures. In some cases the structural fire design with ISO 834 curve can result in under-estimation of the thermal exposure. Some research articles carried out in this context will be presented below.

Patrick Bamonte, etal, (2018) aims at investigating the structural behavior in bending of prestressed concrete members exposed to real fires, e,g. fires with a heating and a cooling phase. The fire scenarios considered are characterized by a heating phase that coincides with the ISO 834 standard fire and a linear cooling branch. Parametric analyses are carried out on typical pre-stressed sections (an I-girder and a double-tee). The results show that limiting the attention to the heating phase is not sufficient. Moreover, within the range of variation of the cooling rate (3–10 °C/min, ranging from slow to fast cooling) and load level ($M/M_u = 0.15–0.30$, ranging from low to high load ratio), the structural behaviour exhibits significant variations in the cooling phase of the fire, from an almost complete recovery of the initial configuration to runaway failure

Rush David, etal (2017), present some selected results from the thermal environment around, and the thermal response of a concrete column from a large scale structural fire test conducted in Tisova, Czech Repub-lic, inside a four-storey concrete frame building, with concrete and composite deck floors. From the results of the fire test, assessments of the fire intensity are made and used to model the potential thermal profiles within the concrete column and the implications that high temperature might have on the post-fire response of the concrete column. These analyses presented there will also show that common methods of defining fire intensity through equivalent fire durations do not appropriately account for the complexities of the thermal and structural response of concrete columns exposed to a fire.

Kordina, K.(1991), indicates that at present, the direct analytical design procedure is limited in many coun-tries to extraordinary structures or buildings, like tun-nels or industrial plants of great importance. In these cases the actual or design fire load is used within a heat and mass balance calculation, in order to determine the gas temperatures and the fire progress. Consequently, the mechanical and thermal behaviour of the structural elements surrounding the fire com-partment has to be determined.

Kan Zhou, etal. (2018), emphasize that extensive literature is available on the structural performance of concrete-encased CFST columns at ambient tempera-ture, however, the fire performance of this type of composite columns has seldom been addressed. To fill in this research gap, the paper thus experimentally studies the performance of concrete-encased CFST columns subjected to full-range fire including heating and cooling. A set of tests were conducted and results are presented, including fire resistance, post fire resid-ual strength, failure modes, temperature versus time relationships and deformation versus time relation-ships. The simple calculation model of analysing the fire resistance of composite column in Eurocode 4 was extended which is applicable to assess the fire resistance of concrete-encased CFST columns.

João Paulo C. Rodrigues, etal.(2010)**,** present in their paper the results of a research program on the behavior of fibre reinforced concrete columns in entire fire. Several fire resistance tests on fibre concrete columns with restrained thermal elongation were carried out. The aim of this research was to study the possibility of replacing the longitudinal reinforcement bars on the concrete columns by steel fibres. For this reason, polypropylene fibres were used in order to enhance the fire behavior of the columns and avoid the concrete spalling.

GuobiaoLou etal.(2018), present experimental and numerical investigations on the collapse behavior of a 12 m × 6 m steel portal frame exposed to fire. A real fire test is conducted with a 4 m × 6 m fire com-partment at the corner of the frame. Extensive thermal and structural responses of the frame are measured and presented. It is found that the measured gas temperatures are higher than the ISO fire, but lower than the parametric fire specified in EN 1991-1-2, indicating the underestimation of the thermal ex-posure for standard fires and unrealistic estimation for parametric fires. It is suggested that a more realistic description of fire scenarios is still needed for a performance-based structural fire design, based on a better consideration of ventilation conditions and thermal properties of boundary enclosure

Ju-young Hwang, etal.(2018), introduce in their paper to precisely simulate the structural response with temperature, the material properties of concrete and steel according to two representative temperature conditions, "under-fire" and "after-cooling", have been taken into account. Moreover, the importance of the after-cooling analysis to ensure the safety of fire-damaged RFC structures has been shown. Finally, through a comparison of the numerical results with the design code EN1992-1-2, it has been concluded that the design code should consider the influence of temperature decrease after experiencing high temperature to ensure the safety of fire-damaged structural members.

Yih-Houng Chen, etal.(2009), report an experimental research into the effect of fire exposure time on the post-fire behavior of reinforced concrete columns. Nine full-size reinforced concrete columns (45×30×300 cm) with two longitudinal reinforcement ratios (1.4% and 2.3%) were unexposed and exposed to the ISO 834 standard fire for 2 and 4 h with a constant preload. One month after cooling, the specimens were tested in axial load combined with uniaxial or biaxial bending. The test results show that the residual load-bearing capacity decreases with increase in fire exposure time. The authors insist that much attention should be given to the deformation and stress redistribution of the reinforced concrete building subject to earthquakes after a fire.

Limin Lu, etal.(2017), indicate in their paper that since traditional ISO 834 standard fire curve is not suitable for large space structures, and a performance-based fire resistance design method is required. A finite element (FE) model has been developed using FE software ANSYS for modelling the structural behavior of an exhibition centre under different fire scenarios. Based on the results generated in this research some recommendations for the fire re-sistance design of large space steel truss structures have been proposed. (More references see "PART I., "Behavior of reinforced concrete columns of buildings exposed to a real fire, PART I, TUJES")

From the references given above it is being clear that a detailed analysis and interaction of fire loads with the fire resistance of RFCs in a real fire are not adequately introduced up to know. Mainly focus is concentrated on the cooling face experimentally and by the way some material models are developed. Be-side that heat balance calculation models are presented but validity of these models experimentally is not shown as in the SFB C3 subproject as big scale compartment fires (Hagen, 1987).

The aim of this paper is, due to this fact, will be to investigate exemplary the structural behavior and fire resistance duration of the columns of a reinforced concrete hall in a real fire of damage and to compare it with standard fire. This should provide practical information about the load-bearing and deformation behavior of such RFCs that are exposed to a real fire.

As Part II, this research work presents the definition and the determination of boundary and safe fire loads $q_{f,B}$ and $q_{f,s}$ of RFCs. Part II will show the certain fire loads as wood cribs, show in real fire case equivalent fire resistance as in the ISO 834-Fire. Beyond the boundary fire loads the fire resistance of RFC is greater than the fire resistance in case of an ISO 834 fire case even no failure can be observed for the RFCs. For this purpose a concrete hall building is chosen and theoretically analysed by means of heat balance and statically calculations. The construction and the theoretical results will be presented in the following sections.

3. Hall building

3.1 concrete hall building and statically system

Fig. 3.1 shows the hall structure, which is investigated, in cross-section and plan view.

The bracing of the hall building is held by the rigid roof and the walls. The roof consists of 15 cm thick lightweight concrete slabs, the walls and the floor are to be constructed in normal concrete. The window openings arranged on both sides in the longitudinal walls ensure the cross ventilation of the hall building.

As a result, the ventilation conditions for the development of a fire are as natural ventilation defined. It is assumed that there is a constant fire load over the entire floor of the hall building. This fire load is converted according to Eq. 3.1 (s. “Behavior of reinforced concrete columns of buildings exposed to a real fire, PART I, Tujes”):

$$q_f = \frac{1}{A_f} \sum_1^n M_i \;.\; E_{ui} \;.\; (m_i) \qquad 3.1$$

to the wood cribs. Thus, the hall construction can be considered as a total fire compartment.

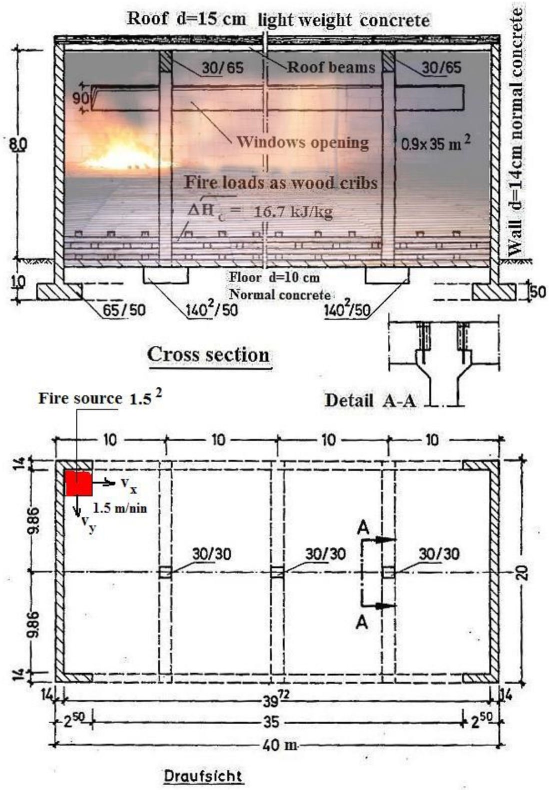


Figure 3.1: Geometry of the investigated reinforced concrete hall building

3.2 Assumptions about the thermodynamic boundary conditions

It was assumed that different fire loads could be present on the hall floor; depending on the usage fire loads of 15, 30 and 60 kg / m2 wood cribs were taken into account for the investigations. The converted on wood cribs fire loads own a calorific value of 4.80 kWh/kg (s. Table 4.1). For the fire source, where the ignition begins, an area of 1.5 x 1.5 m2 in the upper left corner is assumed. The propagation rate of the fire is set on as 2 m/min. It was further assumed that in the fully developed fire phase, 30 kg/m2 wood cribs would burn per hour. These data correspond to the experiences and test results and can be found in the literature (Quintiere., 1976., Roitman., 1972, Hagen, 1987).

For the calculation of the fire temperatures, the window openings A_w/A_F result in a ventilation of 10%. This ventilation condition controls the travelling and the burning rate of the fire loads (Kawagoe, etal., 1963).

4. Fire-room temperatures

4.1 Calculation of the fire room temperatures of the reinforced concrete hall building in case of real fire

The research and determination of the thermodynamic basis of such a theoretical calculation of a damage fire was part of a larger research program in SFB C3-Project dealing with the development of real fires in small and large enclosures. The fire protection investigation of the presented reinforced concrete hall building imposes at first a heat balance calculation of the fire room. The calculation of mean fire room temperatures of a building in a real fire depends on many variable parameters and requires extremely complex and therefore time depending calculations.

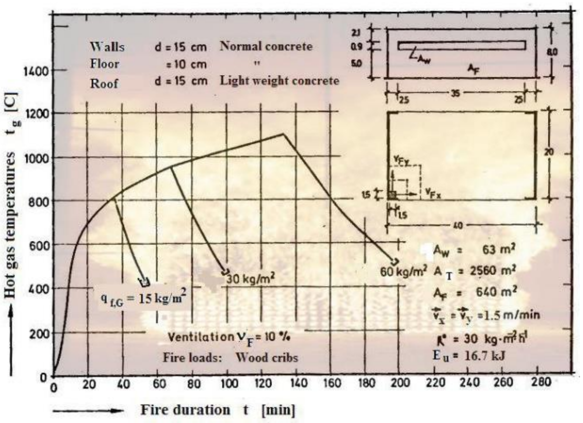


Figure 4.1: Prediction of the fire room temperatures of a real fire

The present heat balance calculation is based on the fact that the fire loads in room release energy when ignited. This released energy is absorbed by the surrounding structural elements and flows in part by convection and radiation through the openings to the outside. However, under certain ventilation conditions, the burning can also take place in such a way that unburned gases flow out through the openings and thus do not contribute to the release of energy.

For the heat balance calculation of such a large hall building 4 basic conditions have been considered:

1- Once it was assumed that the temperature distribution inside this enclosure with windows is homogeneous
2- In addition, the wall surfaces must be so struct-ed that the heat transfer conditions can be described on the surrounding large-area wall components with a one-dimensional approach
3- For the calculation of the ambient temperatures, the heat capacity of the RFCs were neglected.
4- However, the calculation of the temperature fields of the RFCs takes place with a two-dimensional calculation.

In order to be able to check and, if necessary, correct the performance of the established computer program, it was necessary to re-calculate some fire tests carried out in small and large fire compartments. Fig. 4.2 shows such a prediction of a real fire experiment. It deals here a polyethylene fire. In the picture both the measured and the predicted temperatures are illustrated, so that a concrete comparison of measurement and calculation can be possible.

The effectiveness of the established computer program can thus be confirmed by the achieved good agreement between the calculation and experiment. Fig. 4.2 also shows that in the development phase of the fire, an excess of oxygen was present in fire room (α<1). However, in the developing phase of fire oxygen shortage takes place (α>1). In the cooling phase again an excess of oxygen is observed. These results have also been observed in experiments with natural fires in large compartment fire experiments. (Bechtold, R. 1977 and 1976/1977, Hagen, E., 1987)

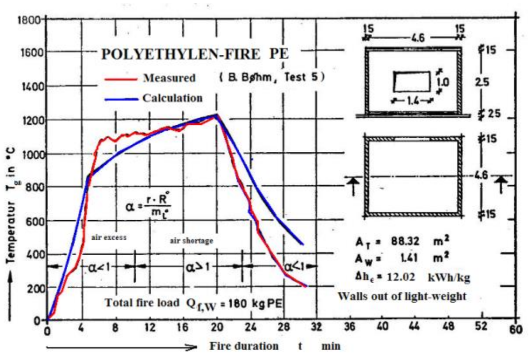


Figure 4.2: Calculated hot gas temperatures in case of a real fire for the hall building (Haksever, A. 1989, s. also PART I, Ref.)

Fig. 4.1 shows the results of a heat balance calculation for the hall enclosure, which was described in the sections 3.1 and 3.2. The calculated hot gas temperatures are shown over the fire duration. They are calculated for fire loads of 15, 30 and 60 kg/m^2 wood cribs. It is being clear from the picture that at a fire load of 15 kg/m2 the maximum temperature reached is about 800 °C and the total fire duration time is about 60 minutes.

The higher fire loads result in both higher temperatures and longer fire durations. For each fire load, the initial development of the fire room temperatures is identical. The fact that the hot gas temperatures in the compartment during the ignition and developing phase of the fire are the same for all fire loads is to be explained by the fact that the same propagation and burning rates as well as ventilation conditions have been taken into account for all fire loads. This assumption can be somewhat crude. In the developing phase of fire temperature zones will occur in the compartment. However for the load bearing calculations of RFCs fully developed fire temperatures gains importance and the propagation rates of fire in the developing phase will play a secondary role (s. PART I section 4.2. Ref.). Calculation shows that the falling branch of the temperatures starts at different times depending on the fire load quantities. The time for the beginning of the cooling phase is determined in the calculations so that a residual energy of 20% of the initial fire loads is reserved for the cooling phase. The heat balance calculations carried out have also shown that sufficient air masses were available in the fire compartment for all fire loads during the total fire duration. It has been shown on the other hand that in the fully developed fire phase, an almost stoichiometric combustion has taken place in the hall building. This phase was reached for all fire loads shortly before the 20th minute of the fire duration, when the fire had already travelled over the whole hall floor.

4.2 Calculation fire load dependent load-bearing capacity of reinforced concrete hall columns.

Fig. 4.3 shows more detailed form two load characteristics for the hall columns. On the right side the results of the load bearing calculations are illustrated according to the action of ISO Fire.

The investigated RFC is exposed from all sides. According to the statically calculations, they have an initial axial force of 455 kN. This results in a fire resistance of about 35 minutes for the acting load in case of standard fire. On the left side of Fig. 4.3, the fire resistance duration of the hall columns is shown in a set of curves, whereby the acting normal force of the RFC has been introduced here as a parameter.

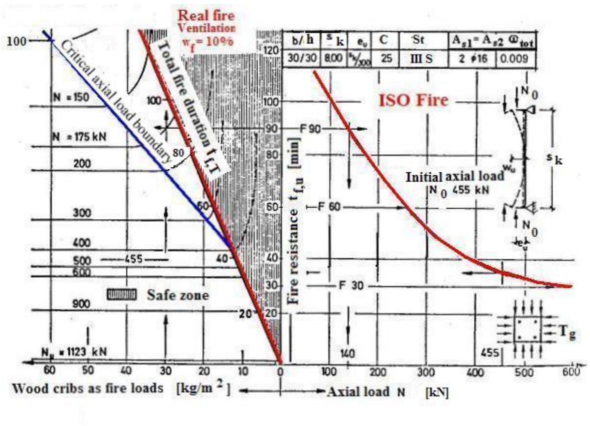


Figure 4.3: Fire resistance of the RFCs of the hall building

The total fire duration of the real fire is shown as a function of the fire loads. This results in, for example, for a fire load of 20 kg /m^2, a total duration of about 70 min. Fig. 4.3 shows two zones separated by the total duration of the fire. The safe region is shown as a hatched zone that means, depending on the fire loads and the axial load of the RFC, some fire and external load combinations may be within this safe range, so that no failure is expected for the hall building columns in case of a real fire. If at a given fire load a higher fire resistance to be achieved, the desired fire resistance must be brought horizontally with the fire load vertically to the section and then an intersection point must be searched on the nearest load curve. This concerning normal force of the load bearing curve indicates which axial load may act on the RFC at most for the given eccentricity. With an existing fire load of 30 kg/m^2, the hall columns then reach a fire resistance time of 90 minutes, if they are relatively low loaded with a normal axial force of 175 kN. For all fire loads that are greater than 11 kg/m^2, a failure results in for the RFC after about 40 (≈ ISO Fire) minutes under the load of 455 kN.

4.3 Fire safety design of the hall construction as industrial building according to the DIN 18230

Fire safety design of the hall building will be shown here as an industrial complex:

It is assumed that on the floor area a 30 kg/m^2 fire load density is present from square timbers as 40x40 mm. The required fire resistance and the class of the structural elements with respect to the

DIN 18 230 results in as follows:

The total surface area amounts to A_T=2560 m^2 (Fig. 3.1)

Window openings: A_w = 63 m^2

The heat energy of the fire load is (s. Table 4.1)

E_u =4.8 kWh/kg

$q_{f,G}$ = 30.20.40/2560~10kg/m^2 = 4,8.10= 48 kWh/m^2

A_w/A_T = 63/2560 = 0,024. From Table 4.2, w = 2,2 and from Table 4.3, γ_{nb} =1,40 for SK_b3. Insulation factor of the building is set as c_i=0,2 for a normal insulation application. The m-factor for square timbers as 40x40 mm is 1,0 according to the table 4.1

Table 4.1 Burning factor m

Nr	Material	Pile Density	m-Factor	E_u
		%	-	kWh/kg
1	Wood and wooden Material			
1.1	Spruce wood			
1.1.1	Boards	50	1.0	4.8
		70	0.8	
1.1.2	Squared timber 40x40 mm^2	50	1.0	4.8
1.1.3	Squared timber 100x100 mm^2	50 90	0.7 0.5	4.8
1.1.4	Squared timber 200x200 mm^2	50 95	0.3 0.2	4.8
1.1.5	Squared Timbers	50 98	0.2 0.2	4.8

The equivalent fire duration results in according to the Eq. 4.1 (s. PART I., "Behavior of reinforced concrete columns of buildings exposed to a real fire, PART I, TUJES"):

$t_e = (q_{f,G})\,.\,c_i.\ m.\ w$ (4.1)

t_e = 48. 0,2. 1,0. 2,2 ~ 20 min.

and necessary fire resistance according Eq. 4.2 is:

$t_f = t_e\,.\,\gamma_{nb}$ =1,4. 20 =28 min. (4.2)

If there is an own fire extinguishing facilities in the hall building the required fire resistance reduces to:

$t_{f,e}$ = 0,6. 28 ~ 17 min. (s. Table 4.4). In this case structural elements need not a special fire resistance and a design with respect to SK_b1 will provide necessary fire safety for the hall building (s. Table 4.5). However –at least- a fire safety class F90 should be designed.

Table 4.2: Heat venting factor w

At / A	<0.05	0.05 <0.10	0.10 <0.15	0.15 <0.20	0.20 <0.25	<0.25
Room opening at one side	3.2	2.0	1.5	1.2	1.0	0.9
Room opening on many sides	2.2	1.5	1.1	0.9	0.7	0.6

Table 4.3: Safety factors for more storey industrial Buildings (DIN 18230)

Area m^2		Safety Factors according to the type of structural member		
		SK_b^3	SK_b^2	SK_b^1
1	1600	1.30	1.00	0.60
2	3000	1.45	1.15	0.80
3	5000	1.60	1.25	0.95
4	7000	1.70	1.35	1.05
4	10000	1.80	1.45	1.15
5	15000	1.90	1.55	1.25
6	20000	2.00	1.65	1.35
7	30000	2.10	1.75	1.45

SK_b^3 – Main structural elements

SK_b^2 – Important structural elements

SK_b^1 – Subordinate structural elements

Table 4.4 Correction factors to take into account the efforts for firefighting according to the DIN18230

Qualified factory fire brigades	γ_{nb} without fire extinguishing system	$\boldsymbol{\gamma_{nb}}$ with fire extinguishing system
Not any	1.0	0.60
2 Parties	0.8	0.50
3 Parties	0.7	0.40
4 Parties	0.6	0.35

Table 4.5: Required fire safety classes of structural elements for industrial buildings according DIN 18230

Required fire resistance for structural element SKb3	Fire safety class	Fire safety design of structural elements		
		SK_b^3	SK_b^2	SK_b^1
≤15	I	-	-	-
>15≤30	II	F 30	F 30	-
>30≤60	III	F 60	F 60	F 30
>60≤90	IV	F 90	F 60	F 60
>90	V	F 120	F 90	F 90

5. Comparison of the fire resistance of RFCs for ISO834 fire and real fire cases

5.1 Definition of the boundary fire loads

The boundary fire load $q_{f,B}$ is a characteristic value for a structure and is given as wood cribs. The RFC shows failure when exposed to the fire with this fire load. However, the fire resistance time is equal to the maximum failure time of the ISO-Fire. The RFC therefore has the same fire resistance duration as in standard fire. Figures 5.1 to 5.7 show the boundary fire loads of various (RFC) under certain boundary conditions. The determination of the boundary fire loads takes place as follows:

On the right side of Figure 5.1, the fire resistance time of a reinforced concrete column is illustrated for different load eccentricities in case of ISO834 fire. This results in almost constant fire resistance duration of the RFCs for different load eccentricities. The left side of the same picture shows the fire resistance of the same RFC according to the exposure of various real fires in the compartment. The load eccentricity is chosen as parameter. The picture shows that the fire resistance time t_f is only slightly affected by different load eccentricities. This means that the boundary fire loads of the examined RFCs can be easily determined, regardless of the load eccentricity. For this purpose, in

Fig. 5.1 the average fire resistance duration in ISO 834 Fire with the fire resistance duration of the same RFC in natural fires must be intersected. This procedure provides on the left side of the picture two intersections. They indicate the upper and lower boundary fire loads. An upper boundary fire load results in because with increasing fire loads a ventilation of 25% will not be sufficient to the complete or stoichiometric combustion and thus to the full development of the fire room temperatures in the enclosure For greater fire loads quantities, therefore, ventilation higher than 25% is needed to ensure at least a stoichiometric combustion.

On the left side of the picture 5.1 three areas can be distinguished. The areas A and C can be described as safe areas with $q_{f,S}$ fire loads, because here higher fire resistance duration than in an ISO834, even no failure is expected. In area B, however, in accordance with ISO 834 an uncertain dimensioning takes place, because in this area the fire resistance duration under the exposure of a real fire is less than would be the case with standard fire exposure. This means that for the area B additional adequate measures must be taken in order to rise the fire resistance duration of the RFC so that it equals at least to an ISO 834 fire.

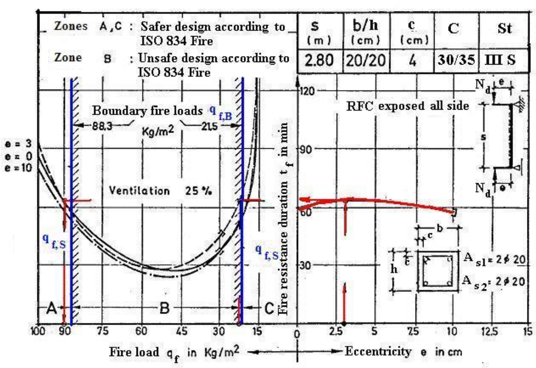


Figure 5.1: Boundary $q_{f,B}$ and safe fire loads $q_{f,S}$ of a RFC at 25% ventilation

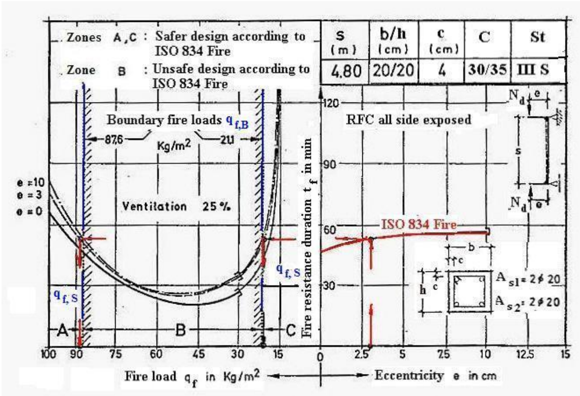


Figure 5.3: Boundary and safe fire loads of a RFC at 25% ventilation

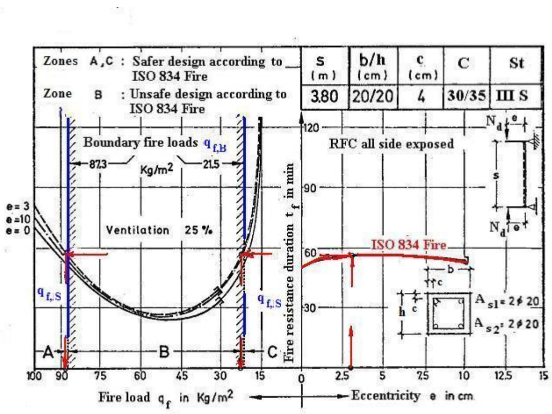


Figure 5.2: Boundary and safe fire loads of a RFC at 25% ventilation

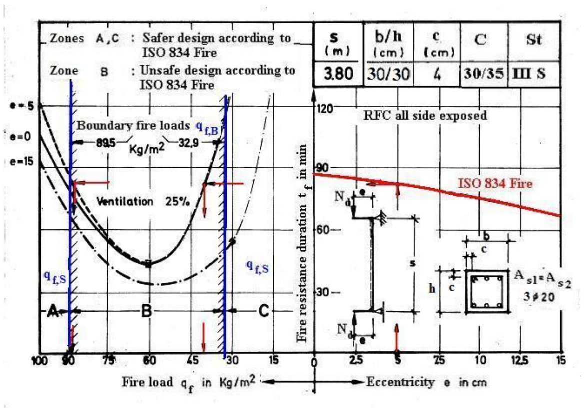


Figure 5.4: Boundary and safe fire loads of a RFC at 25% ventilation

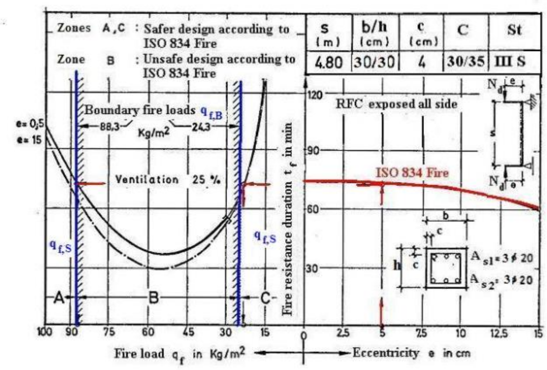


Figure 5.5: Boundary and safe fire loads of a RFC at 25% ventilation

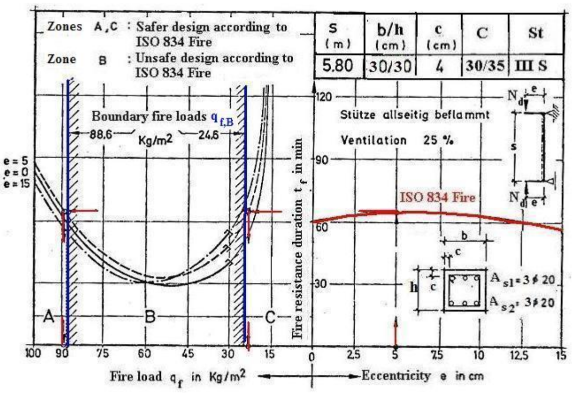


Figure 5.6: Boundary and safe fire loads of a RFC at 25% ventilation

In Figs. 5.1 to 5.6, different reinforced concrete columns were investigated at 25% ventilation for different fire loads and their boundary fire loads were determined. It can then be made a global statement about the boundary fire loads of these RFCs; the upper boundary fire load is therefore 90 kg / m2 and the lower about 20 kg / m2 wood cribs. In order to show the influence of the ventilation on the level of the boundary fire loads, further results of the investigations have been shown in Figs. 5.7 and 5.8. In Figure 5.7, the boundary fire loads were determined for 40% while Fig.5.8 shows the results for 60% ventilation. The increased ventilation shifts the unsafe area to the greater fire loads. For example, at 40% ventilation, the calculations showed an upper boundary fire load of 88 kg/m 2 and a lower load of 45 kg/m^2 as wood cribs. At 60% ventilation, the upper boundary fire load is

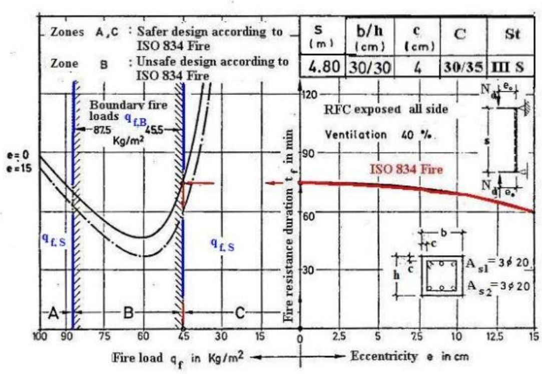


Figure 5.7: Boundary and safe fire loads of a RFC at 40% ventilation

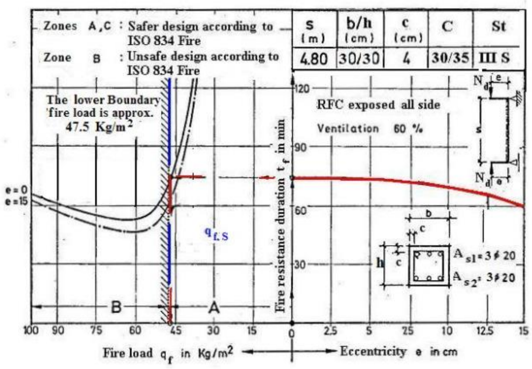


Figure 5.8: Boundary and safe fire loads of a RFC at 60% ventilation

determined almost as 100 kg/m2 and the lower 48 kg /m2 (s. Figs. 5.7 and 5.8).

Table 5.1: (s. §4.3**:** Safe fire loads of the RFCs)

Ventilation %	Safe fire load
25	15
40	30
60	35

5.2 Definition of safe fire load

The safe fire load is defined so that there is no failure at this or beyond these fire loads. In any case fire resistance time is greater than under the exposure of ISO834 fire. Although the reinforced concrete RFC will experience certain cross-sectional degradation, material decomposition and permanent deformation under the action of real fire, there will be no failure due to material or stability failure for a design of RFC according to ISO834 Fire. On the pictures 5.1 to 5.8 such safe fire loads are illudtrsted. For example, Figure 5.1 shows a fire load of 15 kg / m2 wood cribs as a safe fire load. The RFC will stand this fire exposure with a ventilatoin from 25 % without any failure From these investigations, safe areas for real fires can be obtained for the RFCs. This means that the RFCs under the specified boundary conditions in real fires will have a longer fire resistance duration than maximum fire resistance which is present according ISO834 fire action. Generally the following fire loads in A and B regions can be given as safe fire loads for different ventilation conditions (s. Table 5.1):

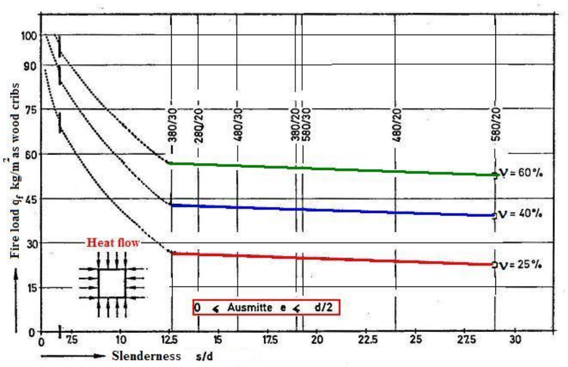


Figure 5.9: Safe fire zones of a RFC in real fire exposure

Figure 5.9 shows such safe areas for various RFCs over the slenderness. These areas have different band widths for different ventilation conditions. Whether a RFC falls into a safe area, can be determined by bringing the slenderness to the existing fire load to intersection. If this point of intersection is below the specified ventilation, the RFC is in the safe area and has at least the fire resistance duration determined according to the ISO834 fire exposure Figure 5.9 shows that the safe area also increases with greater ventilation. In critical ventilation conditions, such as 25% ventilation, there is a reduced safe area for the RFC. The safe areas were determined by evaluating the fire resistance duration of numerous calculated RFCs under different ventilation conditions. The Fig. 5.9 is limited for load eccentrities maximum to e / d = 0.5.

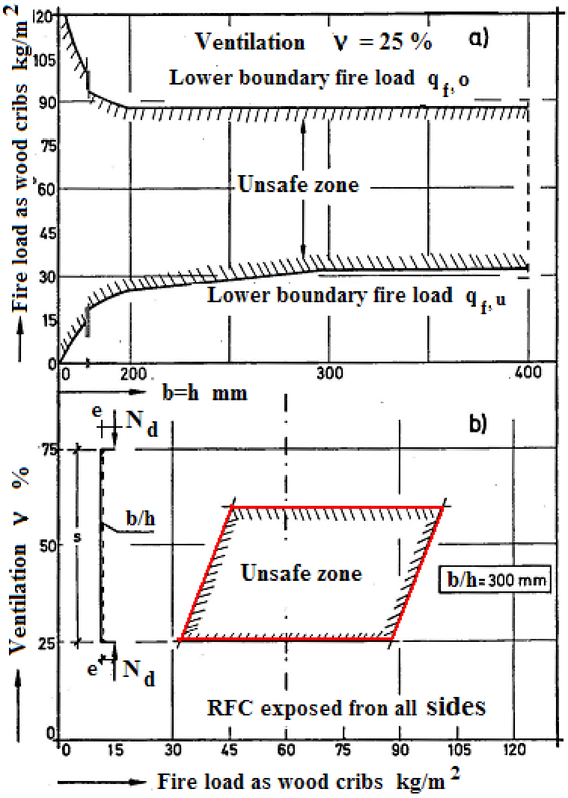
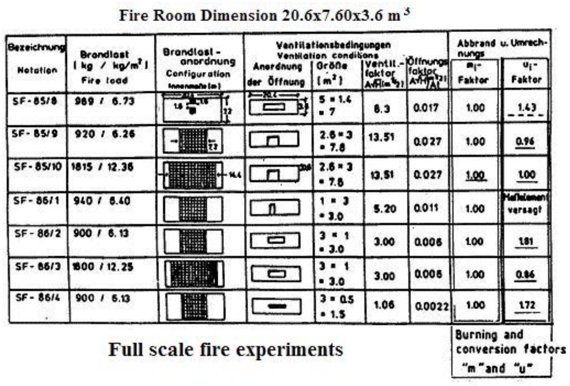


Figure 5.10: a) Distribution of the boundary fire loads bound to the concrete cross section dimensions

b) Safe and unsafe zones of a concrete cross section in real fire case

By means of the Fig. 5.9, the stability of RFCs can be assessed also in industrial buildings under the different ventilation conditions for a real fire case. In Fig. 5.10, several RFCs with a square cross section were further investigated with regard to the boundary fire loads. Fig. 5.10 shows the upper and lower boundary fire loads of the RFCs over the cross-section dimensions. In accordance with the RFC slenderness for square cross sections of 20 to 40 cm, this results in an upper boundary fire load of 90 kg/m2 and a lower boundary fire load of 30 kg/m2 as wood cribs. As decreasing cross-sectional widths, the lower boundary fire loads are reduced, while the upper boundary fire loads increase. This observation can be explained by the fact that at low ventilation and accordingly high fire loads, the mass of air needed for combustion are not available, so that due to the lack of oxygen, the fire room temperatures can not fully develop and for smaller square cross-sections (20-30 cm), the upper fire loads must be very high in order to cause a lack of oxygen and lower temperaturesin the fire room.



Fire Room Dimension 20.6x7.60x3.6 m^3

Bezeichnung Notation	Brandlast [kg / kg/m²] Fire load	Brandlast-anordnung Configuration Innenmaße [m]	Ventilationsbedingungen Ventilation conditions: Anordnung der Öffnung	Größe [m²]	Ventil.-faktor	Öffnungs-faktor	Abbrand u. Umrechnungs: m_i-Faktor	u_i-Faktor
SF-85/8	989 / 6.73			5 · 1.4 = 7	8.3	0.017	1.00	1.43
SF-85/9	920 / 6.26			2.6 · 3 = 7.8	13.51	0.027	1.00	0.96
SF-85/10	1815 / 12.36			2.6 · 3 = 7.8	13.51	0.027	1.00	1.00
SF-86/1	940 / 6.40			1 · 3 = 3.0	5.20	0.011	1.00	Meßelement versagt
SF-86/2	900 / 6.13			3 · 1 = 3.0	3.00	0.006	1.00	1.81
SF-86/3	1800 / 12.25			3 · 1 = 3.0	3.00	0.006	1.00	0.86
SF-86/4	900 / 6.13			3 · 0.5 = 1.5	1.06	0.0022	1.00	1.72

Full scale fire experiments

Burning and conversion factors "m" and "u"

Figure 5.11: Parameters of the fire loads applied in full scale Finland-Experiments

6. Summary and conclusions

This paper deals with the fire behavior of monolithically cast reinforced concrete structures. Starting point for the study is a reinforced concrete building, which is a typical contemporary industrial building. The former studies showed that boundary conditions for columns and beams of an entire structural system can be simulated on a single structural. A real fire development can be modelled by the standard fire condition.

Fire design is based on the verification of adequate structural safety in case of a fully developed compartment fire. In practical application adequate structural safety may be assumed if the needed function of structure or structural member in regulations is maintained during the relevant part of the fire exposure. It applies to the structures and structural members which surround the fire compartment, e.g. external columns and beams. By this the design situation considered may be a fire affecting the structure as a whole or only a part of it. Due to this fact the GFPA-guideline "Ingenieurmethoden des Brandschutzes" (Engineering Methods of fire protection) has been developed in the recent years, which describes and classifies the available possibilities, approaches and models as well as provides suitable support for their application. Nowadays it is unavoidable to continuously improve and extend the available possibilities of numerical fire simulations also in the future to satisfy the rising requirements as sufficiently as possible. Beyond that, the developed model has made a valuable contribution in other fields, where extensions and improvements are still necessary in the future (Dobbernack, R., 1987 and Haksever, A., 1988).

In summary, in this research work the assessment method 3 is generally applied to all types of fires where sufficient practical experience concerning the risks at fires are available (s. section, PART I, 3.2 Fire protection design principles, Fig. 3.5., s. also Hagen, SFB 148 Subproject C3, -Finland-Versuche-"Finland experiments," 1987). Fig. 5.11 shows the full scale experiments carried out in an enclosure with wood cribs in VTT-Helsinki (s. Ref.).

Also an important step was made for the purpose of the fire protection design of structures. The assessment concerns only aspects relating to the performance of structural elements. Special measures for ensuring reparability and reservicability and protection of people are not dealt in this paper. In this concern the research work deals with the definition and the determination of boundary fire loads $q_{f,B}$ and safe fire loads $q_{f,C}$ of RFCs:

1- Boundary fire loads of a RFC is so defined that they result in the same maximum fire resistance during the acting of an ISO 834 fire. Boundary fire loads show two limit values as lower $q_{f,u}$ and $q_{f,o}$ an upper fire load for the RFC. These fire loads result in the same fire resistance of an ISO 834 fire. Within these fire loads the fire resistance of RFCs is lower than the ISO 834 fire case and indicates and unsafe area (s. Figs. 5.1-5.8). Any design with respect to the

ISO 834 fire will cause a less fire resistance than the case of the standard fire.

2- Beyond the boundary fire loads, the regions ($q_{f,S} < q_{f,B}$ and $q_{f,S} > q_{f,B}$) include the safe fire loads $q_{f,S}$. In those areas the fire resistance of RFCs in case of real fire is in any

3- Beside that the design of RFCs with respect to the ISO 834 regulations is presented (s. Section 4.2). In an example it is demonstrated that no additional preventive measures for RFCs of the industrial building may be necessary with respect to the ISO 834, however it is recommended that a F90 design regulations should be chosen.

4- Kodur, etal presents in their article (2018), that an insulated fiber-reinforced polymer strengthened (FRP) reinforced concrete slab (RC) can achieve fire resistance comparable to that of a conventional un-strengthened RC slab, and can withstand higher load levels during fire exposure. Provision of fire insulation on FRP strengthened RC slab can enhance fire resistance by as much as 60-90 minutes, in most practical situations.

5- Hawileh, R. A. Draws also in their article (2011) the following main conclusions based on the results of the finite element inves-tigations:

a-With respect to the fire performance of CFRP-strengthened insulated T-beams exposed to top fire, the temperature in the FRP was relatively low during the fire exposure and therefore its structural integrity was maintained.

b-As a result of heating the top surface of the beam, the downward deflection was decreased. On the other hand, an increase in the downward deflection will occur if the beam is heated from the soffit of the beam. Heating the top surface of the beam seems with a behavior similar to beam prestressing.

In the above given articles it was emphasized *interestingly*, that since the failure state shows different modes according to the amount of fire load in natural fires, the different edge conditions (top or bottom heating, CFRP or FRP use, insulation use ..) will show different modes and this situation will be corralated to the effect of fire loads.

Beyond that, the developed Structural model makes also a valuable contribution in other fields (s. Haksever, TUJES Issues). Finally, the present numerical fire simulations were expanded with the developed model for enclosure fires and verified with large scale fire experiments (s. C3-experiments in Finland) where the improvements are still necessary in particular in upgrading travelling fires and pyrolysis models.

Conflict of Interest:

The authors declared that there is no a potential or existing conflict of interest between their scientific work and their personal situation.

Authorship:

All authors certify that they have participated sufficiently in the work to take public responsibility for the content, including participation in the concept, design, analysis, writing, or revision of the manuscript. Furthermore, each author certifies that this material or similar material has not been and will not be submitted to or published in any other publication.

Acknowledgement and notations:

See “Behavior of reinforced concrete columns of buildings exposed to a real fire, Part I, TUJES, 20(1): 1-21”

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See more in "Behavior of reinforced concrete columns of buildings exposed to a real fire, Part I, TUJES, 20(1): 1-21"