

## BEHAVIOR OF REINFORCED CONCRETE COLUMNS OF BUILDINGS EXPOSED TO A REAL FIRE, PART I

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### Highlights

- Fire Development and structural failure in real fires and design methods
- Fire loads density and ventilation factor. Definition and determination of the equivalent fire duration
- Behavior of reinforced concrete columns RFC under real fire action. Determination of the fire resistance time.

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### Abstract

Structural fire design is concerned mainly with the avoidance of structural failure or damage so that the fires are to be controlled at an initial stage. The present paper is aimed at investigating the structural behavior in bending of reinforced concrete columns RFC in an enclosure exposed to real ventilation controlled fires, i.e. fires with a heating and a cooling phase and presents methods for determining the fire resistance of RFCs exposed to a real fire. The results show that the prolonged heating of the reinforcing steel may cause to failure in the cooling phase of the fire. Beside in a separate research work boundary fire loads of reinforced concrete columns for an enclosure are determined beyond which no failure can be expected or fire resistance is much greater than ISO834 in case of a real fire (s. Ref. Part II).

## YAPILARDA BETONARME KOLONLARIN TABİİ YANGINLARDA DAVRANIŞ DURUMLARI, BÖLÜM I

### Makale Bilgileri

### Öz

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#### Anahtar Kelimeler:

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kolonları;  
yangın dayanımı;  
tabii yangınlar;  
sınır yangın yükleri

Yapılar üzerine olan yangın tasarımı esas olarak, yapı elemanın göçmesini ve zarar görmesini önleyerek yapının başlangıç fazında denetim altında tutulmasını sağlamaktır. Sunulan bu çalışmada, betonarme kolonların ventilasyon denetimli tabii bir yangın etkimesi durumunda, örneğin ısınma ve soğuma fazlarını içeren bir yangın gelişimi için, II. Mertebe teorisini kullanarak, yangın davranışlarını ve dayanımlarını belirlemek olacaktır. Araştırma sonuçları beton donatılarında maksimum sıcaklığın soğuma fazının başlamasından sonra, (hatta saatler sonra) meydana gelebileceğini ve bu durumun da yapı elemanında gecikmiş bir göçmeye neden olabileceğini göstermiştir. Donatının uzun süreli ısınması yangının soğuma fazında yapı elemanın göçmesini sağlamaktadır. Bu nedenle kapalı bir hacimde betonarme kolonların sınır yangın yükleri Bölüm II de saptanmış olup bu sınırın dışında kalan yangınlarda, yapı elemanlarının yangın direnci ISO834 yangınına göre daha büyük olmakta ve hatta bir göçme riski içermemektedir.

## 1. Introduction

The fire protection design and assessment of structural members is currently carried out on the basis of standard fire tests with respect to ISO834 tests. In this case, the structure to be examined is subjected to the fire curve of ISO834 and tested for their structural behavior and their fire resistance duration. With this method, comparability and any reproduction of the experiments to investigate the fire behavior different structural members is secured (Knublauch, 1972).

Structural fire design is concerned mainly with the prevention of fire spread through vertical and horizontal partitions and the avoidance or limitation of structural failure. The basic unit for structural fire design is the fire compartment or fire zone. The level of structural fire safety to be provided by design should thus be determined

1. The risks involved in the case of a severe fire considered as an accidental situation
2. The risk-reducing effect of structural measures

It follows that, for particular types of buildings and occupancies, structural design requirements may be dispensable, because the associated risks are sufficiently small. Certain requirements may also be taken into account, 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 (e. g. by reducing the fire loads).

Generally, building structures should be designed, constructed and maintained so that they display an acceptable performance and fulfil specified functions in the case of fire.

In SFB, 1971-1986 (see references) of the Technical University of Braunschweig (Ger.) fundamental research were carried out in the eighties, to investigate the deformation behavior of reinforced concrete structures both experimentally and theoretically under fire exposure. The experiments are carried out according to

the ISO834 Fire as well as to a real fire. For the simulation of a real fire wood cribs as fire loads with standard density are used as shown in **Fig. 1.1**. An artificial inflow ventilation condition is applied, so that excess air in furnace was present. The test showed that the fire exposure caused a failure during the cooling phase of the real fire (Haksever, A., 1986).



**Figure 1.1:** Concrete column in test furnace of SFB with wood cribs as fire loads

The main scope of this article is to achieve solutions with greater economy and more uniform safety, which indicates that there is a strong need to move towards analytical structural fire engineering design methods. Generally these methods include two main steps:

- 1- A calculation of the temperature distribution within the fire exposed load bearing member during the heating process
- 2- A transformation of these temperature distributions to the variation of the load bearing capacity as a function of time

The first step requires the solutions of the *heat balance* equilibrium equation for given geometries and boundary conditions. For practical applications, numerical methods have to be used to solve this equation.

The second step requires valid models for calculating the mechanical behavior and the load bearing capacity of fire exposed structural members or assemblies. This implies that the structural material properties in a

temperature range of fire must be known.

In this research article the first step is used to determine the temperature development for certain geometries of enclosures in which the concrete column is exposed to the high temperatures.

In the second step the structural response and fire resistance time of RFC is determined analytically. In the development of the fire no extinguishment measures are taken into account so the total burning of the fire loads are permitted in the enclosure that means, the fire compartment should adequately confine the fire to a limited area.

## 2. Theoretical background/experimental

During the last decades, considerable progress has been made in developing analytical and computation methods for fire exposed load bearing members and structural systems. Consequently, an analytical design can be completed today for most cases where steel structures are fire exposed. Validated material models for the mechanical behavior of concrete under high temperature conditions derived during the recent years have significantly enlarged the area of application of analytical design (Roitman. 1972., ECCS, 1983. CEB 1983.).

Concerning literature, it is rare to find fire test results on the structural columns under the exposure of a real fire, especially for RFC. These tests are conducted mainly for isolated steel columns and to determine the failure time  $t_f$  some applicable formulas are derived.

Christopher D.Eamon, Elin Jensen conducted a reliability on reinforced concrete columns subjected to fire load. From an evaluation of load frequency of occurrence, load random variables are taken to be dead load, sustained live load, and fire temperature. A rational interaction model based on the Rankine approach is used to estimate column capacity as a function of fire exposure time. Reliability was computed from 0 to 4 h of fire exposure using Monte Carlo simulation. It was found that reliability decreased

nonlinearly as a function of time, while the most significant parameters were fire type, load ratio, eccentricity, and reinforcement ratio.

Jingsi Huo, Jianguang Zhang, Zhiwei Wang, Yan Xiao (2013) present in their paper an experimental investigation of the effects of preload and cooling phase on the residual strength, stiffness and ductility of reinforced concrete stub columns which were heated and cooled down to room temperature under sustained axial load. Reinforced concrete stub columns were axially loaded and heated to designed temperatures in a specially built electrical furnace. After the specimens cooled down to ambient temperature with the axial loads kept constant, the stub columns were loaded to failure. The test results showed that the mechanical behaviour of the fire-damaged reinforced concrete stub columns with preload was remarkably different from those without preload. Based on the test results, it is recommended that the effects of sustained axial loads during the fire and cooling phase should be taken into consideration in assessing the fire-damaged RFC.

Faris Ali, Ali Nadjai, Gordon Silcock, AbidAbu-Tair (2004) present the results of a major research executed on 99 high and normal strength concrete members. The parametric study investigated the effect of restraint degree, loading level and heating rates on the performance of concrete columns subjected to elevated temperatures with a special attention directed to explosive spalling. The paper illustrates a method of preventing explosive spalling using polypropylene fibres in the concrete.

Kamila Horová, Tomáš Jána, Yong C.Wang, František Wald (2015) make fire test and numerical simulation results of temperature distributions in reverse channel connections to concrete-filled tubular columns during standard and real fire tests. The experiments included a furnace fire test with the composite frame subjected to increasing fire temperature according to the ISO834 standard time-temperature curve and two real fire tests

in a full-scale structure including a cooling phase. From the numerical simulation results, it has been concluded that radiation to the inner surfaces of the reverse channel and the adjacent part of the column tube is only from the gas volume bounded by these surfaces.

Ulf Wickström (1981) gives a method suitable for design purposes which allows the approximate post-flashover compartment fire temperature to be plotted versus time in one curve, the general real fire curve; time is then modified or scaled to take into consideration ventilation conditions. In the analysis the common assumptions of constant and ventilation controlled combustion, uniform temperature. The general real fire curve is given an analytical expression, which is then used to calculate temperature in fire exposed insulated columns by a simple integration procedure. The results are plotted in handy diagrams, and temperatures obtained in columns exposed to real fires and standard fires according to ISO 834 are compared.

Jean-Marc Franssen, Dan Pintea, Jean-Claude Dotreppe (2007) present in their paper methodologies that are used for analysing the fire behaviour of a structure that is subjected to a uniform thermal situation cannot be applied when the fire is localised. The concept of “zoning” can be applied by this the structure is divided into several zones in which the situation is approximated as uniform. After a discussion of the concept and the particularities dictated by the continuous thermal environment, the methodology is utilised and explained for a localised fire.

Samantha Foster, Magdaléna Chladná, Christina Hsieh, Ian Burgess, Roger Plank (2007) present in their paper a numerical investigation of the thermal and structural results from a compartment fire test, conducted in January 2003 on the full-scale multi-storey composite building constructed at Cardington, United Kingdom, in 1994 for an original series of six tests during 1995–1996. The fire compartment's overall dimensions were

11 m×7 m. The compartment was subjected to a real fire of fire load 40 kg/m<sup>2</sup> of timber. Numerical modelling studies have been performed using the numerical software FPRCBC to analyse temperature distributions in slabs, manual Eurocode 3 Part 1.2 calculations for beam temperatures, and VULCAN to model the structural response to thermal and mechanical loading. The comparison between the modelling of basic cases and the test results showed very good correlation, indicating that such modelling is capable of being used to give a realistic picture of the structural behaviour of composite flooring systems.

Tamás Balogh László, Gergely Vigh (2016) indicate in their paper, that a proper estimation of reliability in fire design situation is complicated since there is no comprehensive methodology defined for it; and the commonly used methods apply great simplifications. Further research work is therefore needed to refine rules and define targeted safety levels required by fire codes. Based on First Order Reliability Method the authors have found that for low and moderate consequence classes the calculated reliability indices are in better agreement with the recommendations of ISO 2394 standard and Joint Committee on Structural Safety than with the values recommended in EN 1990:2002 standard.

The authors carry out a reliability analysis conducted on reinforced concrete columns subjected to fire load. From an evaluation of load frequency of occurrence, load random variables are taken to be dead load, sustained live load, and fire temperature. Calculations showed that the fire resistance time is developed for axial capacity, with random variables taken as steel yield strength, concrete compressive strength, placement of reinforcement, and section width and height. It was found that reliability decreased nonlinearly as a function of time, while the most significant parameters were fire type, load ratio, eccentricity, and reinforcement ratio.

Xing Zhang, Sashi Kunnath, Yan Xiao (2017). This study investigates the cyclic behavior of reinforced concrete columns that have been exposed to a major fire following a minor to moderate earthquake. Eight specimens with varying column bar diameters and varying confinement were tested under two primary scenarios. Columns that were subject to earthquake-induced fire experienced significant cover spalling. Compared to columns that did not experience fire, the fire-damaged columns experienced a loss of strength and stiffness. It was also observed that columns with smaller diameter longitudinal reinforcement and better confined columns exhibited better seismic performance. In this paper, only results from two typical column tests are presented.

Patrick Bamonte, Nataša Kalaba, Roberto Felicetti (2018) present in their paper that the behavior of simply supported prestressed members exposed to real fires is studied by means of a sectional approach. They observed that the prolonged heating of the prestressing steel may lead to failure in the cooling phase. However the constitutive behavior of concrete plays a minor role in determining the failure of members subjected to bending.

Pettersson, Ove., Ödeen, Kai (1978). The Authors present a design book for fire protection which has been issued by the government's office. The design rules aim at facilitating a practical application of the methods for calculating performed technical dimensional design of building members with bearing or separation function, as accepted in accordance with the provisions of Swedish Building Regulations 1975, section 37:33. The level of knowledge for a computational fire engineering dimension of bearing or separating building constructions currently varies considerably with the type of construction material. This reflects really in author's presented design basis, which is completely for reinforcing concrete beams and wooden beams. The written directives in related parts have previously been

compiled in the State Plan's Comments to the Swedish Building Model 1976: 1, Fire Technical Dimensioning.

The above given literature review showed that for real fires in compartments a systematic research taking into account the decisive parameters still fails. Some punctual research papers insist only that a failure case for reinforced concrete columns can be awaited also in cooling phase of the fire. The resolution of the technical problems of protection against the fire is the ability to estimate the evolution of the fire in the compartments and the influence of the fire on the building members. Until, present, it is achieved a certain level of safety in the event of a fire for structural building components using the standard curve of the ISO834 fire. However, the evolution of the fire in practice differs from the ISO-Fire so that the interest in the definition of this phenomenon of real fire is of great importance.

The research works carried out in SFB-Subjekt B3 showed that the cooling phase of a real fire significantly influences the material behavior of the reinforced concrete. The computational studies on reinforced concrete structures for a real fire are thus considerably more complicated than the calculations according to ISO-Fire condition. The computational judgement and prediction of the deformation behavior of reinforced concrete members in the cooling zone of a real fire is therefore more calculation intensive, because in this phase, the local stress-strain relationships happen in cross-section on the one hand elastic and on the other inelastic.

Furthermore, the thermodynamic boundary conditions determine the behavior of the building materials due to the heat humidity transport in the structural components significantly. The cooling of the building materials after a previous heating causes, for example an irreversible development of the temperature diffusivity for the normal concrete (Kordina, 1975). Thus, for example, the

thermal conductivity of the concrete on the one hand depends on temperature, but on the other hand also depends on whether a first heating or a subsequent cooling is present. In the presented literature review in the thermal behavior of the concrete has not been discussed. This research work aims at first the investigation structural response of RFC under real fire exposure and presenting necessary design chart taking into account the parameters as ventilation, fire spread, compartment size and fire loads which might be useful for fire protection engineers.

In this context the research work in (Haksever, 1986) presents both the first results of the experiments carried out on the reinforced concrete columns and the results of the theoretical investigations on the estimation of the behavior of the reinforced concrete columns examined in **SFB**. The main topics included in this work are the distribution of wooden cribs, the deformations of building reinforced concrete column during the fire and a heat balance calculation in relation to the weight loss fire wood cribs. In section 4. Test results are discussed in details.

### 3. Development of the real fires and fire risks

#### 3.1. Fire development bound to fire loads

Fire is a phenomenon that has always strongly influenced humanity from its origins to the present day. At some point in the distant past, our ancestors learned to use it, but also to fear it. As long as the fire was tamed, it was considered as a useful energy and heat donor but gets out of control, "it became a force of nature" that often became a disaster, man never forgot to fear the fire. As long as people lived alone or in small groups, the risk of a major fire was relatively low.

When people founded cities and lived close together in houses that used to be made of combustible

building materials, the risk of fire increased considerably. Large fires, firestorms, and major fires were possible. In major fires, districts or entire cities were destroyed. As examples, only the great fires of London in 1666 and Istanbul in 1633 are mentioned here (Kordina, K. et al 1981, s Fig. 3.1).



Figure 3.1: The great London-Fire from 1666

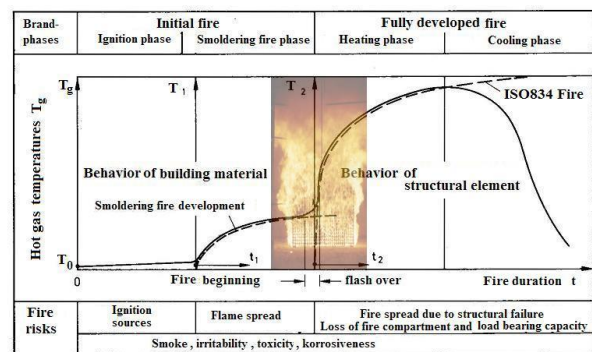
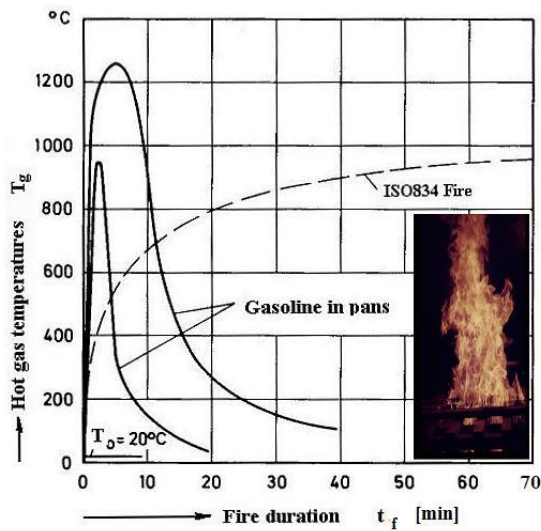


Figure 3.2: Development of a real fire in different phases, (Kordina, et al., 1981, in German)

The development of a fire is in terms of temperature level and time in general according to the Fig. 3.2 in two or four temperature-time periods:

The first two sections are characterized by the beginning of fire, the last two by the fully developed fire. In the second period a smoldering fire follows; the flames can spread in accordance with local conditions so that the affected room is heated more and more by the heat released. It is observed that this fact takes place until all flammable substances present in the room are ignited. If this critical time or area is achieved, it is called "flash over": The smoldering fire goes into a fully developed fire. The duration of the fully developed fire

depends on the amount of flammable substances, oxygen supply into the fire compartment and local conditions.



**Figure 3.3:** Temperatur development in gasoline fires

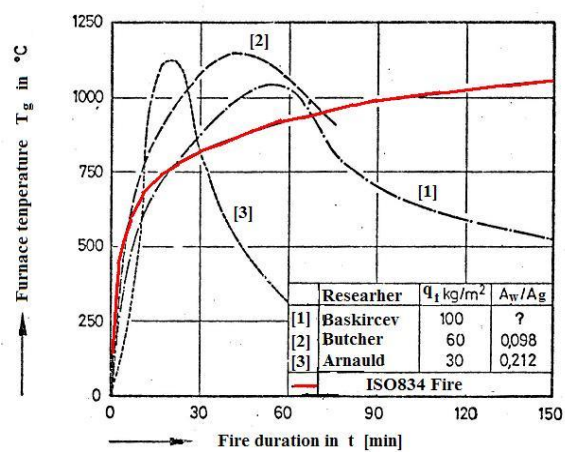
For a large amount of easily inflammable materials, the ignition time may be very short. Almost seam-lessly, a fully developed fire can occur with steeply rising temperatures. The temperatures in such fires usually develop similar to the temperatures in gasoline or cell fires (Hinrichs, 1983., Butcher etal, 1967., Hopfman, 1963.), as illustrated in Fig. 3.3 and compared with the standard ISO834 temperature curve.

While ignition sources and flammability would be the main parameters in the first phase of fires, the second phase of fire is characterized by flame propagation and heat generation (s. Fig. 3.2). In parallel, the risks of smoke, irritation, toxicity and, possibly, corrosiv-eness, which also play a role in the following phases of fire, are generally to be mentioned. At the begin-ning of fire, the fire load (s. Eq. 4.1) and the building material behavior are the main parameters, which influence the fire development. In contrary, the last two phases of fire, the structural behavior of bearing members is generally decisive. The fire propagation can be influenced here by structural failure.

### 3.2 Fire protection design principles

The fire protection design and assessment of structural components are currently made on the basis of standardized fire tests with respect to the ISO834 Fire to which the components in question will be exposed. This experience-based system of requirements and test regulations has so far not led to any particular risks in constructional facilities of normal use. This fact is all the more astonishing since, in particular, the conditions and requirements of standard heating in furnace according to ISO834 are relatively arbitrary. Because of the infinite variety of fire developments, a single, and not even the most unfavorable, fire case is defined as the decisive fire structural design.

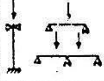
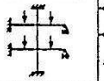

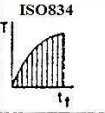
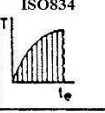
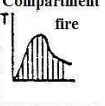
In Fig. 3.4., some temperature-time curves, which were observed under certain conditions in real fire experiments, are compared with standard fire curve. It can be seen that the standard fire does not reflect by far the worst case of fire. Depending on the fire load and the present ventilation conditions conditions a fire development are possible, which lie substantially above the standard fire.



**Figure 3.4:** Temperature development in different real fires and ISO834 standard fire curve

On the other hand, it is also conceivable that in rooms with very low fire loads the developing temperatures in case of a real fire standard fire temperatures curve do not reach and thus they remain far below (Kawagoe, 1963). There are therefore many efforts underway for the supplement the test method according to ISO834-Fire by incorporating the actual expected fire exposures

in practice. In addition to the design on the basis of a standard fire test, the aim is therefore strived to have design methods which meet the thermal exposure conditions to be expected in a real fire.

Structural model		$S_1$	$S_2$	$S_3$
		Members	Sub-assembly	Structures
Heat exposure model				
	$H_1$ 	Test or calculation	Test or calculation (rare)	not recommended
	$H_2$ 	Test or calculation	calculation or test (rare)	not recommended
$H_3$ 	calculation	calculation	calculation mainly for re-research purposes	

**Figure. 3.5:** Heat and structural response models (Workshop CIB W14, 1989)

Structural fire design comprises:

1. Assessment of the heat exposures and structural behavior
2. Structural detailing i. e. an adequate choice of the structural systems, the geometry of possible fire compartment and its various components
3. Material detailing, i.e. an adequate choice of materials with specified thermal and mechanical properties.
4. Design verification should consider the frequency of fires, their expected severity, the nature of the thermal and structural behavior and the actions relevant in fire exposure. The current possibilities are given in Fig. 3.5.

The assessment methods illustrated in Fig. 3.5. are fire exposure models defined on the picture as  $H_1 - H_3$  and can be classified as follows:

1. Assessment method 1 takes into account the ISO 834 fire exposure. The design criterion is that the fire resistance time is determined either by fire tests or analytically.
2. Assessment method 2 bases on the standard fire assessment 1. Method 2 is considered as a simp-

lification of method 3, but in contrast to method 3 does not allow to deal with problems in the real exposure from ISO834. The design criterion is that the fire resistance time will be determined like as fire case.

3. Assessment method 3 takes into account the analytical design directly on the basis of compartment fire exposure. In this procedure nonstandard compartment fire exposure is related to the standard fire by an “equivalent fire duration”, which is a function of the fire load, the geometry of the fire compartment, the ventilation conditions and the thermal properties of the enclosure (Schneider, 1973, 1976). In section 4 this expression will be explained in more detail.

The assessment method 1 is essentially a classification system than functionally based design method. According this method only the structural members are identified with the heat exposure – structural model combination  $H_1 - S_1$  and in some cases  $H_1 - S_2$ .

### 3.3 Standard fire exposure and fire resistance time

The standard fire exposure is defined by the temperature-time-curve according to ISO834. For particular applications some countries use other temperature-time-curves (Magnusson, Nr65).

Fire resistance time of a structural member may be determined:

1. Experimentally according ISO 834 or its national variant
2. Analytically for criterion referring to the load bearing capacity (section 5)
3. By interpolation and analogy from experimental or analytical results
4. By referring to catalogues

Fire resistance time  $t_{f,u}$  of a structural member may be expressed

1. Directly as a fire resistance time  $t_{f,u}$  in minutes or



2. Indirectly in terms of fire resistance classes as F30, F60, F90...which represent the min. fire resistance time (or R30, R60...).

Fire resistance time can be determined

1. As a function of mechanical loading and material properties
2. For a specified design load.

Fire resistance time is achieved when the load bearing capacity  $R_f$  will be less than the acting external stress condition according Eq. 3.1.:

$$R_f \leq P_{e,f} \quad (3.1)$$

Generally fire design, the fire resistance time especially is designed for the design load corresponding normal design loads. More consistently the fire resistance time is determined for an *accidental fire load* as combination of different stressings shown in Eq. 3.2.

$$P_{f,a} = (G_f + \sum_1^n \Psi_i \cdot Q_{f,i} + Q_{f,T}) \quad (3.2)$$

It should be noted that applying an appropriate accidental load combination will contribute to a more uniform level of structural safety in fire exposure. In following section the H<sub>3</sub>-S<sub>1</sub> design methods and decision steps will be explained.

#### 4. H<sub>3</sub>-S<sub>1</sub> Design methods for structural members under real fire exposure

##### 4.1 Fire exposure

The thermal exposure on the structure or structural member during a fully developed compartment fire is determined by the energy and mass balance equations which take into account the characteristics of the fire load, the ventilation of the fire compartment and the thermal properties of the enclosing fire compartment. The thermal exposure can be specified by the time curve of hot gas temperatures or heat flux to the structure or structural member. Controlling parameters are the fire load density, the opening factor and the

thermal properties of the fire compartment (Quintiere, 1976).

##### 4.2 Heat balance calculation

It is generally sufficient to assume a fully developed ventilation controlled fire in a compartment with a uniform temperature distribution (Haksever, 1989). However, assuming a uniform temperature distribution may not be adequate in the case of extremely concentrated fire loads and may be necessary to consider the effect of local fires. The validity of these assumptions may also be questioned for large fire compartments and compartments with extreme ventilation conditions. An assessment of the fire compartment according to a rather crude model may involve that a local fire will not cause progressive collapse of the structural system. For a practical application, in section 5, the determination of the limit fire loads of concrete columns a uniform temperature distribution in the compartment has been taken into account.

##### 4.3 Fire load density and ventilation factor

The fire load density is derived from

$$q_f = \frac{1}{A_f} \sum_1^n M_i \cdot E_{ui} \cdot (m_i) \quad (4.1)$$

$$q_t = \frac{1}{A_t} \sum_1^n M_i \cdot E_{ui} \cdot (m_i) \quad (4.2)$$

Insofar as fire loads are concerned, which are similar to wood cribs, the temperature development in a room can be estimated or approximated in advance, taking into account Eq. (4.1) and (4.2).

The ventilation factor represents the influence of the air flow condition into the fire room in absence of wind and mechanical ventilation as given in Eq.4.3 (CIB W14, 1989):

$$v = A_w \sqrt{h_w} / A_t \quad (4.3)$$

where  $A_w$  is total area of door and window openings,  $h_w$  is mean value of the height of window and door openings and  $A_t$  is total interior area of the surfaces bounding the fire compartment including all openings (Pettersson., et al., 1978).

Ventilation factor is defined in some publications and research article according to Eq. (4.4):

$$v_{f,i} = A_{w,f,i} / W_{f,i} \quad (4.4)$$

where  $v_{f,i}$  is the front ventilation as [%],  $A_{w,f}$  window opening in front area and  $W_{f,i}$  total wall area of the window opening (Arnault., et al, 1973., Bechtold., et al., 1978)

In case of mechanical ventilation, that means, when air supply in the fire room is artificially generated, in the heat balance calculation a fictitious ventilation factor is calculated. It indicates which window size and height is necessary for the fire room if there is real ventilation in the fire room where the same fresh air supply takes place as is provided in the mechanical ventilation. Since the fresh air supply in the fire room is a function of temperature and ventilation factor, the ventilation factor for an artificial air inflow is calculated according to Eq. 4.5 iteratively:

$$\dot{m}_a = f(T_g, v) \quad (4.5)$$

According to Böhm (1977), the ventilation factor in the case of a forced air supply results in from Eq. 4.6, where  $K$  is the cross-contraction number and  $\dot{m}_a$  is air flow rate into the fire room, while  $\mu$  is a functional factor of the hot gas temperature. Thus  $v_a$  represents the artificial ventilation.

$$v_a = A_w \sqrt{h_w} = \dot{m}_a / K \mu \quad (4.6)$$

In the present investigations, the definition of the ventilation parameter from Metz is applied (Arnault, 1973), only in the heat balance calculations the definition of this parameter according to Eq.4.6 is more advantageous, because the ventilation enters as a product in the implied equation systems to be solved.

#### 4.4 Technical fire protection design

Basically, a technical fire protection design can be done in two different ways first, according to the In DIN 4102 defined test method and on the other hand on the basis of a real fire or real hazardous fire. In the latter case, a distinction must be made between direct and indirect design procedure. In Fig 4.1 design methods are shown schematically.

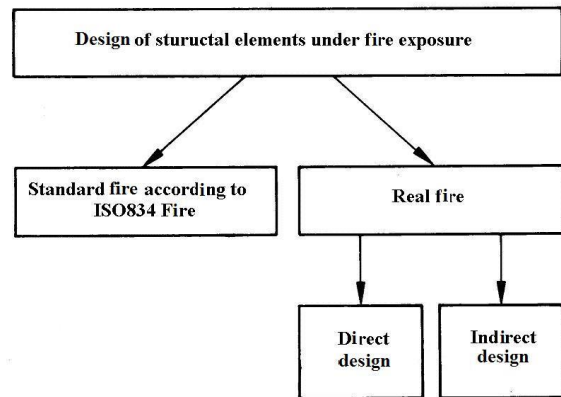


Figure 4.1: Methods of structural design

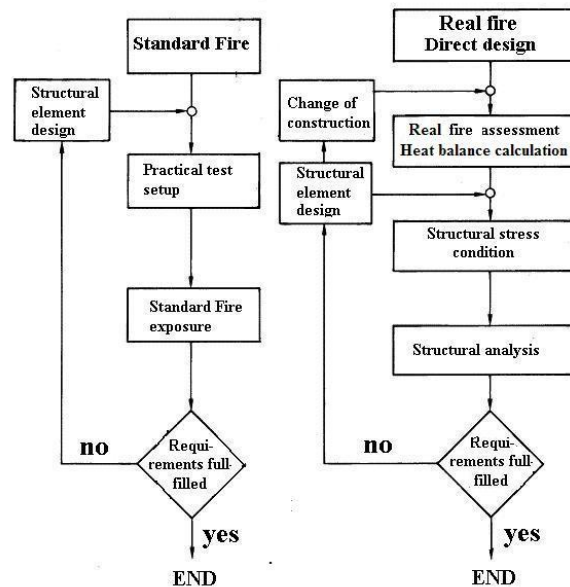


Figure 4.2: structural fire design according to DIN 4102 and for a real fire exposure

Fig. 4.2 (left side) shows the steps of such a design. Overall, only 4 design steps are required to achieve a fire protection assessment of a structural component. In contrast, the fire protection design is based on real hazardous fire, which is also shown on Fig. 4.2 (right side). Since this type of design can only be

realized in practice to a limited extent, there are theoretical investigations and analyses at the principal of this procedure (DIN 18230).

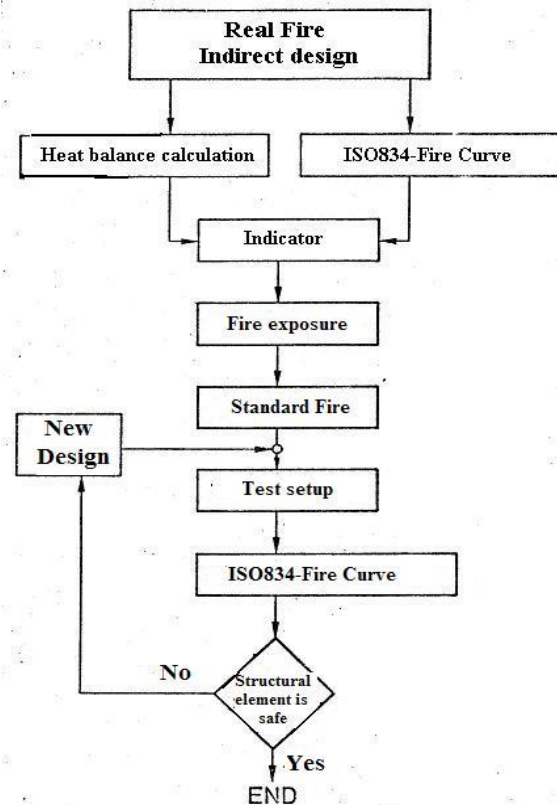
In principle, such a design requires three fundamentally different design procedures, however, two of them are directly coupled with each other, which makes the use of this method extremely difficult:

The first section includes a heat balance calculation (Haksever, 1989) for the assessment of the temperatures to be expected in the event of a real fire.

The second design section is used to determine the stresses in the structure due to the calculated fire development. In this case the loads, all thermally induced changes must be taken into account in the calculation.

The third dimensioning section is a structural analysis. On the basis of this analysis, the applicability of the structure is decided. If the design is not safe, the calculation must be repeated with changed parameters. Obviously that type of fire protection design is exceptionally complicated. The design method depends on many influencing factors and does not seem to be very practical. It is therefore appropriate to couple the methods for standard and real fire exposure with each other in order to ensure that the experience gained in standard firing tests can be applied furthermore for real fires also. Such a possibility is given by the indirect design in fact, as illustrated in Fig. 4.3:

The starting point of the design is, on the one hand, a heat balance calculation in the fire section and, on the other hand, fire exposure according to ISO834.



**Figure 4.3:** Indirect design procedure in case of a real fire

From the standard fire on the one hand and the heat balance calculation on the other hand, imply really different fire stresses that cause certain fire effects in the structural members. These fire stresses or effects can be correlated by measurements. In this connection for the real fire a so-called equivalent fire duration is determined, by which the real fire is compared in its effects with the effects of the standard fire of certain duration on a certain structural member which is called as indicator member. So it will be possible that the subsequent assessment can be made on the basis of standard fire. The real fire is thus attributed to the standard fire via an indicator structural member (Pettersson, 1973).

4.4 Definition of the equivalent fire duration

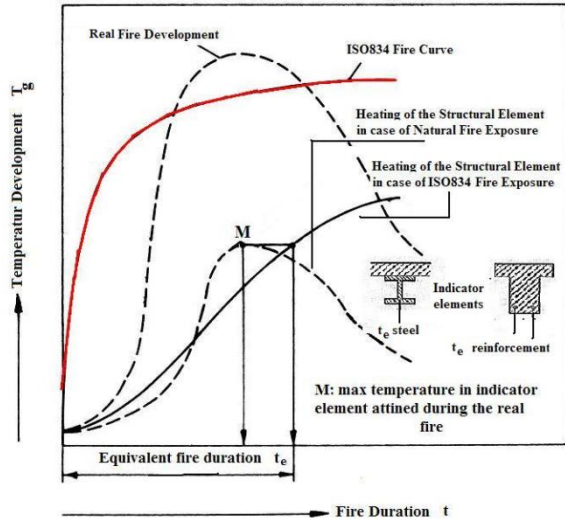


Figure 4.4: Definition of the equivalent fire duration

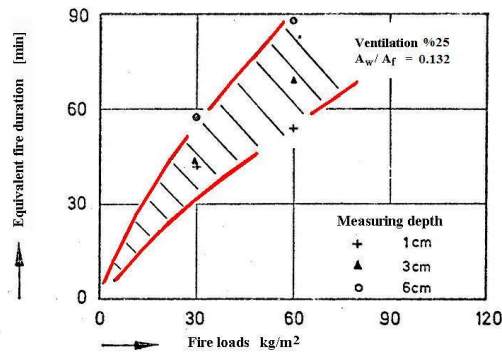


Figure 4.5: Influence of the measuring depth on the equivalent fire duration (Bechtold., etal, 1978, in German)

Fig. 4.4 shows this assessment procedure. By equating the maximum fire effects, which in real fires have occurred, with the fire effects that occur in standard fire after certain fire duration, the equivalent fire duration  $t_e$  determined.

4.5 Determination of the equivalent fire duration for a real fire exposure

The term "equivalent fire duration" implies that the same equivalent fire durations must always be obtained when assessing a real fire according to different indicator members and criterion's. In principle, it should be noted that it has hitherto not been possible to find an indicator member which is representative of all structural members and which

covers the entire range of fire effects in an appropriate way.

In addition to the building material influences in the determination of  $t_e$  also the measuring arrangement play a significant role. On the basis of the Lehrter Experiments (Bechtold, etal, 1978), for example, by means of the temperatures obtained at 24 cm thick reinforced concrete columns the equivalent fire duration for two fire loads were determined. It turned out the measuring points at different measuring depths show distinct differences with regard to  $t_e$  (s. Fig. 4 .5).



Figure 4.6: Indicator element

Fig. 4.6 shows the indicator element used to determine the equivalent fire duration in the furnace tests. Investigations have thus shown that the definition of equivalent fire duration on the basis of indicator members can be useful in the assessment of the effects of real fires.

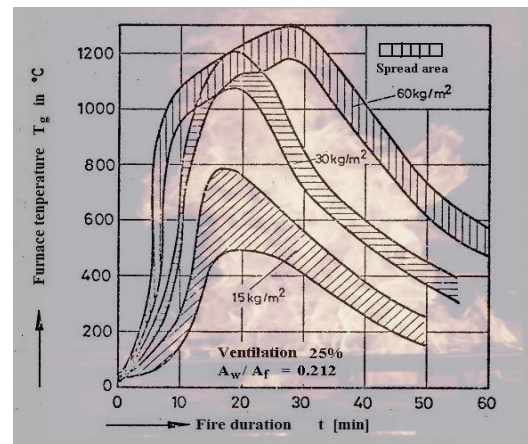


Figure 4.7: Distribution of the fire room temperatures On the other hand, it is clear that the use of this method by means of the temperature criterion implies some certain limits, because different structural members

and building materials in a real fire can also show quite different fire behavior. Beside that the temperature distribution in an enclosure is not uniform as illustrated in Fig. 4.7.

From the distribution area of the fire room temperatures it can be seen that the temperature gradient in the fire room exceeds 200 °C, which cause the definition of an "average fire room temperature" extremely difficult. The difficulty arises from the unsteady development of the fire room temperatures; which means that the temperatures in the ceiling area are much higher than in the area of the floor. It is therefore doubtful to what extent average fire temperatures can reflect temperatures of the actual fires in the entire fire area.

**5. Structural behavior RFCs under real fire exposure**

*5.1 Introduction*

In recent years, some fundamental contributions to the calculation of reinforced concrete columns at room temperature have been published in papers by (Kasperek, 1968, Quast, et al., 2008., Quast, 2009 and Liermann, 1972). Essentially, the methods are based on leading back the problem to an inelastic-theoretical solution by an iterative calculation of the local, stress-dependent bar stiff nesses.

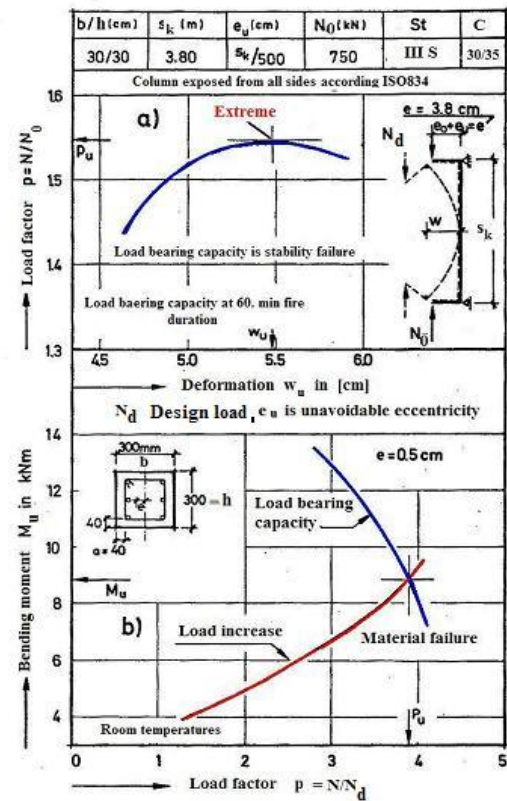
For the standard fire case, i.e. at continuously increasing fire room temperatures, load bearing calculations were presented by (Twilt, 1975, Kordina. 1974, 1977, Lie, 1972, Bizri, 1973, Klingsch, 1978, Anderberg, 1976) and in recent times by the authors given in literature review in section 1.1. In these calculations, the load bearing capacity of the reinforced concrete columns was determined by variation

- eccentricity of design load
- of system length of the column
- of service load combination of the column

The fire resistance time is attained when an equivalent static state exists between the design loads and the load bearing capacity. The load-bearing criterion on a

reinforced concrete column is thus separated into two aspects by introducing a new "time parameter" for the fire case.

- a) Once an optimization process can be determined for specific fire duration  $t_f$  for the imposed loading. The type of load bearing capacity, as a material or a stability failure, is displayed in this case over the course of a load-deformation diagram.
- b) On the other hand, fire resistance time  $t_{f,u}$  can be determined the for a particular design load combination. Also in this case is when attaining the fire resistance time to expect a material or a stability failure.



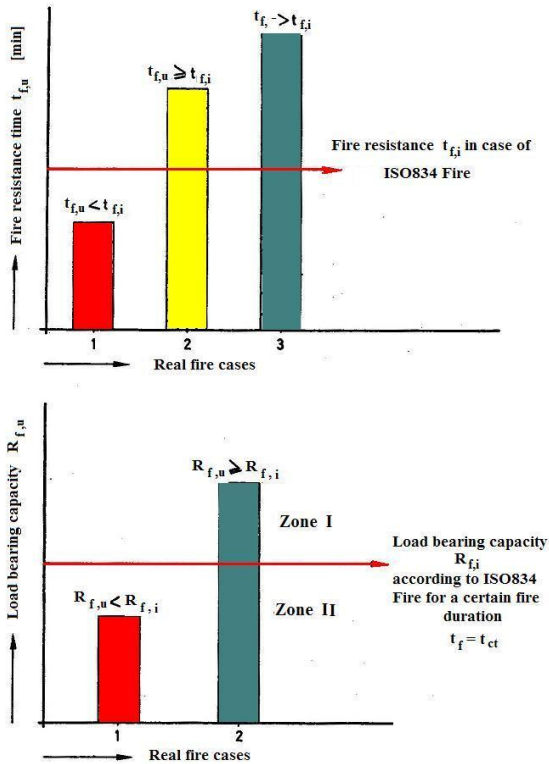
**Figure 5.1:** Load bearing condition of a RFC in case of different failure modes

Fig. 5.1 illustrates the load bearing failures of a RFC in case of fire. The real fires, however, are a special case with regard to the criterion given above. In particular, for the real fire, there are two other aspects:

- 1- If an optimization for the time parameters is possible, then the fire resistance time of the

RFC is achieved in each case within total fire duration of a real fire. The RFC cannot withstand this real fire.

- 2- If no optimization of the time parameters is possible, there is no fire resistance time for the RFC. It withstands such a real fire.



**Figure 5.2:** Fire resistance time and the load bearing capacity situations under real fire exposure

A real fire does not necessarily result in a failure of RFC. If a RFC does not fail, it is of course not possible to speak from fire resistance time  $t_{f,u}$ .

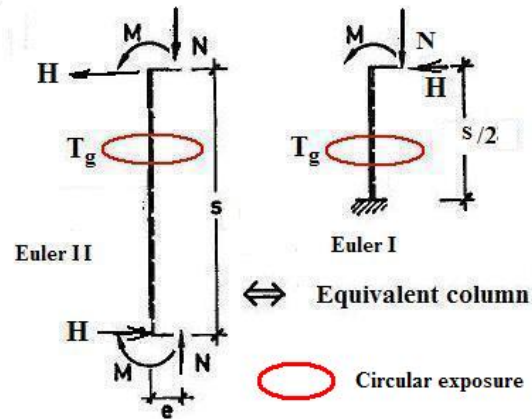
In the event of a failure of the RFC in a real fire, the fire resistance times may vary greatly depending on the development of the real fire:

When a RFC is exposed to a real fire, higher, equal or lower fire resistance times  $t_{f,u}$  and load bearing capacities  $R_{f,i}$  may result compared to ISO834 Fire. In Fig. 5.2 this equilibrium state is shown schematically.

### 5.2 Determination of the fire resistance time and Load bearing capacities of RFC for a representative system

The theoretical stress and strain calculation of reinforced concrete structures under a real fire attack is

made extremely difficult by the relationships described in previous sections. An investigation of RFC with simultaneous interaction with the entire structure would provide better clarity on the structural behavior; however, in the context of this paper, it is neither economical nor practicable because of the extraordinarily high level of calculation and time consuming work. In the present investigations, therefore, the RFC was separated from the entire structure and considered in isolation. A rearrangement of the cross section forces, as it can occur in highly statically indeterminate systems was not considered. Therefore, an Euler II RFC was chosen which is jointed on the supports (s. Fig. 5.3).



**Figure 5.3:** Statically system and imposed loads

For the theoretical analysis of the system, an equivalent structure is assumed as cantilever column and the load boundary conditions are taken over. However, in order to obtain practical results, the system length, the load eccentricity and the external loading of the RFC investigated in Section 6 were selected so that their fire resistance time is comparable to an equivalent RFC in Euler case I. In other words, the fire resistance time of these RFC will be approximately the same as those expected for an increased system length, but with same imposed loads, according to Euler case II, so that a certain continuity effect and cross section force redistribution are taken into account. This principle is formulated by Eq. (5.1), where the upper indices indicate the Euler the states while  $s_k$  is the buckling length of RFC:

$$t_f^I(s_{k,I}) \leq t_f^{II}(s_{k,II}) \quad (5.1)$$

Thus in order to determine the fire resistance time and the load bearing capacity in case of fire, a subsystem is chosen of which load bearing and deformation behavior approximates a section of a RFC from more storey frame work. This subsystem is given with the static boundary conditions in Fig. 5.4.

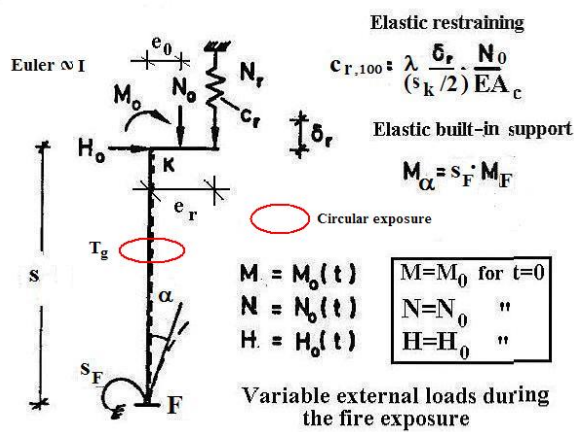


Figure 5.4: Imposed loads and boundary conditions of RFC

The calculation program is written in such a way that all boundary conditions given in Fig. 5.4 can be taken into account. These boundary conditions are described as follows:

- An elastic fixing at the column end is ensured by a spring constant, for a total fixing "s<sub>F</sub>" can be set equal to 1.0.
- Statically loads act on the column head, as plane external forces
- As an axial boundary condition, an elastic restraining is provided. For total restraining λ is set equal 0.
- Depending on the simulated entire system, the eccentricity of the external vertical loads and the restraining forces may be different
- All imposed loads, except time-varying forces in case of restraining, can act as a function of the fire.

In this research work, in order to limit the number of parameters, elastic restraining and temporally varying external loads are not taken into account. In the following section the fire protection design of a RFC of a concrete hall building is exemplary presented and the equivalent fire duration of a RFC bound to fire loads using **statically criterion** is graphically illustrated.

## 6. Design of a RFC for real fire exposure

### 6.1 Design principles

Traditionally, fire behavior of fire exposed structural systems is analyzed with respect to the ISO Fire tests. For many structural systems this constitutes the only way for obtaining the information for a structural fire protection design. In spite of this criterion, standard fire tests can be extremely discussed. Consequently, a considerable variation can arise in the determination of real fire behavior of structural members and systems. Because of these problems and to achieve solutions with greater economy and more uniform safety, there is a strong need to use analytical structural fire engineering design methods.

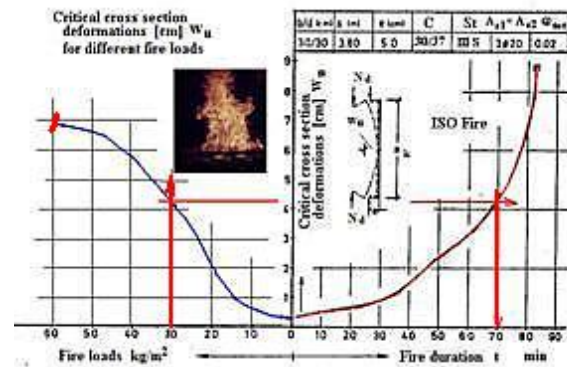


Figure 6.1: Development deformations of the critical cross section of a RFC in real fire exposures

The fire protection regulations for structures based on such standard fire tests ensure a certain level of safety. However, the possibility to construct in an economic frame is limited by the current adjustments. This restriction is expressed in particular fact that the current fire protection design only knows the relatively unfavorable temperature development of the standard fire curve, on the other hand, however, the fire exposure in

real fires is sometimes less intense, as they have a lower or better fire room temperature development.

### 6.2 Deformation behavior of RFC in a real fire

The deformation behavior of a RFC is particularly distinct by the development of the deformations in the critical cross section.

Fig. 6.1 shows the critical deflections of a RFC after a standard fire exposure as well as after various real fires. The mechanical data of the investigated RFC is given in the upper table of the Fig. 6.1. The prediction of RFC results in a fire resistance about 83 minutes in standard fire case. On the left side of the picture, the head deformations of the same RFC were determined under different real fire exposures. As parameter, different fire loads were chosen. In the case of a real fire, it can be observed that the deformations of this RFC increase with the fire loads respectfully. Furthermore, it has been observed that the tested RFC for the fire loads of 15 and 30 kg/m<sup>2</sup> (equivalent wood cribs) under the specified ventilation conditions showed no failure condition. This occurs only at a fire load of 60 kg /m<sup>2</sup>. From Fig. 6.1 the equivalent fire durations can also be obtained. For a 30 kg/m<sup>2</sup> wood crib fire, for example, it results in definitely equivalent fire duration from 70 min.

In Fig. 6.2 equivalent fire duration is illustrated as function of the fire loads (Arnauld, etal. 1973). The temperature criterion is used in order to establish a functional connection. From Fig. 6.2 it can be seen that there is a linear connection between the fire loads and the equivalent fire duration. Taking into account this linear function for 30 kg/m<sup>2</sup> wood cribs as fire loads it results in equivalent fire duration from:

$$A_{T_w} = 2560 - 63 = 2497 \text{ m}^2$$

$$A_w = 63 \text{ m}^2$$

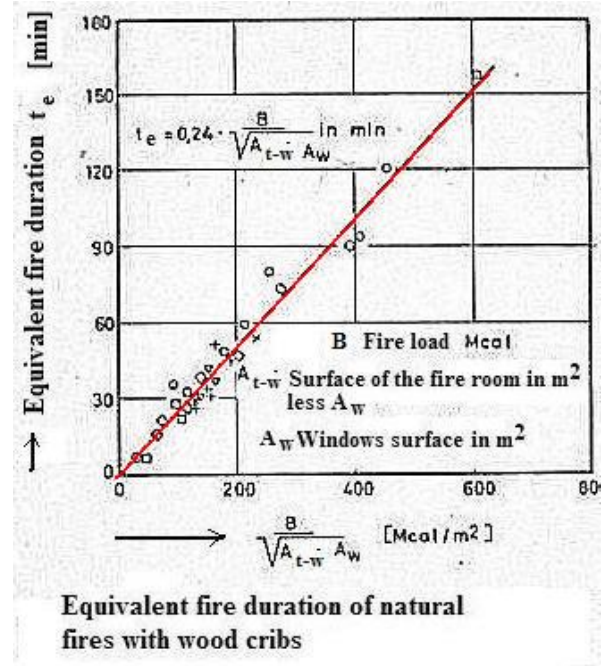
$$\text{Total fire load } Q_{t,c} = 24 \cdot 10^3 \cdot 4.0 = 96000 \text{ Mcal}$$

$$t_e = 0.24 \cdot 96000 / (2497 \cdot 63)^{0.5} \text{ (s. Fig. 6.2)}$$

$$t_e = 58.1 \text{ min.}$$

It is being clear that the temperature criterion can give different results bound to the assumptions and the

indicator element, while statically criterion determines definitely unique equivalent fire duration (s. Fig. 6.1).



**Figure 6.2:** Equivalent fire duration with respect to the temperature criterion (Arnauld, etal. 1973)

## 6. Summary and Conclusions

The key to the solution of fire design problems is the capability to predict the development of enclosure fires as well as their action on constructional elements. Up to now a certain fire safety design is reached in buildings by taking into account the ISO 834 – Fire Curve both in experiments and theoretical investigations. However a real fire development differs in enclosures essentially from the standard fire and consequently the focus of attention has in recent time been in a growing sense on the effects of real fires within building compartments, because the results show that in structural members which are exposed to real fires, limiting the attention to the heating phase is not sufficient, as the maximum temperature in the reinforcements may be reached long (even hours) after the onset of cooling, leading to delayed failure.

This research work presents the theoretical investigations to predict the fire behavior of RFC in a



not prevented real fire. The main purpose of the analyses was concentrated on fire growth in an enclosure, development of hot gas temperatures and heat balance calculations coupled with load bearing calculations under non steady heating conditions.

In particular, the applicability of the classic concept of "equivalent fire duration" was critically discussed in the research work. It could be shown that the "equivalent fire duration" is also decisively determined by the static boundary conditions of the RFC. By applying the various criterion for determining equivalent fire duration it could be presented that these criterion are divided into groups and the groups each result in about the same equivalent fire duration.

It has been shown that the application of the classical temperature criterion for the determination of the equivalent fire duration does not always have to provide a meaningful and reasonable result for engineering practice. It was therefore also the mechanical criterion used in this regard. However, equivalent fire duration needs a more detailed investigation in order to find out its dependency on structural dimensions of RFCs and to show a relationship as in the Fig. 6.2.

In Part II of the research work the definition and the determination of boundary fire loads  $q_{f,B}$  of RFC will be presented (Boundary fire loads of reinforced concrete columns exposed to a real fire, Part II). Part II will show the certain fire loads as wood cribs, which results in real fire case equivalent fire resistance as the ISO834-Fire. Beyond the boundary fire loads the fire resistance of RFC is greater than the fire resistance in case of a ISO834 fire case.

#### 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.

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#### Notations

##### Capital letters

$A_c$	Concrete cross-section	$\text{mm}^2$
$A_T$	Total inner surface area of the enclosure	$\text{m}^2$
$A_w$	Window and door areas	$\text{m}^2$
$A_{T_w}$	Surface of the fire room less $A_w$	$\text{m}^2$
$A_F$	Front area of the wall surface	$\text{m}^2$
$A_{s,1}$	Tensile reinforcement	$\text{mm}^2$
$A_{s,2}$	Compression reinforcement	$\text{mm}^2$
$B$	Fire load (s. Fig. 6.2)	Mcal
$C$	Concrete sort	$\text{N/mm}^2$
$C_E$	Spring constant	%
$E_u$	Minimum calorific value of fire loads	kWh, kCal
$E$	Modulus of elasticity	$\text{N/mm}^2$
$E_u$	min Energy per kg fire load	kWh/kg, kJ/kg
$F$	Fire safety class (DIN4102, ISO834)	min
$F_{30} \dots F_{120}$	120 min. fire resistance times	min
$G_f$	Permanent loads	kN
$H_0$	Initial horizontal load	kN
$H$	Horizontal load (shear force)	kN
$M$	Bending moment	kNm
$M_F$	Built-in bending moment	kNm
$M_\alpha$	Elastic built-in bending moment	kNm
$M_0$	Initial bending moment	kNm
$M_0(t)$	Time dependent bending moment	kNm
$M_u$	Bending moment capacity	kNm
$N$	Axial load	kN
$N_0$	Initial axial load	kN
$N_d$	Axial design load	kN
$N_0(t)$	Time dependent axial load	kN
$N_r$	Restraining force	kN
$P_{f,a}$	Accidental load combination	
$P_{e,f}$	External stress condition	kN, kNm
$Q_f$	Variable loads	
$Q_{f,T}$	Actions due to fire exposure, e. g. restraining	
$Q_{f,T}$	Total fire load as wood cribs	$\text{kg/m}^2$
$Q_{f,w}$	Total fire load	kWh
$Q_{f,C}$	Total fire load	Cal
$R_f$	Load bearing capacity under fire action	kN, kNm
$\dot{R}$	Burning rate of fire loads	$\text{kg}/(\text{m}^2 \cdot \text{h})$
RFC	Reinforced concrete column	
$St$	Steel sort	
$T_0$	Room or initial temperature	$^\circ\text{C}$ , K
$T_g$	Hot gas temperature	$^\circ\text{C}$ , K
(WCR)	Wood cribs	

**Small Letters**

b	Width of the cross section	cm
$c_i$	Insulation factor	
c	Axial concrete cover	cm
$c_{r,100}$	Free Restraining for $t=100$ min. controlled with linear increment	%
e	Eccentricity	cm
$e_0$	Design eccentricity	cm
$e_u$	Unavoidable eccentricity	cm
h	Height of the cross section	cm
$h_w$	Average window height	m
$e_r$	Eccentricity of the restraining force	cm
d	Concrete cover	cm
m	Conversion factor	-
$\dot{m}_a$	Air mass present in fire compartment	kg/h
v	Ventilation (Eq. 4.3)	$m^{1/2}$
$v_{f,i}$	Ventilation factor (Eq. 4.4)	%
$v_a$	Artificial air ventilation (Eq. 4.6)	$m^{5/2}$
$q_{f,G}$	Fire load density on the ground surface	$kg/m^2$
$q_{f,B}$	Boundary fire load density on the ground surface	$kg/m^2$
$q_{f,S}$	Boundary fire load density on the total surface of enclosure	$kg/m^2$
$q_{f,C}$	Safe fire load density on the total surface of enclosure	$kg/m^2$
$q_{f,o}$	Upper boundary fire load density on the ground surface as wood cribs (WCR)	$kg/m^2$
$q_{f,e}$	Lower boundary fire load density on the ground surface as wood cribs (WCR)	$kg/m^2$
$q_{f,W,G}$	Fire load density on the ground surface	$kWh/m^2$
$q_{f,G}$	Fire load density on the ground surface (WCR)	$kg/m^2$
$q_{f,W,T}$	Fire load density on the total surface	$kWh/m^2$
$q_{f,T}$	Fire load density on the total surface (WCR)	$kg/m^2$
$q_{f,W,T}$	Fire load density on the total surface	$kWh/m^2$
s	Column height of RFC	m
SF	Spring factor	%
$t_e$	Equivalent fire duration	min
$t_{e,q}$	Equivalent fire duration for a fire load	min
$t_f$	Fire resistance	min
$t_{f,e}$	Necessary fire resistance when an extinguishing system in the building is present	min
p	Relative axial load	%
r	Oxygen consumption of stoichiometric burning for unit fire load	(-)
t	Time	min
$t_{f,e}$	Fire extinguishing factor	
$t_{f,T}$	Total fire duration	min
$v_x$	Propagation of flame front in x-direction	m/min
$v_y$	Propagation of flame front in y-direction	m/min
$w_u$	Deformation of RFC at critical section	mm
w	Heat venting factor	

**Symbolic letters**

$\alpha$	Prozentual oxygen content in fire room	%
$\delta_r$	Initial axial deformation due to $N_0$	mm
$\mu$	Conversion factor	$m^{5/2}.h/kg$
v	Ventilation factor	$m^{1/2}$
$v_F$	Front ventilation	%
$v_a$	Artificial ventilation	$m^{5/2}$
$\Psi_i$	Combination coefficient	
$\lambda$	Elastic restraining coefficient	
$\gamma$	Firefighting factor	
$\omega_{Tot}$	Total reinforcement	%
$E_u$	Heat energy of the fire load	$kWh/kg$

The other notations are defined where they appear in the text.

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